Road Design and Construction In Sensitive Watersheds

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CHAPTER 1

DEFINITION AND SCOPE OF PROTECTIVE MEASURES FOR ROADS

1.1 General Introduction

This handbook was written as a guide to reducing environmental impacts of forest roads in mountain watersheds and is intended to be used by professional land managers involved in decisions regarding upland conservation, watershed management, and watershed rehabilitation. Its purpose is to (1) identify potential threats to water quality from the construction and maintenance of roads, and (2) recommend procedures, practices, or methods suitable for preventing, minimizing, or correcting erosion problems. It discusses proper planning, reconnaissance, road standard development, erosion control, slope stabilization, drainage design, and maintenance techniques as well as cost analysis procedures that can be applied in the design, construction, and maintenance of forest roads. Specific questions relating to road design procedures, general layout and construction methods can be found elsewhere, and it is left to the reader to locate sources for that type of information.

The types of roads considered here would generally be built to withstand low to moderate traffic levels for purposes of providing access for residents, timber harvesting, reforestation, rangeland management, and other multiple use activities where access to upland areas is required. Availability of some basic heavy equipment, such as bulldozers and graders, is assumed. Whenever possible, emphasis will be given to labor- rather than machinery-intensive methods. However, livestock or human labor may often be substituted wherever machines are mentioned and may in fact be preferable to the use of machines by reducing environmental impacts during operations. This is especially true in the case of road maintenance. Production rates in most cases will be much slower and should be considered when developing cost estimates.

Much of the information cited here reflects years of research and experience gained from various sources. As such, the material presented must be evaluated in light of local geographic, economic, and resource needs; it cannot and should not be a substitute for regional knowledge, experience, and judgment.

1.2 Interaction of Roads and Environment

Forest roads are a necessary part of forest management. Road networks provide access to the forest for harvests, for fire protection and administration, and for non-timber uses such as grazing, mining, and wildlife habitat. New road construction is required to enter previously uninhabited areas or underutilized lands, and will continue to provide access in order to properly manage those lands.

Construction and use of forest roads result in changes to the landscapes they cross. Of all the types of silvicultural activities, improperly constructed and inadequately maintained "logging roads" are the principal human-caused source of erosion and sediment. (US Environmental Protection Agency, 1975) Road failures and surface erosion can exert a tremendous impact on natural resources and can cause serious economic losses because of blocked streams, degraded water quality, destroyed bridges and road rights-of-way, ruined spawning sites, lowered soil productivity, and property damage.

Erosion is related, among other things, to:

1. <u>Physical factors.</u> These would include soil type, geology, and climate (rainfall).

2. Road density. The total length of roads per unit area of watershed is termed road density. Erosion rates are directly related to the total length of roads in a watershed, as shown in Figure 1. A road network of approximately 30 to 40 m/ha is considered optimal for most management purposes.

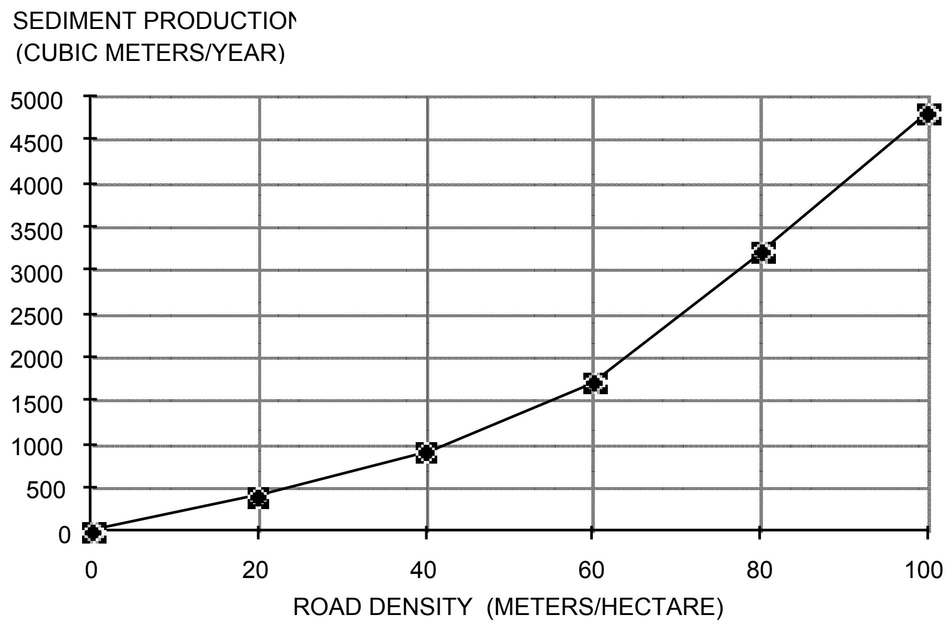


Figure 1 Sediment production in relation to road density (Amimoto, 1978).

3. Road location. The location of the road in relation to slope, stream channels, and sensitive soils has a direct effect on the amount of sediment reaching the stream.

4. Road standards and construction. Designed road width, steepness of cut banks or road fills, methods of construction, and drainage installations will directly affect the area of disturbance and potential for failure following road construction.

Causes of erosion may include: (1) removal or reduction of protective cover, (2) destruction or impairment of natural soil structure and fertility, (3) increased slope gradients created by construction of cut and fill slopes, (4) decreased infiltration rates on parts of the road, (5) interception of subsurface flow by the road cut slopes, (6) decreased shear strength, increased shear stress, or both, on cut and fill slopes, and (7) concentration of generated and intercepted water. (Megahan, 1977)

The severity of the impact is closely related to the overall land surface exposed by roads at a given time, the drainage density of the watershed (degree to which stream courses dissect the land), slope (gradient, length, shape, and position on the slope), geologic factors (rock type, strength, and hardness, bedding planes, faulting, subsurface drainage), and climate. Generally, the greater the intensity of storm events and the more drainage dissects the landscape, the more acute the necessity to plan for avoiding water quality impacts in constructing and stabilizing roads. In central Idaho, Megahan and Kidd (1972) observed sediment production rate increases of 770 times per unit area of road prism for a six year study period. Although surface erosion following road construction decreased rapidly with time, the major impact occurred from one road fill failure after a single storm event. Other research in the region points to mass failures as the most serious erosion process contributing to reduced water quality on forest lands. (Swanson and Dyrness, 1975; Fredriksen, 1970; Dyrness, 1967; Megahan, 1967)

It is well documented that water quality impacts caused by roads can best be dealt with by prevention or by minimizing their effects, rather than attempting to control damage after it has occurred (Brown, 1973; Megahan, 1977). This can best be done by minimizing the total mileage of roads through proper planning, properly locating roads in relation to topography and soils, minimizing exposed constructed road surfaces by proper road standard selection and alignment, and using proper road construction and culvert installation techniques.

Additionally, these same researchers have found that the majority of sediment generated on roads occurs within the first year following construction. This would emphasize the need for concurrent erosion control measures during and immediately following construction. Merely seeding bare soil surfaces may not be sufficient to curb soil erosion.

1.3. Erosion Processes.

Recognition of the type of erosion occurring on an area and knowledge of factors controlling erosion are important in avoiding problem areas and in designing control structures. Erosion can be broadly categorized as surface erosion and mass erosion. Mass erosion includes all erosion where particles tend to move *en masse* primarily under the influence of gravity. It includes various types of landslides and debris torrents. Surface erosion is defined as movement of individual soil particles by forces other than gravity alone such as overland flow or runoff, raindrop impact, and wind. Dry creep or dry ravel, the movement of individual particles resulting from wetting and drying, freezing and thawing, or mechanical disturbance, is considered a surface erosion process.

Surface erosion is a function of three factors: (1) the energy available from erosion forces (raindrop splash, wind, overland flow, etc.), (2) the inherent erosion hazard of the site (soil physical and mineralogical characteristics, slope gradient, etc.), and (3) the amount and type of cover available to protect the soil surface (vegetation, litter, mulch, etc.). Mass erosion is controlled by the balance between stabilizing factors (root strength, cohesion) and destabilizing factors (slope gradient, seepage forces, groundwater) operating on a hillslope. Another way of stating this relationship is the relative magnitude of shear strength versus shear stress. When shear stress is less than or equal to shear strength, the slope will remain stable; when stress exceeds strength, the slope will fail.

Factors that might be considered when assessing the impact of road construction and subsequent development of a site might include:

<u>Soil and Geology</u>
- soil - physical and chemical characteristics
- geologic conditions (stratigraphy, mineralogy, etc.)
- groundwater occurrence and movement
- slope stability
- seismic characteristics

<u>Climate and Precipitation</u>
- start and end of rainy season
- intensity and duration of storms
- occurrence of summer storms
- seasonal temperature
- frost-free period
- wind erosion
- snow melt runoff

rainfall runoff before and after development

Topography

slope angle
slope aspect
slope length
density and capacity of drainageways
suitability of sites for sediment basins

Vegetative Cover

type and location of native plants
fire hazard
ease in establishing vegetative cover
adequacy of existing plants in reducing erosion

Manner of Development

percent grade and layout of roads
density of roads
distribution of open space
structures affecting erodible areas
number of culverts, stream crossings
size of areas, duration and time of year when ground is left bare

1.4 Assessment of Erosion Potential

1.4.1 Surface Erosion

Soil properties important in the evaluation of a site for its resistance to erosion include particle size, permeability, water retention characteristics, compressibility, shear strength, void ratio or porosity, shrink-swell potential, liquid limit and plasticity index. Soil developmental characteristics such as horizonation, depth to bedrock or parent material, and depth to seasonal water table are also helpful. Other factors which influence erodibility include vegetation characteristics (foliage density, height above soil surface, rooting characteristics) and litter cover. Raindrop energy may be partially dissipated by overstory or understory vegetation, thereby reducing the amount of energy transmitted directly to the soil surface. The litter layer contributes the most in protecting the soil from erosion by absorbing the net energy that finally reaches the surface after filtering through vegetation canopies. Any surface runoff that may occur on a natural soil surface will generally take place below the litter layer, however, the flow velocity is very slow because of the tortuosity of the path that the water must take to pass through the litter. Particle detachment, therefore, is unlikely where good litter cover is present.

In order to discuss soil characteristics in a uniform and accurate manner, several classification systems have been developed which provide guidance in identifying a particular soil's desirability or value for various engineering uses. The Unified Engineering Soil Classification system was developed as a method of grouping soils for military construction and is shown in Table 1. Other classification systems include the United States Department of Agriculture Soil Textural Classification system and the American Association of State Highways and Transportation Officials (AASHTO) system.

A guide for evaluating soil erosion potential in the field can be made by visual inspection of the soil and by such techniques as shaking, patting, and kneading. Subsurface samples can be extracted with the use of hand augers or shovels. Classification of soils into erodibility groups based on the Unified System is presented in Table 2. A discussion of erodibility in relation to cross-drain spacing requirements is presented in Chapter 3.4.3.

Several methods are available in order to evaluate the potential for soil loss from surface erosion, and two different approaches have been utilized in estimating surface soil loss. The first of these is empirical in nature using predictive equations developed from analysis of "real" data. The second consists of the use of process models--models developed through analysis of cause and effect relationships. The empirical procedure most commonly used is the Universal Soil Loss Equation (USLE) which was originally developed for use on Midwestern United States agricultural soils and has since been modified for use in forest environments. The Modified Soil Loss Equation (MSLE) uses a vegetation management factor (VM) to replace the cropping factor (C) and the erosion control practice factor (P) used in the USLE (U. S. Environmental Protection Agency, 1980). The MSLE is:

$$A = R \quad K \quad L \quad S \quad VM$$

where:

A = estimated average soil loss per unit area in tons/acre for the time period selected for R (usually one year)

R = rainfall factor, usually expressed in units of rainfall erosivity index (EI) and evaluated from an iso-erodent map

K = soil erodibility factor, usually expressed in tons/acre/EI units for a specific soil in cultivated continuous fallow, tilled up and down the slope

L = slope length factor expressed as the ratio of soil loss from the field slope length to that from a 72.6 foot (22.1 meter) length on the same soil, gradient, cover, and management

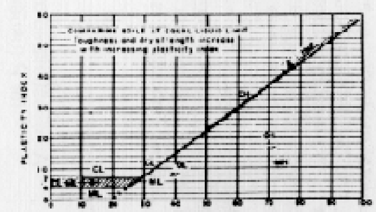
S = slope gradient factor expressed as the ratio of soil loss from a given field gradient to that from a 9 percent slope with the same soil, cover, and management

VM = vegetation management factor expressed as the ratio of soil loss from land managed under specific conditions to that from the fallow condition on which the factor K is evaluated.

Numerical values for each of the factors are based on research data and differ dramatically from one region to another, from one locality to another, and even from one field to another. However, approximate values for potential soil loss from a site may be calculated with the understanding that strict adherence to the assumptions made in selecting values for individual factors is required if a reasonable answer is to be obtained. Even so, errors in the range of an order of magnitude of the true erosion rate are not uncommon. A procedural guide in using the MSLE is presented in Chapter IV, An Approach to Water Resources Evaluation of Non-Point Silvicultural Sources, US Forest Service, 1980.

UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

FIELD IDENTIFICATION PROCEDURES (Excluding particles larger than 3 inches and basing fractions on estimated weights)						GROUP SYMBOLS *	TYPICAL NAMES	INFORMATION REQUIRED FOR DESCRIBING SOILS	LABORATORY CLASSIFICATION CRITERIA	
COARSE GRAINED SOILS. More than half of material is larger than No. 200 sieve size. (The No. 200 sieve size is about the smallest particle visible to the naked eye)	GRAVELS. More than half of coarse fraction is larger than No. 4 sieve size. (For visual classification, the ¼" size may be used as equivalent to the No. 4 sieve size)	CLEAN GRAVELS (Little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes.			GW	Well graded gravels, gravel-sand mixtures, little or no fines.	Give typical name; indicate approximate percentages of sand and gravel; max. size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses.	Use grain size curve in identifying the fractions as given under field identification. Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% GW, GP, SW, SP; More than 12% GM, GC, SM, SC; 5% to 12% Borderline cases requiring use of dual symbols.	$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing.			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines.			Not meeting all gradation requirements for GW
		GRAVELS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)			GM	Silty gravels, poorly graded gravel-sand-silt mixtures.	For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.		Atterberg limits below "A" line, or PI less than 4. Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			Plastic fines (for identification procedures see CL below)			GC	Clayey gravels, poorly graded gravel-sand-clay mixtures.			Atterberg limits above "A" line with PI greater than 7
	SANDS. More than half of coarse fraction is smaller than No. 4 sieve size.	CLEAN SANDS (Little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes.			SW	Well graded sands, gravelly sands, little or no fines.			$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing.			SP	Poorly graded sands, gravelly sands, little or no fines.	EXAMPLE: Silty sand, gravelly, about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains, coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)		Not meeting all gradation requirements for SW
		SANDS WITH FINES (Appreciable amount of fines)	Non-plastic fines (for identification procedures see ML below)			SM	Silty sands, poorly graded sand-silt mixtures			Atterberg limits below "A" line or PI less than 4. Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols.
			Plastic fines (for identification procedures see CL below)			SC	Clayey sands, poorly graded sand-clay mixtures			Atterberg limits above "A" line with PI greater than 7
FINE GRAINED SOILS. More than half of material is smaller than No. 200 sieve size.	IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE		DRY STRENGTH (CRUSHING CHARACTERISTICS)	DILATANCY (REACTION TO SHAKING)	TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)					
	SILTS AND CLAYS. Liquid limit less than 50		None to slight	Quick to slow	None	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses.	PLASTICITY CHART FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS	
			Medium to high	None to very slow	Medium	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.			
			Slight to medium	Slow	Slight	OL	Organic silts and organic silt-clays of low plasticity.	For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.		
	SILTS AND CLAYS. Liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.			
			High to very high	None	High	CH	Inorganic clays of high plasticity, fat clays.	EXAMPLE: Clayey silt, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)		
			Medium to high	None to very slow	Slight to medium	OH	Organic clays of medium to high plasticity.			
HIGHLY ORGANIC SOILS			Readily identified by color, odor, spongy feel and frequently by fibrous texture.			Pt	Peat and other highly organic soils.			



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS

* Boundary classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.
** All sieve sizes on this chart are U.S. standard.

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Unified Soil Classification Chart (Sheet 1 of 2) from drawing no. 103-D-347.

table 5 Unified Soil Classification System (adapted from U.S. Department of Interior, Bureau of Reclamation, Earth Manual , Denver)

Erosion Class		I	II	III	IV	V	VI	VII	VIII	IX	X
Erosion Index		10	20	30	40	50	60	70	80	90	100
Standard soil textures and unified system soil groups		SM	SM	Silt (unconsolidated (B)	Silt (consolidated (B)	Silty clay loam A)	Clay loam (A)	Loamy sand (C)	Coarse sand (C)	Fine gravel (C)	Rock (C)
		ML	ML	OL	OL	Silty clay (A)	Silt loam (A, B)	Sandy loam (B)	SW	SW	Cobble (C)
				MH	MH	Clay, varying with type, cohesiveness and compaction (A)			SP	SP	Gravel (C)
					CL	Sandy clay (B)	Sandy clay loam (B)		Sand (B, C)		GW, GP
						SC, GM OH, CH	CL, GM	GC			
Special cases; General names and descriptions.		Decomposed granodiorite (C)	Decomposed sandstone, e.g. (B,C)	Fine soils derived from rocks high in mica, (C)	Coarse soils derived from rocks high in mica (C)	Some volcanic ash or extremely fine pumice sometimes difficult to distinguish from residual soils (B)			"Shot" as found in "shot-load" (B, C)		Fractured loose basalt or shale (C)
		Highly decomposed granites (C)	Moderately decomposed granites (B)	Greasy decomposed rock high in clay (A)							Coarse volcanic cinders (C)
				Pumice, varying with location, particle size, denisty, topography, and compaction (B, C)							Bed rock (A)

*Erosion classes rate the soil textures and geologic types listed as if they contained 100 percent of the material specified. To place a soil mixture in the proper erosion class, multiply the estimated percent content of the various "Components" (rock, cobble, gravel, etc. and a given soil texture) by their respective erosion indices and add the results. The total indicates the erosion class of the mixture. (See text for examples)

Capital letters following a soil texture or geologic type indicate the infiltration class of the material as follows: (A) indicates nonporous materials; (B) indicates moderately porous materials; (C) indicates highly porous materials.

table 6 Guide for placing common soil and geologic types into erosion classes. (Forest Soils Committee of the Douglas-fir Region of the Pacific Northwest, 1957)

1.4.2. Mass Soil Movement

Accurate models and data needed to predict mass soil movement over broad areas are currently lacking. A widely used technique involves the relatively simple planar infinite slope analysis described by Sidle (1985). This method is particularly useful when the thickness of soil is small in comparison to the length of slope and where the failure plane parallels the soil surface. The infinite slope model is illustrated in Figure 2 .

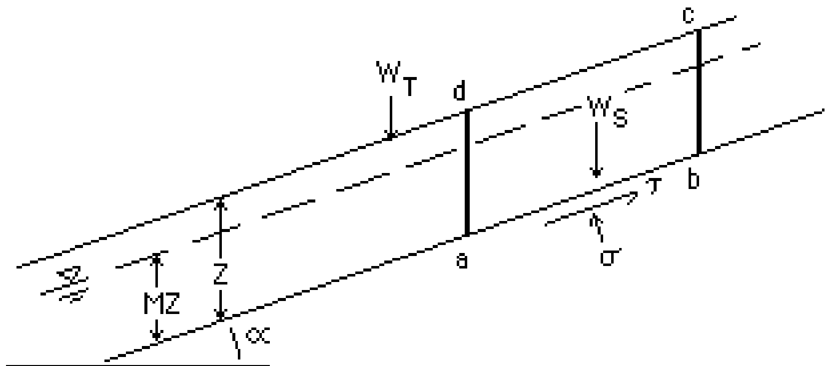


Figure 2. Infinite slope analysis for planar failures

The forces acting on the soil mass "a b c d" in Figure 2 include the vegetative weight per unit area, WT, and the weight of soil WS which give rise to the tangential and normal shear stresses acting on line a-b. The height of the water table is MZ. The vertical height of water above the slide plane, designated M, is a fraction of the soil thickness, Z, above the plane.

The resistance to failure or shear strength S along line a-b is:

$$S = [C' + \Delta C] + [\sigma - \mu] \tan \phi'$$

where

C' = effective soil cohesion

ϕ' = effective angle of internal friction for the soil

σ = total normal stress

μ = pore water pressure

ΔC = cohesion caused by root systems.

Using this method, calculation of unknowns is as follows:

$$\mu = MZ\gamma_w \cos^2\alpha$$

$$\sigma = [(1 - M)\gamma_m + M\gamma_{sat}] Z \cos^2\alpha + W_T \cos^2\alpha$$

$$\tau = [(1 - M)\gamma_m + M\gamma_{sat}] Z \sin\alpha \cos\alpha + W_T \sin\alpha$$

where

γ_m = soil unit weight at field moisture content

γ_{sat} = saturated soil unit weight

γ_w = water unit weight

α = slope angle

and the factor of safety:

$$FS = S/T$$

Theoretically, the factor of safety represents a ratio of forces causing a slope to remain stable (shear strength) to forces causing it to fail (shear stress). A factor of safety greater than 1.0 implies a stable slope, while a value less than 1.0 suggests the potential for a slope failure. Figure 3 illustrates the relationship between frictional resistance and the downslope component governing the disposition of a 45 kg (100 lb) block on uniform dry sand. For slope gradients greater than 70 percent, the block will slide because the driving force (E) is greater than the frictional resistance (F) to sliding. Frictional resistance for a normal soil at the plane c d is a function of soil, geology, and moisture content of the soil, and root strength.

Soil cohesiveness tends to prevent movement and generally increases with increased weathering producing finer textured soil particles. However, relative cohesion will decrease as soil moisture content increases causing the block a b c d to "float" above the failure plane c d. As a dry soil absorbs water, its shear strength decreases because water films tend to separate soil particles. This, in turn, reduces the cohesive strength produced by the frictional and electrical forces which cause clay particles to attract each other and form aggregates. An additional force component, buoyancy, tends to nullify the interlocking forces of soil particles which contribute to stability. The uplift force of groundwater is equal to 93.1 kg/m (62.4 lb/ft) of water in the soil. The effective normal force is equal to the weight of soil resting on the surface minus the uplift force of the groundwater. Figure 4 shows the effect of adding 15.2 cm (6 in) of water to 0.6 m (2 ft) of soil (again, dry sand). The effective normal force is reduced significantly by the addition of water resulting in failure when slopes equal or exceed 58 percent.

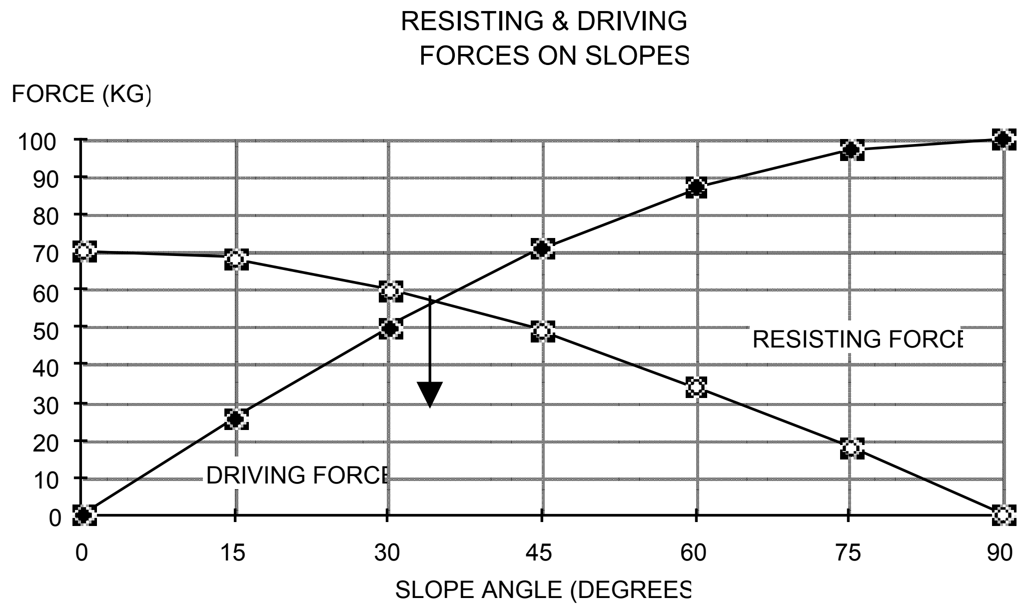


Figure 3. Relationship between frictional resistance (F) and driving force (E) promoting downslope movement. (Burroughs, et al., 1976).

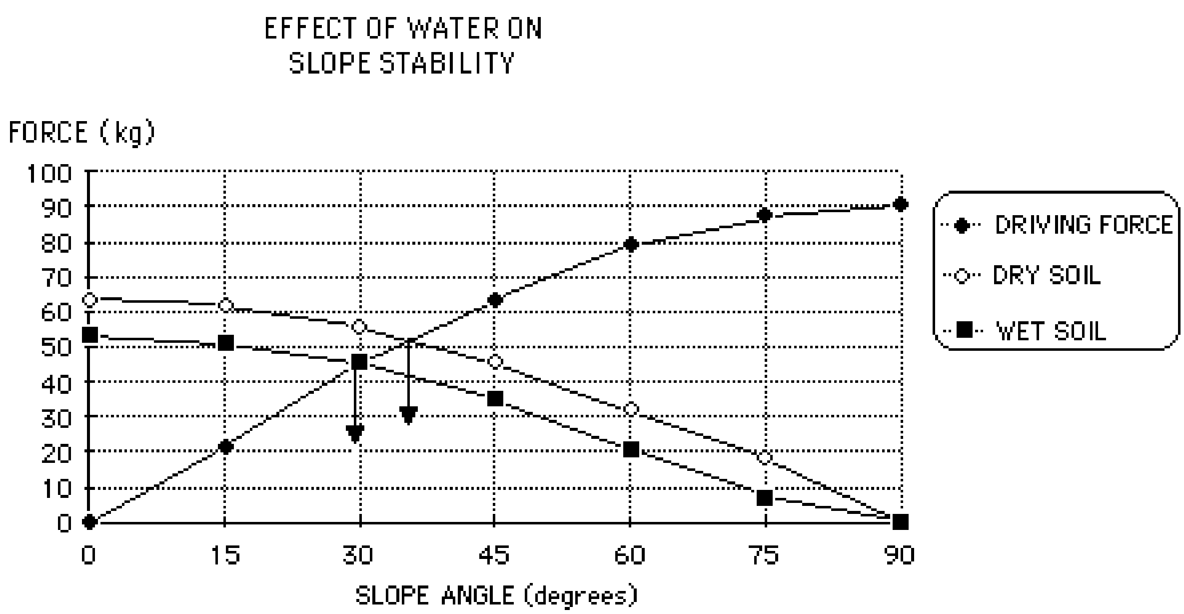
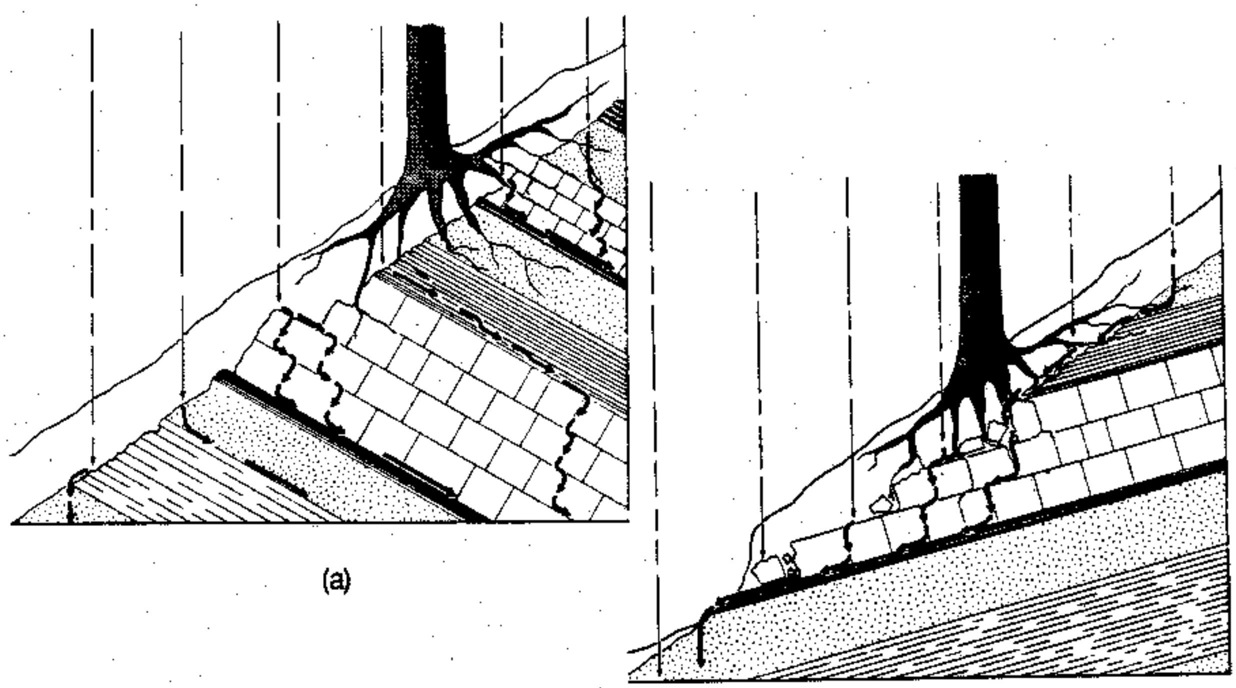


Figure 4. Sixty cm of soil with 15 cm of ground water will slide when the slope gradient exceeds 58 percent. (Burroughs, et al., 1976)

The problems and limitations in applying this or similar models are many. Estimation of these factors is extremely difficult given the high degree of anisotrophy and heterogeneity of soil properties. Detailed analysis of factors leading to failure of natural slopes, especially piezometric information, is lacking. As difficult as the prediction of the factor of safety is, predicting the course or type of deformation a failure will take is far more difficult. The contribution of plant root systems in reinforcing the soil matrix is often significant but difficult to quantify.

The orientation of the underlying geologic strata plays an important part in overall stability. When bedding planes are oriented in the direction of the slope (Figure 5a), potential zones of weakness and failure surfaces are ready-made. Additionally, the beds will tend to concentrate subsurface water and return it to the surface. Any excavation on such slopes may also remove support and create excessive road maintenance problems by rock and soil sliding on to the road. Conversely, geologic strata which are more or less normal to the surface slope (Figure 5b) resist sliding since weakness in the bedding planes do not contribute to the downslope component nor do they concentrate percolating rainwater near the surface.



(a)

(b)

Figure 5. (a) Subsurface rainwater flows in the direction of the slope when geologic strata dip toward the slope. (b) Subsurface rainwater percolates downward and out of the root zone when geologic strata dip in that direction (Rice, 1977).

Other factors influencing slope stability include seepage forces exerted by groundwater as it moves downslope through the soil and support provided by live tree roots in contributing to soil strength. Although not fully understood, the presence of root strength is most important where soils are shallow and where winter storms can cause groundwater levels to rise sharply. Roots tend to anchor shallow soils on steep slopes to fractures in the underlying rock. Reports from US Forest Service researchers in Alaska indicate that the number of landslides from cut-over areas increases within 3 to 5 years after logging--about the time when root decay becomes nearly complete. (Bishop and Stevens, 1964) Researchers in Japan and the U. S. have found that root systems of different species have differing decay rates. Rice (1977) postulates that harvest scheduling according to relative contribution of a particular specie's root system to slope stability might provide additional support to slopes at or near threshold strength values.

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CHAPTER 2

ROAD PLANNING AND RECONNAISSANCE

2.1 Route Planning

Planning with respect to road construction takes into account present and future uses of the transportation system to assure maximum service with a minimum of financial and environmental cost. The main objective of this initial phase of road development is to establish specific goals and prescriptions for road network development along with the more general location needs. These goals must result from a coordinated effort between the road engineer and the land manager, forester, geologist, soil scientist, hydrologist, biologist and others who would have knowledge or recommendations regarding alternatives or solutions to specific problems.

The pattern of the road network will govern the total area disturbed by road construction. The road pattern that will give the least density of roads per unit area while maintaining minimum hauling distance is the ideal to be sought. Keeping the density of roads to an economical minimum has initial cost advantages and future advantages in road maintenance costs and the acreage of land taken out of production.

Sediment control design criteria may be the same as, or parallel to, other design criteria, which will result in an efficient, economical road system. Examples of overlap or parallel criteria are:

1. Relating road location and design to total forest resource, including short and long term harvest patterns, reforestation, fire prevention, fish and wildlife propagation, rural homestead development, and rangeland management.

2. Relating road location and design to current and future timber harvesting methods.

3. Preparing road plans and specifications to the level of detail appropriate and necessary to convey to the road builder, whether timber purchaser or independent contractor, the scope of the project, and thus allow for proper preparation of construction plans and procedures, time schedules, and cost estimates.

4. Writing instructions and completing companion design decisions so as to minimize the opportunity for "changed conditions" during construction with consequent costs in money and time.

5. Analyzing specific road elements for "up-front" cost versus annual maintenance cost (for instance culvert and embankment repair versus bridge installation, ditch pavement or lining versus ditches in natural soil, paved or lined culverts versus unlined culverts, sediment trapping devices ("trash racks", catch basins, or sumps) versus culvert cleaning costs, retaining walls or endhauling sidecast versus placing and maintaining large embankments and fill slopes, roadway ballast or surfacing versus maintenance of dirt surfaces, and balanced earthwork quantities versus waste and borrow).

The route planning phase is the time to evaluate environmental and economic tradeoffs and should set the stage for the remainder of the road development process. Although inclusion of design criteria for sediment control may increase initial capital outlay, it does not necessarily increase total annual cost over the life of the road which might come from reductions in annual maintenance, reconstruction, and repair costs (see Section 2.2). If an objective analysis by qualified individuals indicates serious erosional problems, then reduction of erosional impacts should be a primary concern. In some areas, this may dictate the location of control points or may in fact eliminate certain areas from consideration for road construction as a result of unfavorable social or environmental costs associated with developing the area for economic purposes.

2.1.1 Design Criteria

Design criteria consist of a detailed list of considerations to be used in negotiating a set of road standards. These include resource management objectives, environmental constraints, safety, physical environmental factors (such as topography, climate, and soils), traffic requirements, and traffic service levels. Objectives should be established for each road and may be expressed in terms of the area and resources to be served, environmental concerns to be addressed, amount and types of traffic to be expected, life of the facility and functional classification. Additional objectives may also be defined concerning specific needs or problems identified in the planning stage.

1. Resource management objectives: Why is the road being built; what is the purpose of the road (i.e., timber harvesting, access to grazing lands, access to communities, etc.)?

2. Physical and environmental factors: What are the topographic, climatic, soil and vegetation characteristics of the area?

3. Environmental constraints: Are there environmental constraints; are there social-political constraints? Examples of the former include erosiveness of soils, difficult geologic conditions, high rainfall intensities. Examples of the latter include land ownership boundaries, state of the local economy, and public opinion about a given project.

4. Traffic requirements: Average daily traffic (ADT) should be estimated for different user groups. For example, a road can have mixed traffic--log or cattle trucks and community traffic. An estimate of traffic requirements in relation to use as well as changes over time should be evaluated.

5. Traffic service level: This defines the type of traffic that will make use of the road network and its characteristics. Table 3 lists descriptions of four different levels of traffic service for forest roads. Each level describes the traffic characteristics which are significant in the selection of design criteria and describe the operating conditions for the road. Each level also reflects a number of factors, such as speed, travel time, traffic interruptions, freedom to maneuver, safety, driver comfort, convenience, and operating cost. Traffic density is a factor only if heavy non-logging traffic is expected. These factors, in turn, affect: (1) number of lanes, (2) turnout spacing, (3) lane widths, (4) type of driving surface, (5) sight distances, (6) design speed, (7) clearance, (8) horizontal and vertical alignment, (9) curve widening, (10) turn-arounds.

6. Vehicle characteristics: The resource management objectives, together with traffic requirements and traffic service level criteria selected above, will define the types of vehicles that are to use the proposed road. Specific vehicle characteristics need to be defined since they will determine the "design standards" to be adopted when proceeding to the road design phase. The land manager has to distinguish between the "design vehicle" and the "critical vehicle". The design vehicle is a vehicle that ordinarily uses the road, such as dual axle flatbed trucks in the case of ranching or farming operations, or dump trucks in the case of a mining operation. The critical vehicle represents a vehicle which is necessary for the contemplated operation (for instance, a livestock truck in the case of transporting range livestock) but uses the road infrequently. Here, the design should allow for the critical vehicle to pass the road with assist vehicles, if necessary, but without major delays or road reconstruction.

7. Safety: Traffic safety is an important requirement especially where multiple user types will be utilizing the same road. Safety requirements such as stopping distance, sight distance, and allowable design speed can determine the selected road standards in combination with the other design criteria.

8. Road uses: The users of the contemplated road should be defined by categories. For example, timber harvest activities will include all users related to the planned timber harvest, such as silviculturists, foresters, engineers, surveyors, blasting crews, and construction and maintenance crews, as well as the logging crews. Administrative users may include watershed management specialists, wildlife or fisheries biologists, or ecologists, as well as foresters. Agricultural users would include stock herders and

rangeland management specialists and will have a different set of objectives than timber objectives. An estimate of road use for each category is then made (e.g., numbers of vehicles per day). For each category, the resource management objective over several planning horizons should be indicated. For instance, a road is to be built first for (1) the harvest of timber from a tract of land, then (2) access for the local population for firewood cutting or grazing, and finally (3) access for administration of watershed rehabilitation activities. The planner should determine if the road user characteristics would change over the life of the road.

9. Economics: The various road alternatives would undergo rigorous economic evaluation.

As part of this process a "roads objectives documentation" plan should be carried out. This process consists of putting the road management objectives and design criteria in an organized form. An example of such a form is given in Table 4.

	A	B	C	D
FLOW	Free flowing with adequate passing facilities.	Congested during heavy traffic such as during peak logging or recreation activities.	Interrupted by limited passing facilities, or slowed by the road condition.	Flow is slow or may be blocked by an activity. Two way traffic is difficult and may require backing to pass.
VOLUMES	Uncontrolled; will accommodate the expected traffic volumes.	Occasionally controlled during heavy use periods.	Erratic; frequently controlled as the capacity is reached.	Intermittent and usually controlled. Volume is limited to that associated with the single purpose.
VEHICLE TYPES	Mixed; includes the critical vehicle and all vehicles normally found on public roads.	Mixed; includes the critical vehicle and all vehicles normally found on public roads.	Controlled mix; accommodates all vehicle types including the critical vehicle. Some use may be controlled to minimize conflicts between vehicle types.	Single use; not designed for mixed traffic. Some vehicles may not be able to negotiate. Concurrent use between commercial and other traffic is restricted.
CRITICAL VEHICLE	Clearances are adequate to allow free travel. Overload permits are required.	Traffic controls needed where clearances are marginal. Overload permits are required.	Special provisions may be needed. Some vehicles will have difficulty negotiating some segments.	Some vehicles may not be able to negotiate. Loads may have to be off-loaded and walked in.
SAFETY	Safety features are a part of the design.	High priority in design. Some protection is accomplished by traffic management.	Most protection is provided by traffic management	The need for protection is minimized by low speeds and strict traffic controls.
TRAFFIC MANAGEMENT	Normally limited to regulatory, warning, and guide signs and permits.	Employed to reduce traffic volume and conflicts.	Traffic controls are frequently needed during periods of high use by the dominant resource activity.	Used to discourage or prohibit traffic other than that associated with the single purpose.
USER COSTS	Minimize; transportation efficiency is important.	Generally higher than "A" because of slower speeds and increased delays.	Not important; efficiency of travel may be traded for lower construction costs.	Not considered.
ALIGNMENT	Design speeds is the predominant factor within feasible topographic limitations.	Influenced more strongly by topography than by speed and efficiency.	Generally dictated by topographic features and environmental factors. Design speeds are generally low.	Dictated by topography, environmental factors, and the design and critical vehicle limitations. Speed is not important.
ROAD SURFACE	Stable and smooth with little or no dust, considering the normal season of use.	Stable for the predominant traffic for the normal use season. Periodic dust control for heavy use or environmental reasons. Smoothness is commensurate with the design speed.	May not be stable under all traffic or weather conditions during the normal use season. Surface rutting, roughness, and dust may be present, but controlled for environmental or investment protection.	Rough and irregular. Travel with low clearance vehicles is difficult. Stable during dry conditions. Rutting and dusting controlled only for soil and water protection.

table 7 Traffic service levels definitions used to identify design parameters (from U.S. Forest Service, Transportation Eng. Handbook).

2.1.2 Design Elements

A road design standard consists of such elements as the definitive lengths, widths, and depths of individual segments (e.g., 4.3 meter travelled way, 0.6 meter shoulders, 3/4:1 cutslopes, 1 meter curve widening, 15 cm of crushed aggregate surfacing). Figure 6 illustrates the road structural terms that will be used throughout the rest of this handbook. Selection of the appropriate road design standard is critical to the overall efficiency of the road network to be installed, and certain elements will have a more rigid standard than others depending on the location of the road or road segment. The entire range of values for each standard must be evaluated and selected according to their appropriateness for a given segment. Then, the various design elements must undergo testing to ensure that the final design meets the previously agreed upon management objectives. For instance, on steeper grades vertical alignment has a greater effect on travel speed than horizontal alignment. Therefore, surfacing and horizontal alignment should not be improved to increase speed where the road gradient is the controlling element.

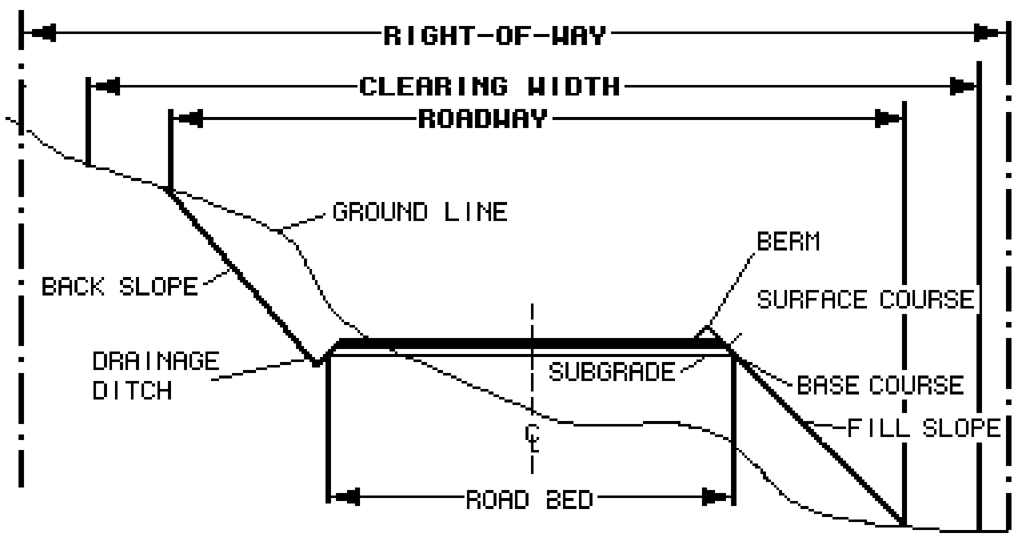

Figure 6. Road structural terms.

table 8 Example of a roads objective documentation form (from U.S. Forest Service, Transportation Eng. Handbook).

ROAD MANAGEMENT — DOCUMENTATION

ROAD NUMBER 8000, 8010, 8020 DATE 12/3/85 ROAD NAME ________

TOWNSHIP 15N RANGE 5E SECTION 5

DESIGN CRITERIA						ELEMENT SELECTED A. DESIGN B. TRAFFIC MGT C. MAINTENANCE
USER	TIME	1. REC MGT OBJ	2. ENVIR. CONST.	OTHER CRITERIA *	ROAD CLASS	
Timber	1988	2669.16 MBF Timber Sale	Wildlife Mgt	3. Inadequate for mixed traffic, narrow road 4. ridge edge - gentle side slope 5. 8-12 ADT (log truck) 6. Low-boy critical vehicle 7. Lowest cost to purchaser for Class C road and DNR specs.	C	A. 8000 Rd - Single lane 12' T.O. Road Class C. Curve widening for road class C. Surfaced with pit rock. Design speed 15 MPH. Culverts every 400' and where needed. 8010 Rd - Single lane 10' T.O Road Class C. Curve widening for class C. Surfaced with pit rock. Design speed 15 MPH. Culverts every 400' and where needed. B. Discourage Pass, Cars, RV, & Trail - restrict use during haul C. Close 8000-8010 system, after silvicultural procedures, for 10 yrs. Maintain ditch clearance.
Rec	1988	Firewood				
Timber	1998	1430.2 MBF Timber Sale	Same As Above (S.A.A.)	3-6. S.A.A. 7. Reconstruction costs for 8000-8010 road system.	C	A. S.A.A. B. S.A.A. C. Close 8010 Rd - no further timber sales are planned to use this road except for silvicultural mgt. Maintain ditch clearance. Maintain 8000 Rd.
Rec	1998	Firewood				

*OTHER CRITERIA: 3. SAFETY 4. PHYSICAL 5. TRAFFIC 6. VEHICLE 7. ECONOMY

2.1.2.1 Number of Lanes and Lane Width

The majority of forest development road systems in the world are single-lane roads with turnouts. It is anticipated that most roads to be constructed or reconstructed will also be single-lane with turnouts because of the continuing need for low volume, low speed roads and their desirability from economic and environmental impact standpoints. In choosing whether to build a single- or double-lane road, use the best available data on expected traffic volumes, accident records, vehicle sizes, and season and time-of-day of use. Historically, the United States Forest Service has used traffic volumes of approximately 100 vehicles per day to trigger an evaluation for increasing road width from one to two lanes. Considering a day to consist of 10 daylight hours, traffic volumes greater than 250 vehicles per day ordinarily require a double-lane road for safe and efficient operation. Intermediate traffic volumes (between 100 and 250 vehicles per day) generally require decisions based on additional criteria to those listed above: (1) social/political concerns, (2) relationships to public road systems, (3) season of use, (4) availability of funding, and (5) traffic management.

Many of the elements used in such an evaluation, although subjective, can be estimated using traffic information or data generated from existing roads in the area. For instance, if heavy public use of the road is anticipated, a traffic count on a comparably situated existing road will serve as a guide to the number of vehicles per hour of non-logging traffic. Some elements can be evaluated in terms of relative probabilities and consequences and can be identified as having a low, moderate, or high probability of occurrence and having minor, moderate, or severe consequences. The more criteria showing higher probabilities and more severe consequences, the stronger the need for a double-lane road.

2.1.2.2 Road width

The primary consideration for determining the basic width of the roadbed is the types of vehicles expected to be utilizing the road. Secondary considerations are the general condition of the traveled way, design speed, and the presence or absence of shoulders and ditches. Tables 5 and 6 list recommended widths for single- and double-lane roads, respectively.

table 9 Traveled way widths for single-lane roads.

Type and Size of Vehicle	Design Speed (Km/Hr)		
	30	40	50
	Minimum Traveled Way Width (m)		
Recreational, administrative and service vehicle, 2.0 to 2.4 m wide	3.0	3.0	3.6
Commercial hauling and commercial passenger vehicles, including buses 2.4 m wide or greater			
1. Road with ditch, or without ditch where cross slope is 25% or less	3.6	3.6	4.2
2. Roads without ditch where ground cross slope is greater than 25%. The steepness of roadway backslope should be considered to provide adequate clearance.	3.6	3.6	4.2

The presence of a ditch permits a narrower traveled way width since the ditch provides the necessary clearance on one side. Except for additional widths required for curve widening, limit traveled way widths in excess of 4.4 m (14 ft) to roads needed to accommodate off-highway haul and other unusual design vehicles. Double-lane roads designed for off-highway haul (all surface types) should conform to the following standards:

table 10 Lane widths for double-lane roads

Size and Type of Vehicle	Type of Road	Type of Surface	Design Speed (Km/Hr) 15	30	45	60	80
			Minimum Lane Width (m)				
Recreational, adm. and service:							
1. up to 2.0 m wide	Recreation or	All surface	2.7	2.7	3.0	3.3	3.0
2. 2.0 to 2.4 m wide	administrative	types	3.0	3.0	3.3	3.3	3.3
Commercial hauling and comm. passenger vehicles incl. buses	Roads open to truck traffic or mixed	Gravel or native	-	3.3	3.6	3.6	-
2.4 m wide or greater	traffic	Bituminous	-	3.3	3.3	3.3	3.6

Gravel or native surface roads should not have design speeds greater than 60 km/hr
Additional width is required for lower quality surfaces, because of the off-tracking corrections needed compared to a higher quality surface.

Vehicles wider than the design vehicle (a "critical vehicle") may make occasional use of the road. Check traveled way and shoulder widths to ensure that these vehicles can safely traverse the road. Critical vehicles should never attempt to traverse the road at or even approaching the speeds of the design vehicle.

Shoulders may be necessary to provide parking areas, space for installations such as drainage structures, guardrails, signs, and roadside utilities, increase in total roadway width to match the clear width of an opening for a structure such as a bridge or tunnel, a recovery zone for vehicles straying from the traveled way, additional width to accommodate a "critical vehicle", lateral support for outside edge of asphalt or concrete pavements (0.3 m is sufficient for this purpose). The space required for these features will depend on the design criteria of the road and/or the design of specific structures to be incorporated as part of the roadway.

Minimum Width of Traveled Way
for Design Speed

Bunk Width	30 km/hr(20 mph)	50 km/hr (30 mph)	60 km/hr (40 mph)
3 .0m (10 ft)	6.7 m (22 ft)	7.3 m (24 ft)	7.9 m (26 ft)
3.7 m (12 ft)	7.9 m (26 ft)	8.5 m (28 ft)	8.5 m (28 ft)

2.1.2.3. Turnouts

Turnout spacing, location, and dimensions provide user convenience and safety and allow vehicles to maintain a reasonable speed. Spacing can be computed using the following formula and the curves from Figure 7 and Table 7 :

$$T = 1.609^*(DS)/36$$

Where: T = Increase in travel time for the interrupted vehicle (percent)
D = Delay time per kilometer for the interrupted vehicle (seconds)
S = Design speed (kilometers per hour).

Solve the equation for T and then use the graph in Figure 7 to determine the turnout spacing required to accommodate the number of vehicles passing over the road per hour (VPH).

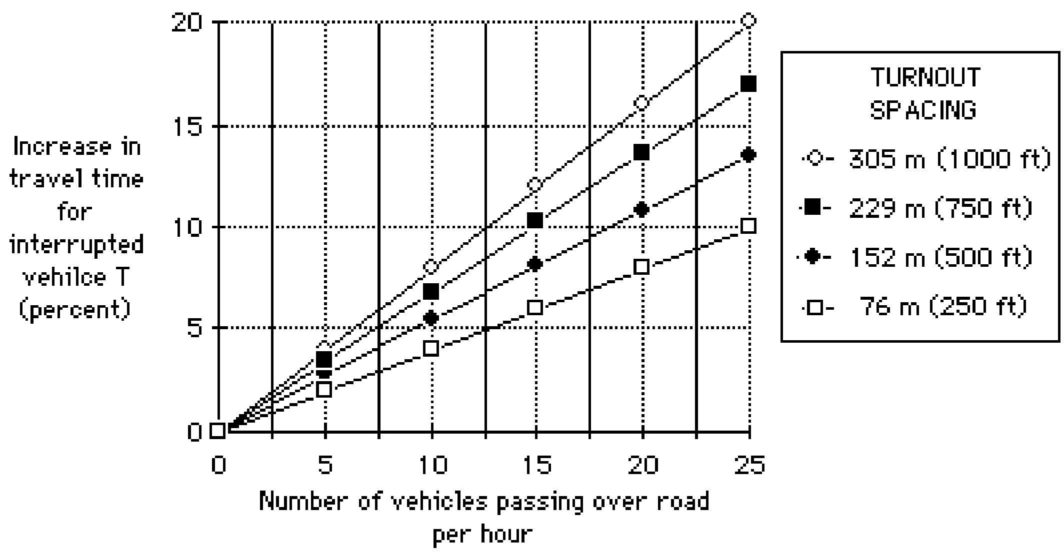

Figure 7. Turnout spacing in relation to traffic volume and travel delay time.

Figure 8 illustrates a typical turnout in detail. Turnouts should be located on the outside of cuts, the low side of fills, or at the runout point between through cuts and fills, and preferably on the side of the unloaded vehicle. Table 8 gives recommended turnout widths and lengths for various traffic service levels. The maximum transition length should be limited to 22.5 m for all service levels.

table 11 Recommended turnout spacing--all traffic service levels

Traffic Service	Turnout Spacing	Operational Constraints
A	Make turnouts intervisible unless excessive costs or environmental constraints preclude construction Closer spacing may contribute to efficiency and convenience. Maximum spacing is 300 m.	Traffic: Mixed Capacity: Up to 25 vehicles per hour Design Speed: Up to 60 km/hr Delays: 12 sec./km or less
B	Intervisible turnouts are highly desirable but may be precluded by excessive costs or environmental constraints. Maximum spacings 300 m.	Traffic: Mixed Capacity: Up to 25 vehicles per hour Design Speed: Up to 40 km/hr Delays: 20 km/hr or less Use signs to warn non-commercial users of traffic to be expected. Road segments without intervisible turn-outs should be signaled.
C	Maximum spacing is 300 m. When the environmental impact is low and the investment is economically justifiable, additional turnouts may be constructed.	Traffic: Small amount of mixed Capacity: Up to 20 vehicles per hour Design Speed: Up to 30 km/hr Delays: Up to 40 sec./km Roads should be managed to minimize conflicts between commercial and non-commercial users.
D	Generally, only naturally occurring turnouts, such as on ridges or other available areas on flat terrain, are used.	Traffic: Not intended for mixed Capacity: Generally 10 VPH or less Design Speed: 25 km/hr or less Delays: At least 45 sec./km expected. Road should be managed to restrict concurrent use by commercial and non-commercial users.

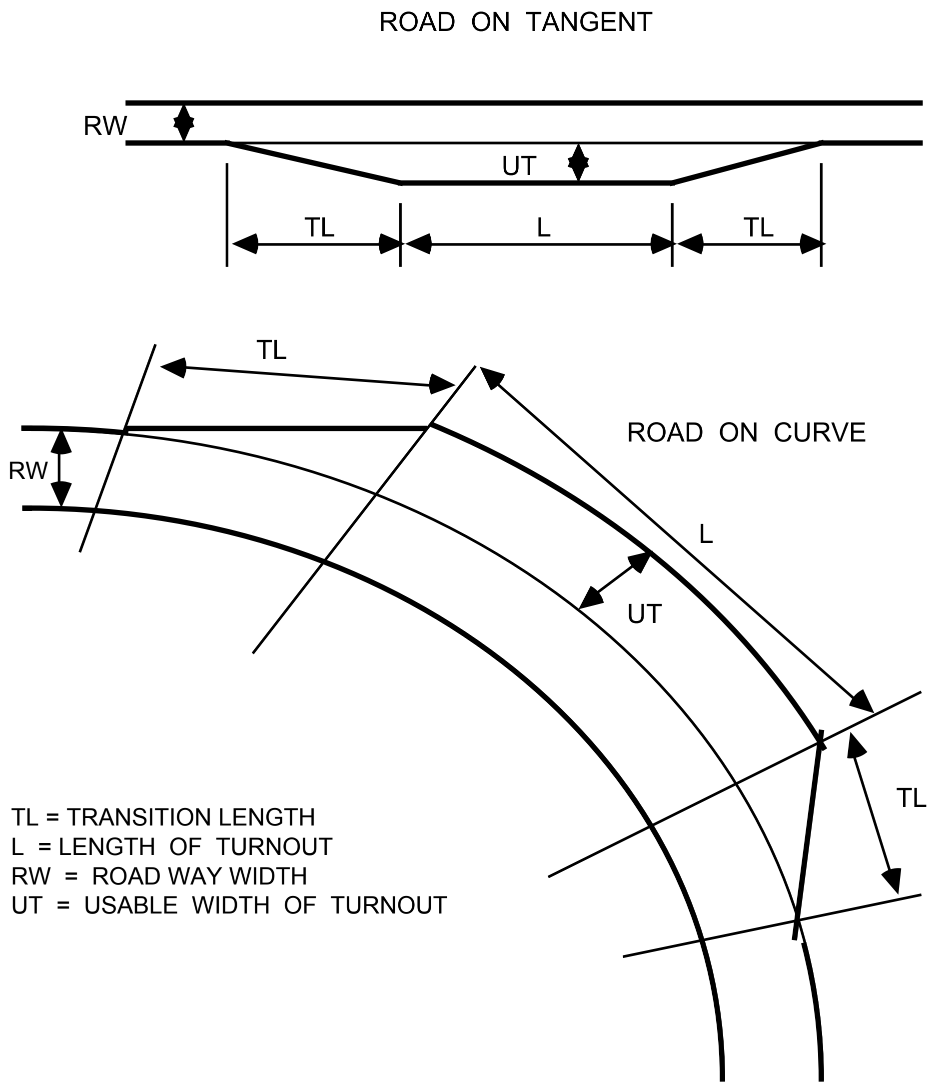

Figure 8. Typical turnout dimensions

table 12 Turnout widths and lengths

Traffic Service Levels	Turnout Width	Turnout Length & Transition Length
A	3.0 m	Design vehicle length or 22.5 m minimum, whichever is largest. Minimum 15 m transition at each end.
B	3.0 m	Design vehicle length. Minimum 15 m transition at each end.
D	Make the minimum total width of the traveled way and turnout the width of two design vehicles plus 1.2 m	Empty truck length (trailer loaded on truck) Minimum 7.5 m transitions at each end.

2.1.2.4. Turn-arounds

Turn-around design should consider both critical and design vehicles and should be provided at or near the end of single-lane roads, and at management closure points, such as gates or barricades. If intermediate turn-arounds are necessary, signing should be considered if they create a hazard to other users. The turn-around should be designed to allow the design vehicle to turn with reasonably safe maneuvering.

2.1.2.5. Curve Widening

Widening may be required on some curves to allow for off-tracking of tractor-trailer vehicles and for some light vehicle-trailer combinations. Widening of the traveled way on curves to accommodate the design vehicle is considered a part of the traveled way. Generally, the need for curve widening increases as curve radius decreases with shorter curves requiring less curve widening than longer curves. Criteria for establishing the need for curve widening given traffic service levels are given in Table 9.

table 13 Curve widening criteria

Traffic Service Level	Curve Widening
A	Design curve widening to accommodate the design vehicle (normally low-boy) at the design speed for each curve. Curve widening for critical vehicles to be provided by the use of other road elements, if planned, such as turnouts and shoulders. Provide widening if needed width is not available. Critical vehicle should be accommodated in its normal traveling configuration. Curve widening to be provided in each lane of double-lane roads.
B	Same as A.
C	Same as A, except the critical vehicle configuration may need alteration.

D Curve widening to be provided only for the design vehicle. Loads carried by the critical vehicle should be off-loaded and walked to the project or transferred to vehicles capable of traversing the road. Temporary widening to permit the passage of larger vehicles may be accomplished by methods such as temporarily filling of the ditch at narrow sections.

2.1.2.6 Clearance

The desired minimum horizontal clearance is 1.2 m (4 ft) the minimum vertical clearance is 4.3 m (14 ft). At higher speeds consideration should be given to increasing the clearances.

2.1.2.7. Speed and Sight Distance

Design speed is the maximum safe speed that the design vehicle can maintain over a specified segment of road when conditions are so favorable that the design features of the road govern rather than the vehicle operational limitations. The selected design speed establishes the minimum sight distance for stopping, passing, minimum radius of curvature, gradient, and type of running surface. Alternative combinations of horizontal and vertical alignment should be evaluated to obtain the greatest sight distance within the economic and environmental constraints. Suggested horizontal curve radius for a packed gravel or dirt road with no sight obstruction is 33 and 62 m (108 and 203 ft) for design speeds of 24 and 32 km/hr (15 and 20 mph), respectively. For curves with a sight obstruction 3 m (10 ft) from the travel way, horizontal curve radii are 91 and 182 m (300 and 600 ft), respectively. Suggested vertical curve length is 61 m (200 ft). Recommended stopping distances for single-lane roads with a maximum pitch of 2 percent (horizontal and vertical control) and traffic service level C or D are:

km/hr (MPH)	Stopping Distance, meters (feet)
16 (10)	21.3 (70)
24 (15)	36.5 (120)
32 (20)	54.9 (180)
48 (30)	94.5 (310)

For a more comprehensive discussion on stopping sight distance and passing sight distance, the reader is referred to the following sources: Route Location and Design, by Thomas F. Hickerson; USDA, Forest Service Handbook #7709.11, "Transportation Engineering Handbook"; Bureau of Land Management, Oregon State Office, "Forest Engineering Handbook"; or "Geometric Design Standards for Low Volume Roads", Transportation Research Board.

2.1.2.8. Horizontal and Vertical Alignment

For low volume roads with design speeds of 24 kph (15 mph) or less, a horizontal alignment that approximates the geometric requirements of circular curves and tangents may be used. Alignment should be checked so that other design elements, such as curve widening and stopping sight distance can be considered. A minimum centerline radius of curvature for roads should be 15 meters (50 ft) except for some recreation and administrative roads. Superelevation should not be used for design speeds less than 32 kph (20 mph). If snow and ice are factors, the superelevation rate should not exceed 6 percent and should be further reduced on grades to accommodate slow truck traffic. Transition segments into and out of superelevated sections should be provided to avoid abrupt changes in the roadway template.

Vertical alignment, or grade, is of critical concern because of its potential for environmental damage and becomes increasingly important for grades exceeding 10 percent. Erosion potential increases as a function of the square of the slope and the cube of water velocity. The most desirable combination of grade and other design elements should be determined early in the road location phase with additional caution exercised when grades exceed 8 percent. Vertical alignment normally governs the speed of light vehicles for grades exceeding 15 percent favorable and 11 percent adverse and of loaded trucks for grades exceeding 8 percent favorable and 3 percent adverse. The ability of a vehicle to traverse a particular grade is dependent on vehicle weight and horsepower and on the traction coefficient of the driving surface.

Travel time and cost are affected by horizontal alignment, such as curve radius and road width. Figure 9 shows the relationship between average truck speed and curve radius for several road widths. For example, there is a 15 percent difference in average truck speed on a 30.5 m (100 ft) radius curve for a 3.7 m wide road when compared to a 4.3 m wide road.

Horizontal alignment has been classified on the basis of curve radius and number of curves. The U. S. Forest Service, for example, uses the following classification system:

[Average radius (m)] / [# of curves / km]

Poor	=	< 4	Good	=	10 - 20
Fair	=	4 - 10	Excellent	=	> 20

The effect of grade on truck speed (loaded and unloaded) is shown in Figure 10. The speed of a loaded truck is most sensitive to grade changes from 0 to 7 percent in the direction of haul. For grades steeper than 7 percent other considerations are more important than impact on speed.

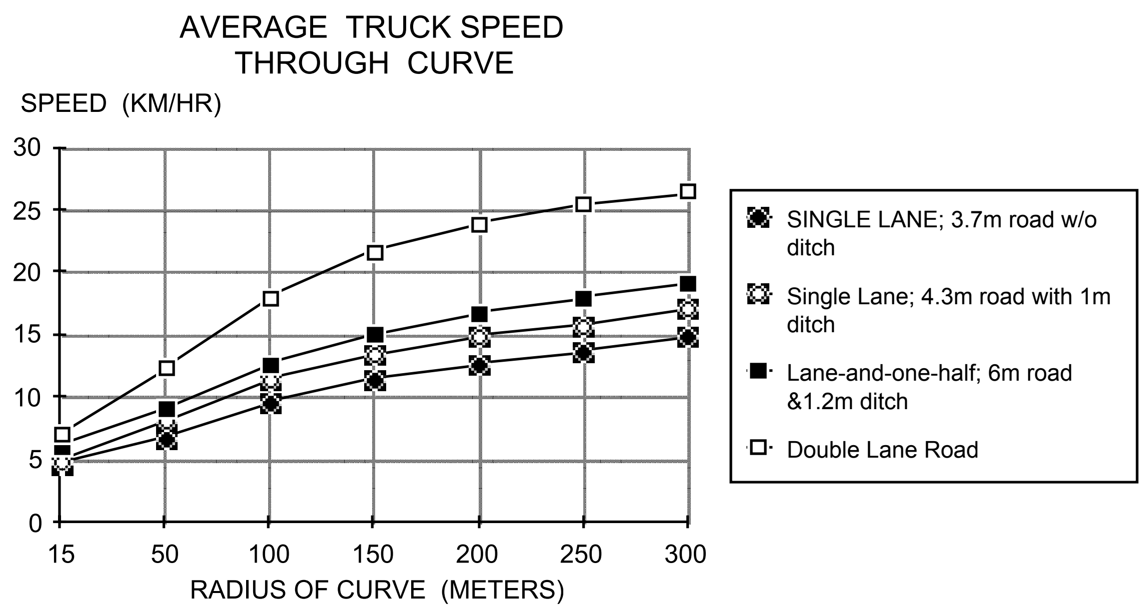

Figure 9. Relationship between curve radius and truck speed when speed is not controlled by grade.

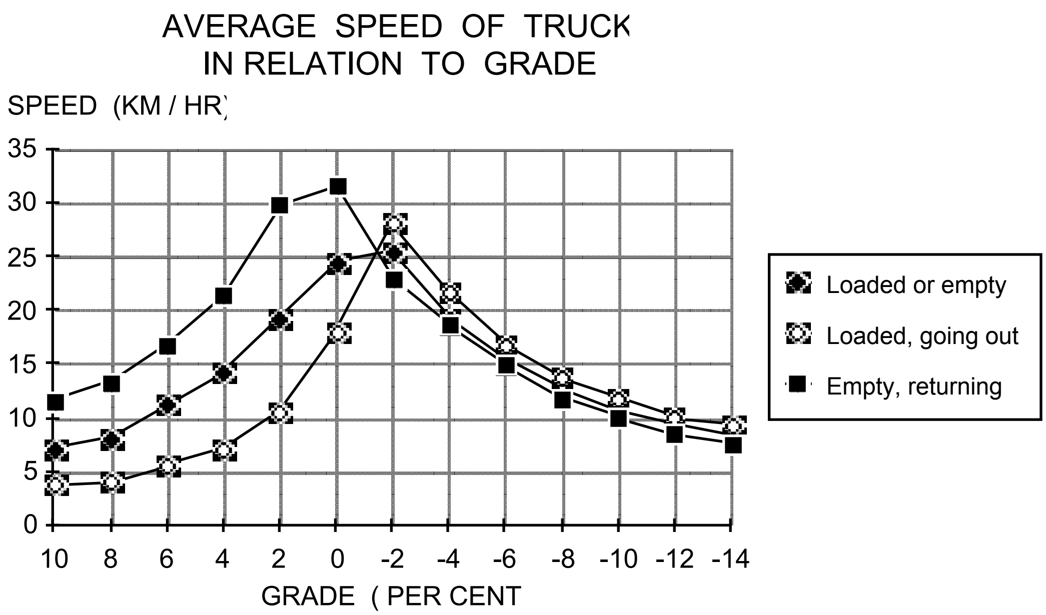

Figure 10. Relationship between grade and truck speed on gravel roads.

2.1.2.9. Travel Time

It is important to emphasize that travel time is influenced by grade, nature of road surface, alignment, roadway width, sight distance, climate, rated vehicle performance, and psychological factors (such as fatigue, degree of caution exercised by driver, etc.). Table 10 shows travel time for loaded and empty trucks over paved, graveled, and dirt surfaces as influenced by vertical and horizontal alignment. The information from Table 10 is helpful in the planning stage to assess the effects of vertical and/or horizontal alignment, road surface and width on travel time and costs. The planned road should be divided up into segments of similar vertical and/or horizontal alignment sections. Average times can be calculated for each segment and/or road class and summed.

table 14 Relationship between round trip travel time per kilometer and surface type as influenced by vertical and horizontal alignment; **adverse grade** in direction of haul (U.S. Forest service, 1965).

Class of Road[1]	Percent Grade in Direction of Load (Adverse)									
	+10	+9	+8	+7	+6	+5	+4		+2	0
	--	--	--	--	min/km --		--	--	--	--
1. Lane and one-half with turnouts (car lane and truck lane with 4-ft. ditch)[2]..										
A. Alignment excellent:										
1. Paved	7.93	7.28	6.59	5.93	5.28	4.59	3.95	-	2.95	2.42
2. Gravel	8.21	7.56	6.87	6.21	5.53	4.84	4.23	-	3.20	2.42
B. Alignment good:										
1. Paved	7.93	7.28	6.59	5.93	5.32	4.78	4.25	-	3.25	3.01
2. Gravel	8.21	7.56	6.87	6.21	5.57	5.03	4.53	-	3.50	3.01
C. Alignment fair:										
1. Paved	7.93	7.28	6.62	6.12	5.62	5.09	4.56	-	3.61	3.61
2. Gravel	8.21	7.56	6.90	6.40	5.87	5.34	4.84	-	3.81	3.61
2. Single lane with turnouts (truck lane with 3-ft. ditch)[2]:										
A. Alignment excellent:										
1. Paved	7.93	7.28	6.59	5.93	5.28	4.59	4.02	-	3.02	2.58
2. Gravel	8.21	7.56	6.87	6.21	5.53	4.84	4.30	-	3.27	2.58
B. Alignment good:										
1. Paved	7.93	7.28	6.59	5.93	5.38	4.85	4.32	-	3.32	3.20
2. Gravel	8.21	7.56	6.87	6.21	5.63	5.10	4.60	-	3.57	3.20
3. Dirt	8.49	7.81	7.12	6.43	5.85	5.35	4.82	-	4.18	3.20
C. Alignment fair:										
1. Paved	7.93	7.28	6.75	6.25	5.75	5.22	4.68	-	3.89	3.89
2. Gravel	8.21	7.56	7.03	6.53	6.00	5.47	4.97	-	3.94	3.89
3. Dirt	8.49	7.81	7.28	6.75	6.21	5.72	5.18	-	4.15	3.89
D. Alignment poor:										
1. Gravel	8.45	7.95	7.42	6.92	6.39	5.86	5.36	-	4.68	4.68
2. Dirt	8.73	8.20	7.67	7.14	6.61	6.11	5.58	-	4.68	4.68
3. Single lane with turnouts (truck lane without ditch)[2]:										
B. Alignment good:										
3. Dirt	8.49	7.81	7.12	6.45	5.92	5.42	4.88	-	3.85	3.28
C. Alignment fair:										
3. Dirt	8.49	7.93	7.40	6.86	6.33	5.83	5.30	-	4.27	4.08
D. Alignment poor:										
3. Dirt	9.03	8.49	7.96	7.43	6.90	6.40	5.87	-	5.28	5.28

[1] Alignment classification basis:

Poor .... = $\dfrac{\text{Average radius (meter)}}{\text{No. of curves per km}}$ = less than 4

Fair .... = do. = 4 to 10
Good .... = do. = 10 to 20
Excellent... = do. = over 20

[2] On single-lane or land-and-one-half roads, increase the time for passing vehicles on turnouts by the percent shown in following tabulation. Consider all vehicles for single-lane roads and only trucks for land-and-one-half roads.

Turnout spacing (meter)	Increased time when number of vehicles passing over road per hour is			
	5	10	15	20
	Percent	Percent	Percent	Percent
75	2.0	4.0	6.0	8.0
150	2.6	5.4	8.0	10.7
225	3.4	6.8	10.2	13.6

table 15 Relationship between round trip travel time per kilometer and surface type as influenced by vertical and horizontal alignment; **favorable grade** in direction of haul (U.S. Forest service, 1965).

Class of Road[1]	Percent Grade in Direction of Load (Favorable)									
	0	-2	-4	-6	-7	-8	-9	-11	-12	-14
	--	--	--	--	min/km	--	--	--	--	--
1. Lane and one-half with turnouts (car lane and truck lane with 4-ft. ditch)[2]..										
A. Alignment excellent:										
1. Paved	2.42	2.42	2.47	2.89	3.22	3.53	3.87	4.53	4.87	5.59
2. Gravel	2.42	2.42	2.53	2.97	3.31	3.65	3.97	4.62	4.97	5.68
B. Alignment good:										
1. Paved	3.01	3.01	3.01	3.05	3.22	3.53	3.87	4.53	4.87	5.59
2. Gravel	3.01	3.01	3.01	3.13	3.32	3.65	3.97	4.62	4.97	5.68
C. Alignment fair:										
1. Paved	3.61	3.61	3.61	3.61	3.61	3.67	3.87	4.53	4.87	5.59
2. Gravel	3.61	3.61	3.61	3.61	3.61	3.80	3.97	4.62	4.97	5.68
2. Single lane with turnouts (truck lane with 3-ft. ditch)[2]..............										
A. Alignment excellent:										
1. Paved	2.58	2.58	2.58	2.89	3.22	3.53	3.87	4.53	4.87	5.59
2. Gravel	2.58	2.58	2.62	2.97	3.31	3.65	3.97	4.62	4.97	5.68
B. Alignment good:										
1. Paved	3.20	3.20	3.20	3.20	3.34	3.53	3.87	4.53	4.87	5.59
2. Gravel	3.20	3.20	3.20	3.25	3.44	3.65	3.97	4.62	4.97	5.68
3. Dirt	3.20	3.20	3.20	3.31	3.50	3.72	4.06	4.72	5.06	5.75
C. Alignment fair:										
1. Paved	3.89	3.89	3.89	3.89	3.89	3.89	4.02	4.53	4.87	5.59
2. Gravel	3.89	3.89	3.89	3.89	3.89	3.95	4.11	4.62	4.97	5.68
3. Dirt	3.89	3.89	3.89	3.89	3.89	4.02	4.20	4.72	5.06	5.75
D. Alignment poor:										
2. Gravel	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.88	5.07	5.68
3. Dirt	4.68	4.68	4.68	4.68	4.68	4.68	4.68	4.98	5.17	5.75
3. Single lane with turnouts (truck lane without ditch)[2]..............										
B. Alignment good:										
3. Dirt	3.28	3.28	3.28	3.33	3.52	3.72	4.06	4.72	5.06	5.75
C. Alignment fair:										
3. Dirt	4.08	4.08	4.08	4.08	4.08	4.09	4.28	4.72	5.06	5.75
D. Alignment poor:										
3. Dirt	5.28	5.28	5.28	5.28	5.28	5.28	5.28	5.28	5.47	5.82

[1] Alignment classification basis:

Poor .... = $\frac{\text{Average radius (meter)}}{\text{No. of curves per km}}$ = less than 4

Fair .... = do. = 4 to 10
Good .... = do. = 10 to 20
Excellent .. = do. = over 20

[2] On single lane or lane-and-one-half roads, increase the time for passing vehicles on turnouts by the percent shown in following tabulation. Consider all vehicles for single-lane roads and only trucks for land-and-one-half roads.

Turnout spacing (meter)	Increased time when number of vehicles passing over road per hour is			
	5	10	15	20
	Percent	Percent	Percent	Percent
75	2.0	4.0	6.0	8.0
150	2.6	5.4	8.0	10.7
225	3.4	6.8	10.2	13.6

2.2 Economic Evaluation and Justification

2.2.1 Economic Analysis Methods

A long-range plan, including road planning, is the basis for an economically, as well as environmentally, sound road system. A well planned road system will result in the least amount of roads to economically serve an area or watershed. It will also result in the least amount of sediment delivery to streams as shown in Figure 1.

The first step in road access planning is to determine the optimum road spacing for a given commercial use. A break-even analysis can often be applied. Plotted graphically, the optimum spacing would lie at the minimum total cost, or the intersection of the cost lines. Additional information can be found in Pearce (1960), Dietz et al (1984), von Segebaden (1964), and others.

An economic evaluation of a particular road standard will require a rough estimate of road construction costs be determined from road design data and from locally available cost information for the various cost components. Likewise, annual maintenance cost per kilometer of road is best estimated based on local experience for comparable roads. Trucking cost data will consist of the average cost per round-trip kilometer of haul over the road and would take into consideration travel time (see Section 2.1), fixed costs (depreciation, interest, insurance, etc.), operating costs per minute driving time (fuel, lubrication, repairs), dependent costs per minute driving time plus delay time (driver's wage, social security tax, unemployment compensation, administration), and tire cost per mile by surface type.

The combined annual costs of road construction, maintenance, and trucking make up the annual cost:

$$A = R + I + M + T$$

where A is total annual cost per kilometer, R is annual cost per kilometer of road construction for the amortization period, I is average annual interest cost, M is annual maintenance cost per kilometer, and T is average trucking cost per kilometer for the annual commodity volume to be hauled over the road.

EXAMPLE:
Assume the following costs (in US dollars) have been estimated for three classes of road. (Annual volume of commodity, 10 million cubic meters.)

ROAD CLASS	I	II	III
Construction cost per kilometer	$40,000.00	$22,000.00	$15,000.00
Maintenance cost per kilometer	300.00	400.00	500.00
Trucking cost per 1,000 m^3 per kilometer	0.25	0.30	0.35
Trucking cost per annum per kilometer	2,500.00	3,000.00	3,500.00
<u>Annual cost per km over 25 years</u>			
R road construction cost	1,600.00	880.00	600.00
I interest costs	700.00	383.00	262.00
M maintenance cost	300.00	400.00	500.00
T trucking cost	2,500.00	3,000.00	3,500.00
A Total Annual Costs	$5,100.00	$4,663.00	$4,862.00

(If amortization period is 25 years, the annual rate is 4 percent of the construction cost. If the interest rate is 3.5 percent, the average annual interest rate is 1.75 percent.)

Note that in the above calculation the Class II road is the most economical by a margin of $199.00 over the Class III road. Over the period of amortization of 25 years, the margin for the Class II road will be $4,975.00 per kilometer.

Another method in choosing the most economical of two road standards is to calculate the annual amount or volume of commodity at which the costs of the two roads will be equal. If annual volume exceeds the calculated amount the higher road standard will be justified; likewise, if annual volume is less than the calculated amount, the lower standard is justified. The formula for calculating V is:

$$V = \frac{(R + I + M)_H - (R + I + M)_L}{T_L - T_H}$$

The subscripts H and L indicate high and low standard, respectively, and T is expressed as cost per 1000 m^3 per kilometer. All other values are expressed in units stated above.

EXAMPLE

Using the same costs as in the previous example for the Class II and Class III road, the annual volume is calculated as:

$$V = \frac{(880 + 383 + 400) - (600 + 262 + 500)}{(0.35 - 0.30)} = 6{,}020 \times 10^3 \ m^3$$

Hence, for volumes exceeding 6.02×10^6 m^3 the Class II road is the more economical choice; for volumes less than 6.02×10^6 m^3 the Class III road would be chosen. If the two roads differ in length, multiply the costs per kilometer by the number of kilometers of each road for use in this formula.

2.2.2 Analysis of Alternative Routes

The above formulas can also be used to evaluate two or more alternatives to a proposed route. One common situation is to choose between a longer route on a gentle favorable grade and a shorter route involving an adverse grade and a steeper favorable grade.

EXAMPLE.

1. <u>Longer route segment:</u> 3.67 km of 3% favorable grade. Trucking cost = $.562 per 1000 m^3; construction cost $55,050 at 6% amortization plus interest = $3,303; annual maintenance at $300/km = $1,101. Total annual cost = $4,404.

2. <u>Shorter route segment:</u> 2.0 km of 8% favorable grade, 1 km of 5% adverse grade. Trucking cost = $.81 per 1000 m^3; construction cost $41,000 at 6% amortization plus interest = $2,460; annual maintenance at $400/km (steeper grade, sharper curves) = $1,200. Total annual cost = $3,660.

$$V = (4{,}404 - 3{,}660)/(0.81 - 0.562) = 3 \times 10^6 \ m^3$$

(According to the formula, the longer route will be the more economical if the annual volume hauled exceeds 3 million cubic meters.)

In justifying the added capital investment to achieve greater road stability the risk of potential cost of a road failure must also be weighed in the balance. This type of risk analysis is commonly done when determining culvert size for a particular stream crossing. The probability of occurrence of a peak flow event which would exceed the design capacity of the proposed culvert installation must be determined and incorporated into the design procedure. The 1964-65 winter season floods occurring throughout the Pacific Northwest Region of the United States have been classified as 50- to 100-year return interval events. ("Return interval" is defined as the length of time that a storm event of specified magnitude would be expected to reoccur. A 50-year event, therefore, would be expected to occur, on the average, once every 50 years.) Damages to transportation structures (roads, bridges, trails) in Oregon was estimated at $12.5 million, or, 4 percent of the total investment of $355 million not including destruction of stream habitat, water quality, private property, and "down time" and other inconveniences associated with these losses.

As mentioned earlier in this handbook, constructing roads specifically to control erosion may not cost any more than constructing roads using conventional methods. The money invested to achieve satisfactory levels of stability while still meeting design criteria will generally be recouped over the life of the road in reduced maintenance costs, serviceability, longer life, and reduced impacts on stream habitat and water quality. The goal of fitting roads to the terrain and adopting appropriate road standards to achieve that goal will often result in reduced earthwork per station.

Incremental costs for roads built to high standards of construction (compacted fills, surface treatments, terraced fills, etc.) associated with the amount of reduction of sediment yield is difficult to generate since such wide variability exists in equipment and labor costs, environmental factors (such as soil erodibility), and operator skill. Gardner (1971) has developed some rudimentary data for comparing annual road costs for single and double lane roads with differing surface treatments depreciated over 20 years and using 6 percent capital recovery. The author suggests that user cost for environmental protection is represented as the difference in annual cost between two-lane paved and one-lane gravel roads in Table 11. More detailed comparisons of annual cost per km at different user levels is presented in Tables 12 and 13.

table 16 Comparison of single-lane versus double-lane costs at three different use levels.

Number of Vehicles per year	Total annual cost per kilometer 1 lane gravel	2 lane paved	Difference
	--------------------US Dollars --------------------		
10,000	3,440	4,200	-760
20,000	5,800	5,690	+112
40,000	10,530	8,680	+1,790

table 17 Comparison of annual road costs per kilometer -- 10,000 vehicles per year.

Cost distribution	2 lane paved	2 lane chip-seal	2 lane gravel	1 lane gravel	1 lane spot stabilization	1 lane primitive
	Road Standard					
	----------------Dollars per kilometer-------------------					
Initial Construction	$31,070	$24,860	$18,640	$12,430	$9,320	$6,210
	--------Dollars per kilometer per year (20-year period)-------					
Depreciation[1]	2,710	2,170	1,620	1,080	810	540
Maintenance	120	250	370	500	680	310
Vehicle use	1,370	1,430	1,680	1,860	2,730	5,280
Total annual	4,200	3,850	3,670	3,440[2]	4,230	6,130

[1] 20 years at 6% using capital recovery.

[2] Lowest annual cost.

table 18 Comparison of annual road costs per kilometer for 20,000 and 40,000 vehicles per year

Cost distribution	2 lane paved	2 lane chip-seal	2 lane gravel	1 lane gravel	1 lane spot stabilization	1 lane primitive
	Road Standard					
	----------------Dollars per kilometer-------------------					
Initial construction	31,070	24,860	18,640	12,430	9,320	6,210
	--------Dollars per kilometer per year (20-year period)------- 20,000 vehicles per year					
Depreciation[1]	2,710	2,170	1,620	1,080	810	540
Maintenance	250	500	750	1,000	1,370	620
Vehicle use	2,730	2,860	3,360	3,730	5,470	10,560
Total annual	5,690	5,530[2]	5,730	5,810	7,650	11,720
	--------Dollars per kilometer per year (20-year period)------- 40,000 vehicles per year					
Depreciation	2,710	2,170	1,620	1,080	810	540
Maintenance	500	1,000	1,490	1,990	2,730	1,240
Vehicle use	5,470	5,720	6,710	7,460	10,940	21,130
Total annual	8,680[2]	8,890	9,820	10,530	14,480	22,910

[1] 20 years at 6% using capital recovery.

[2] Lowest annual cost.

Gardner (1978) analyzed alternative design standards and costs in addition to observing the initial performance of the experimental road and its esthetic acceptability. Alternate design features included reducing road width to a level that would accommodate the tracks of the proposed yarding equipment (3.81 m (12.5 ft)), treating slash by chipping and scattering below the toe of the fill, using turnouts only when the terrain was favorable thus keeping road widths to a minimum, creating stepped backslopes (Figure 11) where bedrock competence was good and planting shrubs and grasses with and without straw mulches, and, finally, incorporating neoprene down- spouts below culverts to dissipate energy and protect the road prism. Sections I and II of the experimental road had the following characteristics:

	Average grade (percent)	Average curve radius (meters)	# curves / km (mi)
Section I	7.26	25.00	12.1 (19.4)
Section II	5.90	19.30	10.8 (17.4)

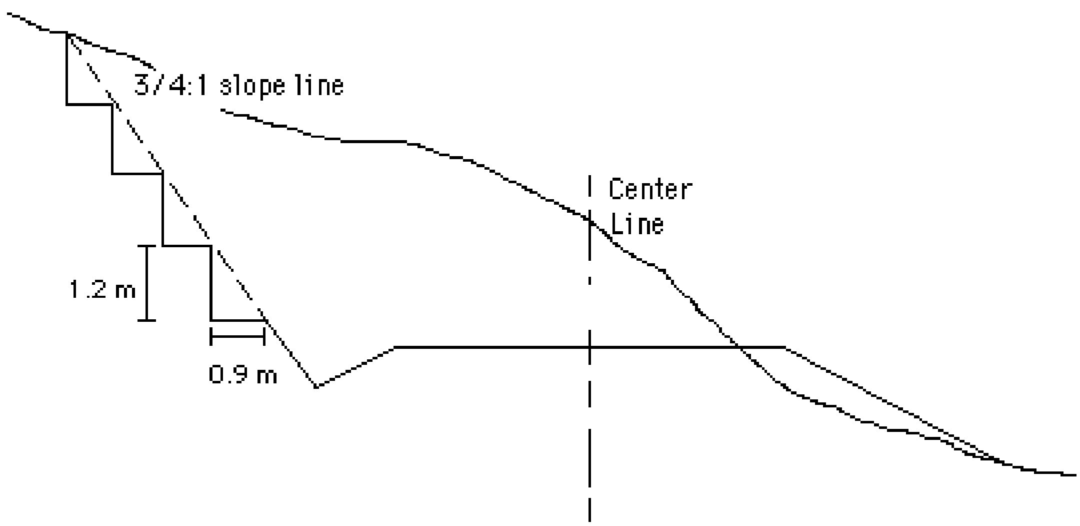

Figure 11 Stepped backslope (no scale).

Gardner found that using 1/10:1 backslopes and reducing clearing widths in the experimental road saved approximately $4,333 in construction cost and had no adverse effect on logging or hauling cost (Table 14). The effects on harvesting costs were not analyzed in this study.

table 19 Cost summary comparison (5 vehicles per hour--1/2 logging trucks, 1/2 other traffic); assumes 8-hour hauling day, 140 days/year use, 20 year road life, 23.8 m^3 (6.0 M bd. ft.) loads for logging trucks, cost of operating logging trucks including driver's wage--$0.25/min, cost of operating other vehicles--$0.04/minute, 5,535 m^3 (1 1/2 MM bd. ft.) timber harvested. (Gardner, 1978)

Road standard*	Annual amortized difference in cost	Annual difference hauling cost	Annual difference other traffic	Net difference
		-------------------Dollars----------------------		
Experimental	0			
III	+1,842.99	-3,187.65	-431.20	-1,775.86
IV	+11,790.22	-15,287.59	-2,371.60	-5,868.97

* Experimental road: single lane, 4.27 m (14 ft) width, 24.1 kph (15 mph) design speed, 0.91 m (3 ') ditch.
III road: single lane, 4.88 m (16 ft) width, 27.4 km/hr (17 mph) design speed, 0.91 m (3 ft) ditch.
IV road: double lane, 7.32 m (24 ft) width, 38.6 km/hr (24 mph) design speed, 1.22 m (4 ft) ditch.

Table 14 indicates that any environmental values gained by the construction of the experimental road would cause little economic sacrifice at vehicle use levels of 5 per hour. At higher use levels, however, the trade-offs become more significant and decisions regarding standards become more difficult.

2.3 Route Reconnaissance and Location

Keep in mind that a bad road in a good location is preferable to a good road in a bad location. A bad road can nearly always be fixed. However, no amount of quality survey or design work can correct any significant location error. For instance, a road constructed across a steep headwall area is more likely to intercept surface and subsurface water flow and has a far greater potential for failure than a road constructed along the ridgeline above the headwall. Since excess moisture is nearly always associated with landslides, it is always best to avoid drainage areas where water is expected to collect. Some important factors to remember when locating roads include:

1. Avoid high erosion hazard sites, particularly where mass failure is a possibility.

2. Utilize natural terrain features such as stable benches, ridgetops, and low gradient slopes to minimize the area of road disturbance.

3. f necessary, include short road segments with steeper gradients to avoid problem areas or to utilize natural terrain features.

4. Avoid midslope locations on long, steep, or unstable slopes.

5. Locate roads on well-drained soils and rock formations which dip into slopes rather than areas characterized by seeps, highly plastic clays, concave slopes hummocky topography, cracked soil and rock strata dipping parallel to the slope.

6. For logging road, utilize natural log landing areas (flatter, benched, well-drained land) to reduce soil disturbance associated with log landings and skid roads.

7. Avoid undercutting unstable, moist toe slopes when locating roads in or near a valley bottom.

8. Roll or vary road grades where possible to dissipate flow in road drainage ditches and culverts and to reduce surface erosion.

9. Select drainage crossings to minimize channel disturbance during construction and to minimize approach cuts and fills.

10. Locate roads far enough above streams to provide an adequate buffer, or provide structure or objects to intercept sediment moving downslope below the road.

11. If an unstable area such as a headwall must be crossed, consider end hauling excavated material rather than using sidecast methods. Avoid deep fills and compact all fills to accepted engineering standards. Design for close culvert and cross drain spacing to effectively remove water from ditches and provide for adequate energy dissipators below culvert outlets. Horizontal drains or interceptor drains may be necessary to drain excess groundwater.

2.3.1 Road Reconnaissance

Erosion and sedimentation rates are directly linked to total road surface area and excavation. The closer the road centerline follows the natural topographic contour, the smaller the erosional impact will be. On low-volume roads it is permissible and even advisable to use non-geometric alignment standards, or the "free alignment method". The beauty of this system is its ability to permit design decisions to be made in the field while allowing for tighter control in areas with critical grades and alignments such as draws, switchbacks, steep topography, or ridges, and less control in areas where resource risks are minimal. Clearing and excavation quantities are substantially reduced compared to conventional geometric alignment methods. More time is spent "on the ground" in the road location step and preliminary survey so that major alignment changes are not necessary during the design phase.

The road locator runs two types of tag or grade line. On more gentle ground the tag or grade line follows closely, or is identical to the proposed road centerline (Figure 12).

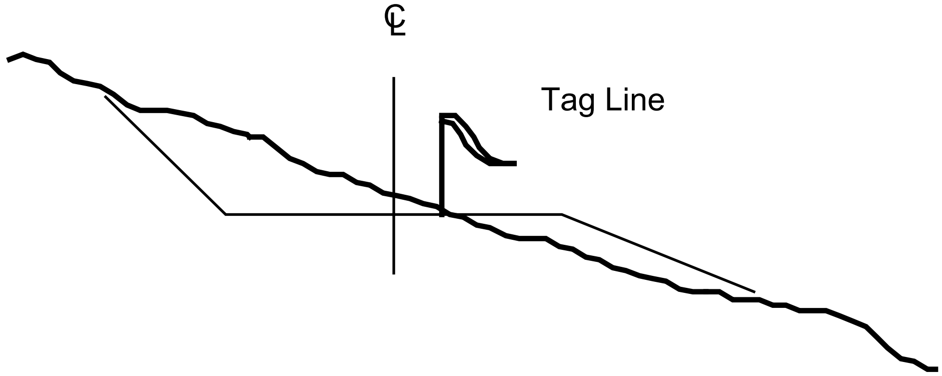

Figure 12. Tag line location and centerline location of proposed road. Sideslopes are typically less than 40 to 50 percent.

On steeper ground where heavy cuts on centerline are required (sideslopes greater than 50 to 60 percent), the tag line is marked on the "grade-out" or "daylight" point (Figure 13).

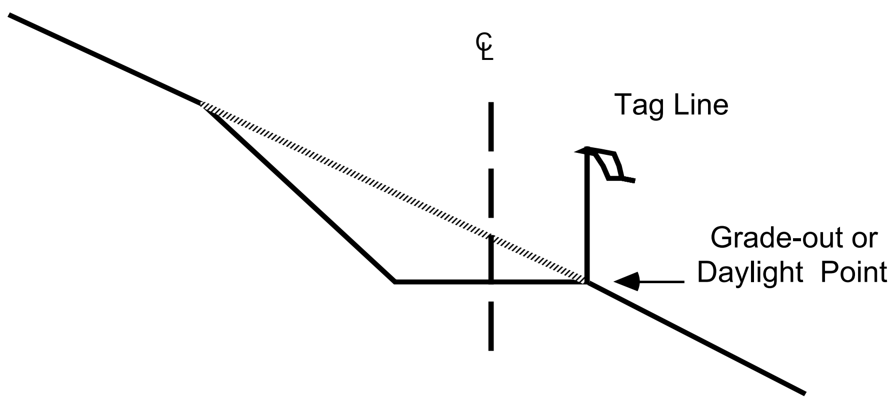

Figure 13. Tag line location and centerline location of proposed road. Sideslopes are typically 50% or steeper.

The following procedure has been proven to be successful for direct location of the centerline. First, the tag line is run with abney or clinometer. Tags, flagging, or ribbons are hung at eye level (approximately 150 to 170 cm) above ground. The ribbon should be intervisible and hung every 15 to 25 m depending on topography and vegetation density. Once a satisfactory tag line has been established, a second pass is made marking tangents and points of intersection (PI) of tangent (Figure 14).

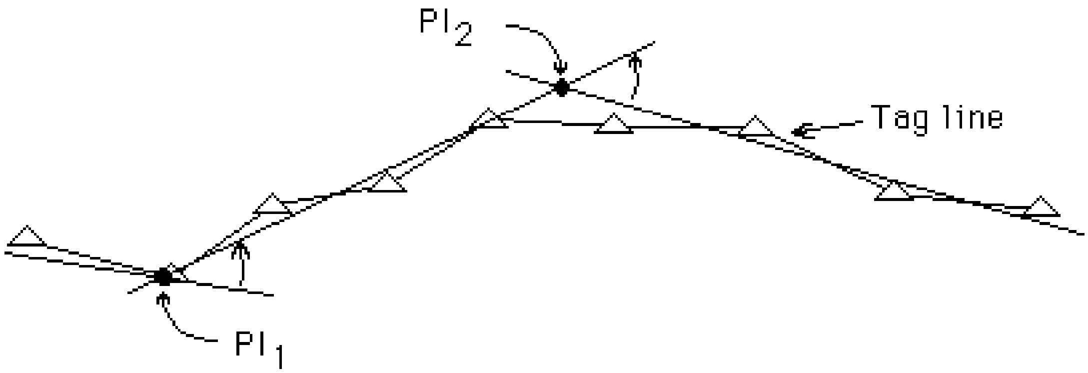

Figure 14. Selection of the road alignment in the field by "stretching the tag line"; This "stretched", or "adjusted"tag line is surveyed and represents the final horizontal location of the road.

It is good practice to cut a pole of sufficient height with brightly colored flagging to be placed at the proposed PI. This allows the road locator to clearly see the proposed tangent in relation to the marked tag line. By moving the tag ribbon horizontally "on-line" with the tangent, the road locator can evaluate the required cut/fill at centerline (Figure 15). Likewise, he can measure the deflection angle at the PI, and, based on the selected curve radius, determine the suitability of centerline location along the curve. As a rule, the selected tangent should be uphill for the majority of the ribbons marking the tag line. The longer the tangents are, the larger the offset will be and the greater the impact from cuts and fills. Therefore, on low volume, low design speed roads, short tangents should be favored in order to minimize earthwork. For example in Figure 14 an additional tangent could be inserted near the PI 2. As shown in Figure 15, still closer proximity of the tag line to the selected road centerline would result.

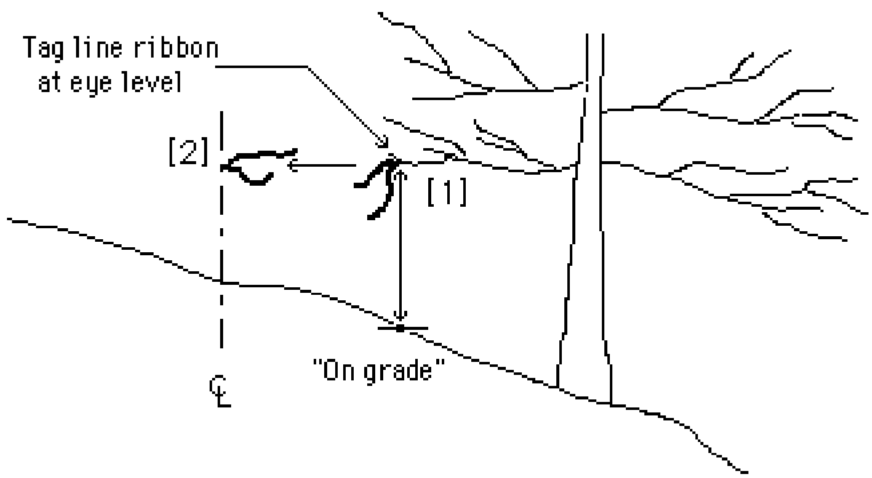

Figure 15. Position [1] shows tag line ribbon at approximately eye-level. The feet of the road locator are "on grade". Position [2] shows the ribbon on-location over the centerline or tangent as selected in the field after stretching. The ribbon has been moved horizontally, thereby allowing an estimate of required cut or fill at centerline.

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Tag lines in the field should always be run 1 or 2 percent less than the allowable maximum grade. For example, if a projected road on the map shows 10 percent grade, the road locator should use 8 or 9 percent in the field. The final design grade of the proposed road will likely be 1 or 2 percent steeper than the tag line grade in the field.

Tag line grades around sharp-nosed ridges or steep draws should be reduced, or preferably located along the proposed curve. Otherwise, the designed centerline will be significantly shorter than the marked tag line, resulting in an unacceptably steep design grade (Figure 16).

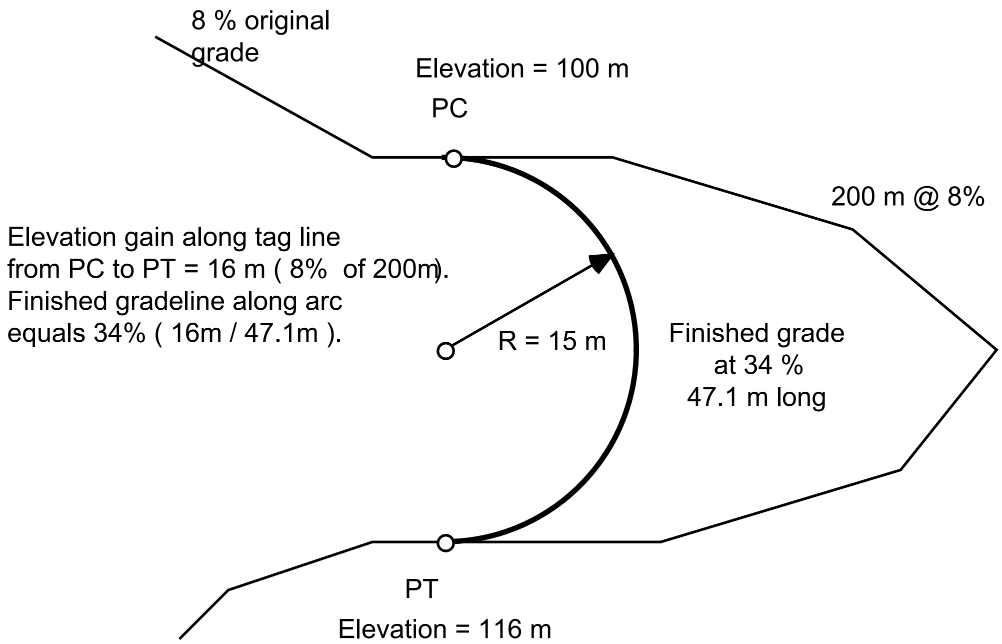
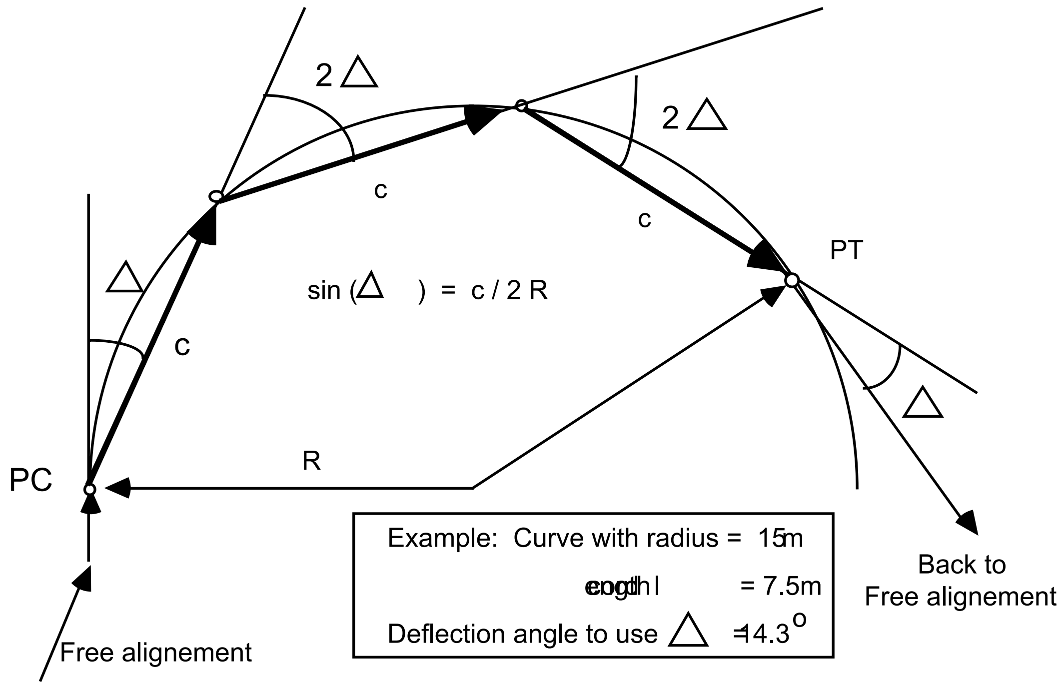


Figure 16. Example of the effect of shortened centerline through a draw or around a sharp ridge. This situation develops when running the tag line into the draw or around a sharp ridge without allowing for proper curve layout and design location.

In such cases, the tag line should be set "on location"by setting curve points using the deflection method (Figure 17). The points are selected with hand compass by turning the appropriate deflection angle and measuring the corresponding chord length.



Note: Following deflection angles are double the first deflection angle

Figure 17. Curve layout by deflection method, a useful approach during the original road location phase.

By setting the ribbon to the corresponding grade percent, the road locator can immediately evaluate the effect of his decision. Table 15 lists some convenient deflection angles and cord lengths for various curve radii.

table 20 Deflection angles for various chord lengths and curve radii.

Radius of curve (meters)	Deflection per meter	Chord Lengths c (meters)		
		5	7.5	10
degrees / meter	Deflection Angles (degrees)*			
15	1.9	9.6	14.3	19.1
20	1.4	7.2	10.7	14.3
25	1.15	5.7	8.6	11.5
30	0.96	4.8	7.2	9.6
35	0.82	4.1	6.2	8.2

* First deflection angle; subsequent deflection angles in layouts are double the indicated value

The following techniques during tag line installation should be followed to avoid increased final design grades:

1. In the case of steep draws, run the desired grade into the draw until the opposite hillside is at a distance equal to twice the minimum radius. Now, sight across the draw at zero grade, find that point on the other hillside and continue from that point with the original grade (Figure 18).

2. In the case of sharp ridges, the procedure is similar. Find the starting point for the curve. At that point, lay the tag line at zero percent around the ridge until you are opposite your beginning point and at the desired ending point for the curve. At this point resume your original grade.

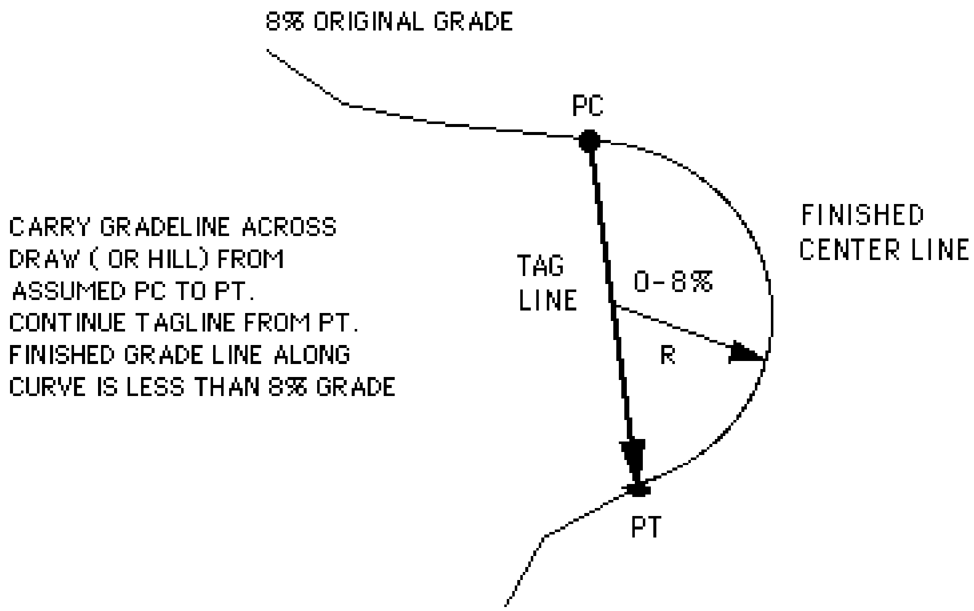


Figure 18. By sighting across draw at 0 percent grade, the desired curve is laid out without increasing the grade.

For more information on reconnaissance and road location procedure, the reader is referred to Forest Engineering Handbook (1960), by J. K. Pearce.

Location of switchbacks requires careful location in the field in order to minimize impacts on travel (excessive grades) as well as on road construction (excessive cuts and fills). As a rule, grades through a switchback at centerline should not exceed 6 to 8 percent. Because of the shortened distance along the inside road edge, the grade there will typically be 2 to 3 percent steeper. The result is that inside truck wheels will start to slip causing a "wash-board" effect. Likewise, increased erosion and sedimentation rates will result because of the continued spinout of the traction wheels. The grade along the inside edge of the road can be calculated by the following formula:

$$Grade_{R_i} = Grade_{CL} * \frac{R_{CL}}{R_{CL} - \frac{W}{2}} \quad \frac{\Delta}{180}$$

where

$Grade_{R_i}$ = grade along the inside road edge

$Grade_{CL}$ = grade along the center line of the road

R_{CL} = radius of curve to center line

Δ = deflection angle at PI

W = road width

Example: A switchback has a grade at centerline of 8 %. The deflection angle measures 160 degrees and road width (travelled width) is 3.6 meters. Additional curve widening of 1.5 meters is required on the inside of the switchback.

What is the grade along the inside edge of the road?

$Grade_{CL}$ = 8 %
Δ = 160°
R_{CL} = 10 m
W = 3.6 m

$$R_{CL} - \frac{W}{2} - \text{additional widening} = 6.7 \text{ m}$$

$$Grade_{R_i} = 8 * (10/6.7)*(160/180) = 10.6\ \%$$

The grade along the inside would be 10.6%, considerably higher than what is desirable.

Several steps can be taken to minimize the impact of excessive grade. If the grade cannot be reduced through a larger radius, for example, adequate surface material should be used that can withstand the added tire action and provide enough traction to prevent spinout. Switchbacks should not be located on slopes in excess of 35 percent because of the excessive amount of earthwork required. Natural topographic features, such as benches, saddles, or ridge tops should be used for locating switchbacks. The following example illustrates the effect of slope on cuts and fills (Figure 19):

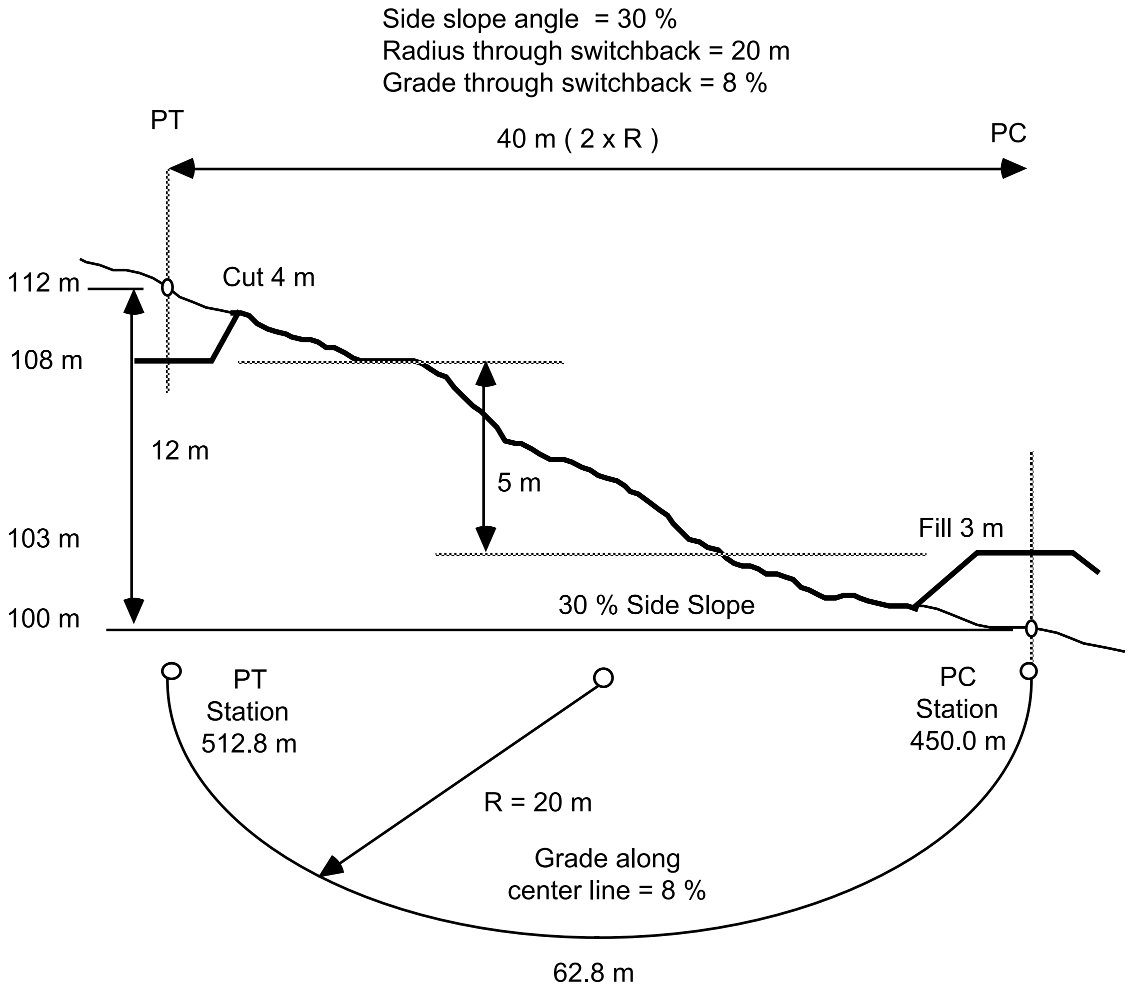


Figure 19. Cut and fill apportioning through a switchback to maintain a given grade.

From this it follows that an elevation difference (DE) of 12 m has to be overcome between the PC (beginning point) and PT (ending point) of the switchback. However, road length along centerline is 20 * π = 62.8 m. The required grade of 8 % along 62.8 m overcomes only 5.0 m of the total DE of 12 m. Therefore, 7 m (12 m - 5 m) have to be made up through either cuts or fills. Local conditions would dictate how the 7m would be apportioned between cuts and fills. (For example, 4 m of cut at the PT and 3 m of fill at the PC would be required to overcome the elevation difference on a 30 percent sideslope.). As a general rule "cutting" or excavation should be favored over filling or embankments. Proper fills are more difficult to construct than excavations.

2.3.2 Faults

Alternative routes should be carefully reviewed in the office and at the site, utilizing all available background information and technical expertise. Among the most useful tools available to the road engineer is a recent set of aerial photos. These must be of a scale small enough to reasonably identify surface features such as natural drainage characteristics, topographic characteristics (ridgelines, slope gradients, floodplains, wet areas, landslides), existing cultural features (roads, buildings, etc.), vegetation or stand type and density, bare soil areas, and geologic features such as faults.

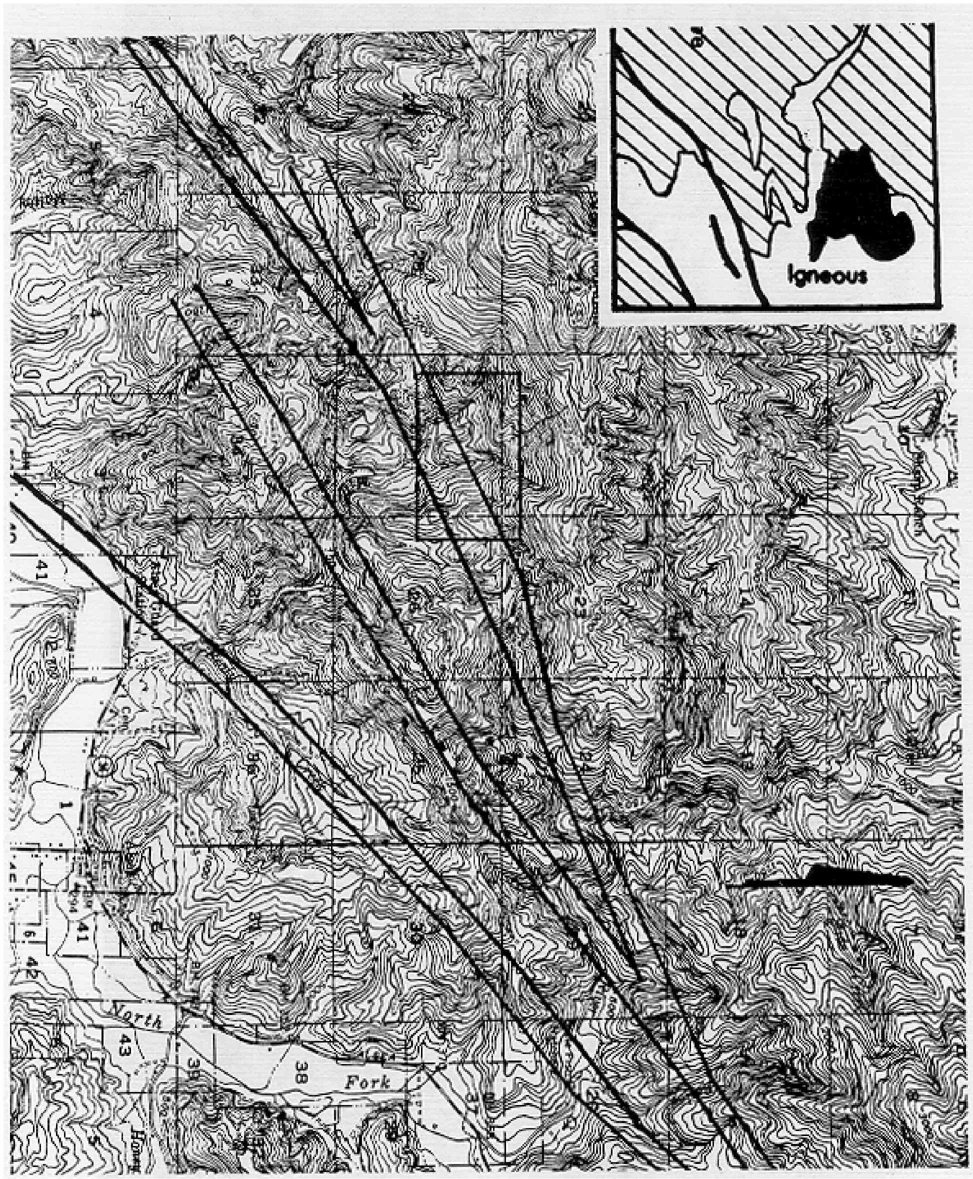


Figure 20. Suspected fault zones are indicated by the alignment of saddles in ridges and by the direction of stream channels. Geologic map is found in upper left corner. Major faults are shown as heavy dark lines on geologic maps (Burroughs,et al.,1976)

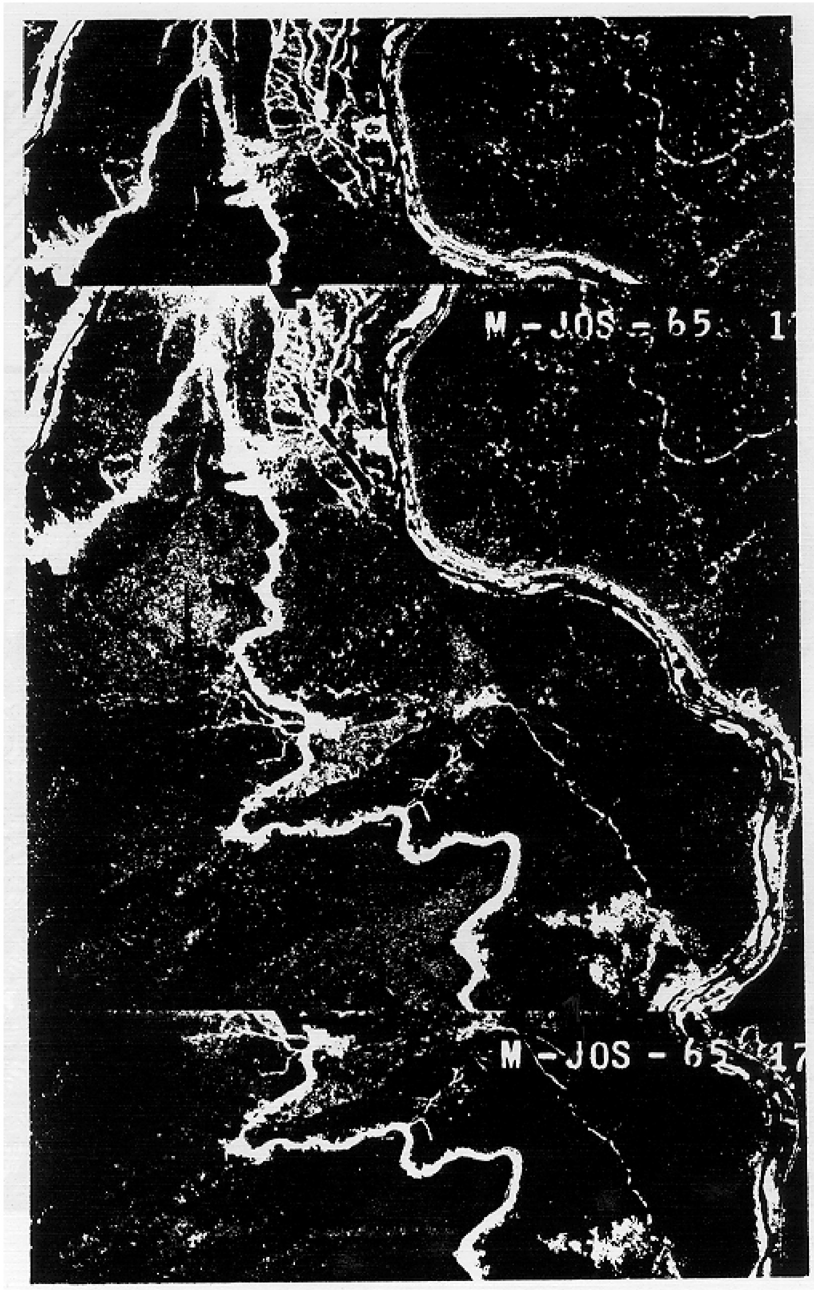


Figure 21. Stereogram of a possible fault zone. The location of the fault is indicated by the dashed line through the low saddle between the large, older slump at A and the newer slope failure at B (Burroughs, et al.,1976).

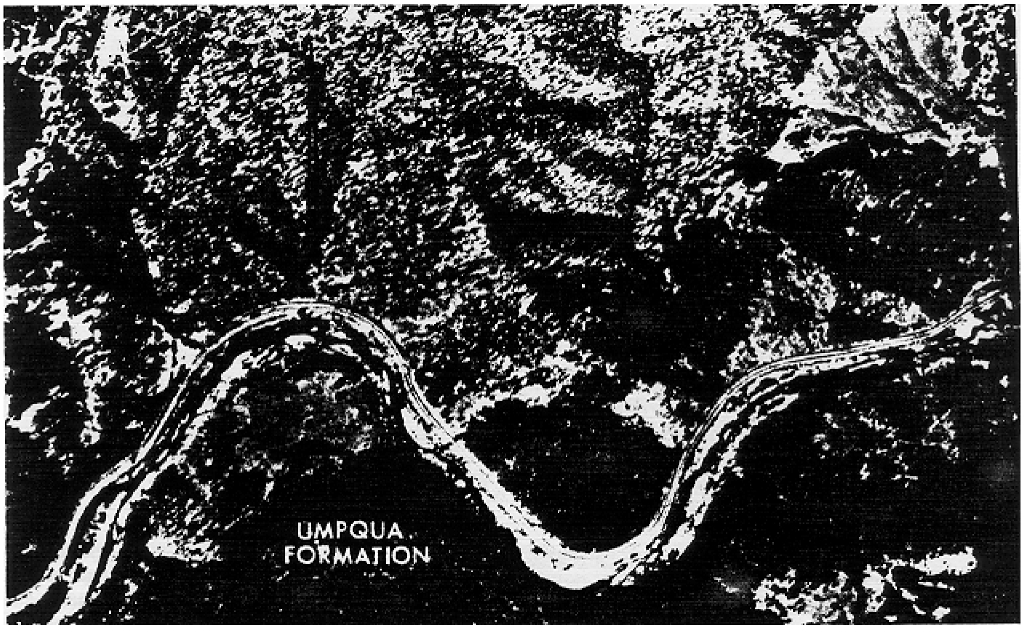


Figure 22. Approximate boundary between serpentine (metamorphic rock) material and the Umpqua formation is shown by the dashed line. The determination is based primarily on the basis of vegetation density. Timber on portions of the Umpqua formation have been harvested which accounts for a reduction in vegetation density, particularly in the northwest corner of the photo. (Burroughs, et al., 1976)

Many of the geologic features that affect slope stability can be detected in the field and on topographic maps and photos. Mountain ranges will often indicate a pronounced directional trend in which faulting can be identified. Since faults are focal points for stress relief and for intrusions of igneous and metamorphic rocks, these zones usually contain rock that is fractured, crushed, partially metamorphosed, or highly weathered and are critical to road location. (Burroughs, et al., 1976) Overlaying geologic maps with topographic maps often reveals the location of major fault zones (Figure 20). Indicators of fault zones include saddles, or low sections in ridges, which are aligned in the same general direction from one drainage to another and streams that appear to deviate from the general direction of nearby streams. Aerial photographs can be examined for clues to possible fault zones when neither geologic nor topographic maps can provide assistance or are unavailable. Figure 21 is a stereogram of an area in southwest Oregon and indicates a possible fault zone that passes through several saddles and begins and ends in the river channel. A large old slide is indicated at A and a newer slide at B. Maps and photos will also provide clues as to the relative engineering properties, or competence, of rocks in the area.

Geologic maps and topographic maps can help locate boundaries between geologic materials with different values of competence and resistance to weathering. Changes in vegetation patterns on aerial photos can also help in identifying such boundaries (Figure 22). Field personnel should be alert for on-the-ground indicators of faulting --fractured and uptilted rock and individual rocks with "slickensides", or shiny surfaces resulting from the intense heat developed by friction on sliding surfaces within the fault zone.

2.3.3 Indicators of Slope Stability

Certain features can serve as indicators of potential slide-prone areas. With some practice, these can be easily identified in the field.

Hummocky topography. This type of landscape generally contains depressions and uneven ground that has resulted from continued earthflow or slumping. Some areas that are underlain by particularly incompetent parent material, deeply weathered and subject to heavy rainfall, show a characteristically hummocky appearance (Figure 23). "Sag ponds (areas of standing water),seeps,and springs are often found within these areas. Certain plant species, called hydrophytes, frequently indicate the presence of groundwater near the surface and potential instability.

Pistol-butted, tipped and "jackstrawed" trees. Pistol-butted trees were tipped downslope while small as a result of sliding soil or debris, or as a result of active soil creep. As the tree grew, the top regained a vertical posture. These are good indicators of slope instability in areas with climates dominated by rain; deep heavy snowpacks at high elevations may also cause pistol-butting. Tipping and jackstrawed or "crazy" trees that lean at many different angles within the stand indicate unstable soils and actively moving slopes.

Tension cracks or "cat steps". Soil movement builds up stresses in the soil mantle which are sometimes relieved by tension cracks. These features may be hidden by vegetation but are a definite indicator of active movement.

Soil mottling. When groundwater is present intermittently within the soil mantle, the iron compounds present in the soil will oxidize to form distinctive orange or red spots. If groundwater levels are more persistent throughout the rainy season, iron reduction occurs giving the soil profile a gray or bluish-gray color. The occurrence of these "gleyed" soils indicates a soil that is saturated for much of the year. The presence of mottles alone is not an indication of instability, but together with other indicators such as those described can point to the need for special consideration in the location and design of a road. They often point to the need for drainage and/or extra attention to the suitability of a subsoil for foundation material.

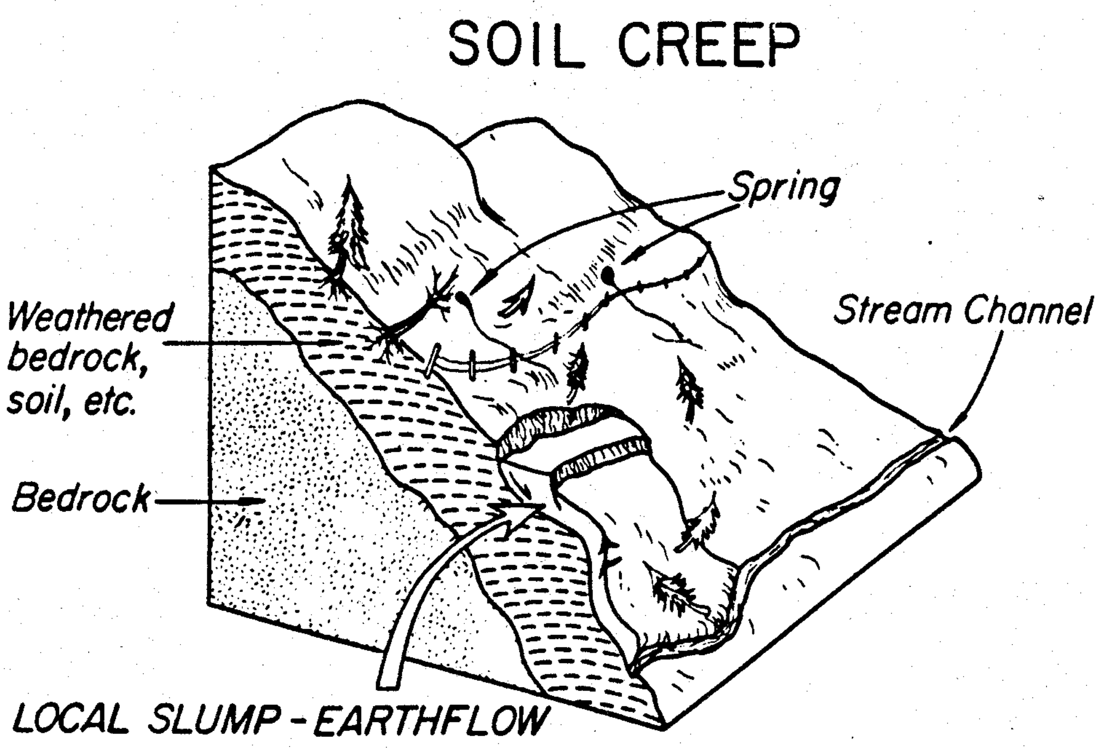


Figure 23. "Hummocky" topography with springs, curved or tilted trees, and localized slumps characterize land undergoing active soil creep.

Less quantitative methods involve subjective evaluations of relative stability using soils, geologic, topographic, climatic, and vegetative indicators obtained from aerial photos, maps, and field observations A headwall rating system such as the one presented in Figure 24 can be used to broadly evaluate relative stability of a particular site. The rating obtained in the field is entered into an empirical slope stability model to evaluate various timber harvesting options. As with most subjective rating systems, consistency among field personnel is a major problem. However, they accurately represent the relative importance of individual factors and their effects on likelihood of failure by mass movement type. The weighted values for hazard indices are presented as guides only, and can be adjusted to reflect local conditions.

HEADWALL RATING SYSTEM

SALE ____________ UNIT ____________

HEADWALL I.D. ____________ DATE ____________ SCORE [ ]

ZONE OF INTEREST

① SLOPE STEEPEST PORTION ON FALL LINE, LOWER 2/3 HEADWALL

(16) [ ] 90% + (8) [ ] 80% + (4) [ ] 70% + (2) [ ] <70%

② VEGETATION

(8) [ ] FEW TREES, SALMONBERRY UNDERSTORY, PISTOL BUTTING OF UPSLOPE CONIFERS

(6) [ ] PATCHY CONIFER STANDS, SOME ALDER, SOME PISTOL BUTTING, PRIMARILY SALMONBERRY UNDERSTORY

(4) [ ] EVENLY DISTRIBUTED HARDWOOD STAND, SALMONBERRY/SWORDFERN UNDERSTORY

(2) [ ] EVENLY DISTRIBUTED CONIFER STAND. STRAIGHT; SWORDFERN UNDERSTORY

③ SIDESLOPE SHAPE

(8) [ ] (6) [ ] (4) [ ] (2) [ ]

④ SOIL DEPTH

(6) [ ] Shallow <1 2 m

(4) [ ] No Data or indicators

(2) [ ] Deep >1 2 m

⑤ HEADWALL CONFIGURATION

(8) [ ] Multiple Convergent Depressions

(4) [ ] Single Depression

⑥ SLOPE ASPECT

(6) [ ] North 270°-090°

(3) [ ] South 090°-270°

⑦ MICROTOPOGRAPHY

(12) [ ] TENSION CRACKS, ISLANDS HUMMOCKY MICRO-RELIEF BLOWDOWN OFTEN COMMON

(8) [ ] SCARPS, BENCHES, BULGES, OFTEN SCATTERED BLOWDOWN

(4) [ ] SMOOTH, GENERALLY EVEN SLOPE

*RECENT SLIDING (0-10 YEARS), ADD 5 PTS.

COMMENTS

ANIMAL BURROWS, SOIL PIPES
LINEATIONS/FAULTS ON PHOTOS
SEEPS, UNUSUAL AMOUNTS OF WATER PRESENT
IGNEOUS BANDS (DIKES-SILLS)
EXPOSED OR RESISTANT BEDROCK BAND/FACES
HIGH POTENTIAL LEAVE AREA BLOWDOWN

OTHER COMMENTS

SLOPE FAILURE POTENTIAL

HIGH	> 48
MODERATE	34-48
LOW	< 34

Figure 24. Empirical headwall rating system.used for shallow, rapid landslides on the Mapleton Ranger District, U.S. Forest Service, Region 6, Oregon.

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CHAPTER 3

ROAD DESIGN

3.1 Horizontal and Vertical Alignment

Centerline alignment influences haul cost, construction cost, and environmental cost (e.g., erosion, sedimentation). During the reconnaissance phase and pre-construction survey the preliminary centerline has been established on the ground. During that phase basic decisions regarding horizontal and vertical alignment have already been made and their effects on haul, construction, and environmental costs. The road design is the phase where those "field" decisions are refined, finalized and documented.

3.1.1 Horizontal Alignment Considerations

The preferred method for locating low volume roads discussed in Section 2.3, the so called non-geometric or "free alignment" method, emphasizes the importance of adjusting the road alignment to the constraints imposed by the terrain. The main difference between this and conventional road design methods is that with the former method, the laying out and designing of the centerline offset is done in the field by the road locator while substantial horizontal offsets are often required with the latter method (Figure 25).

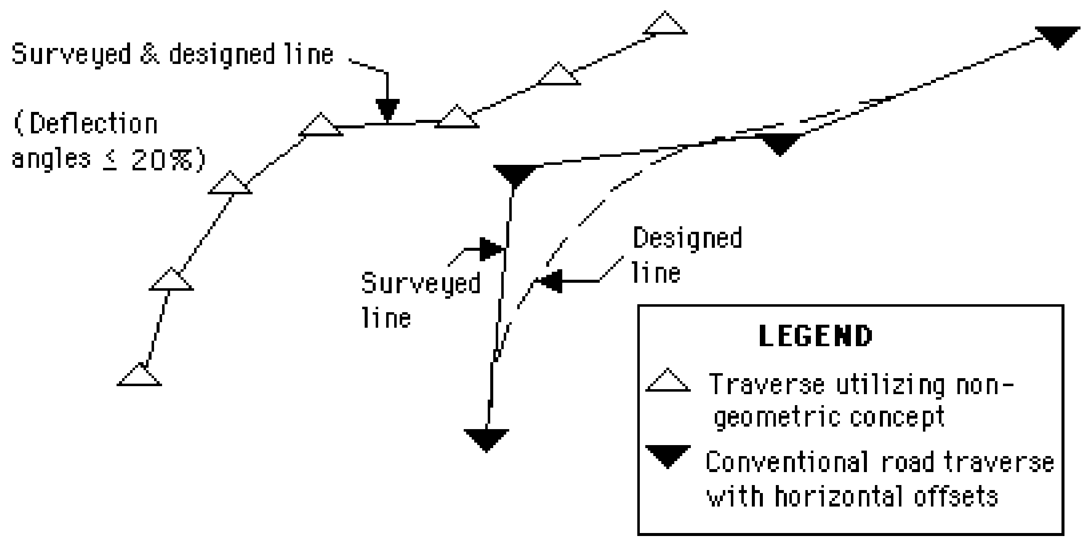


Figure 25. Non-geometric and conventional p-line traverses

Adjustments in horizontal alignment can help reduce the potential for generating roadway sediment. The objective in manipulating horizontal alignment is to strive to minimize roadway cuts and fills and to avoid unstable areas. When unstable or steep slopes must be traversed, adjustments in vertical alignment can minimize impacts and produce a stable road by reducing cuts and fills. The route can also be positioned on more stable ground such as ridgetops or benches. Short, steep pitches used to reach stable terrain must be matched with a surface treatment that will withstand excessive wear and reduce the potential for surface erosion. On level ground, adequate drainage must be provided to prevent ponding and reduce subgrade saturation. This can be accomplished by establishing a minimum grade of 2 percent and by rolling the grade.

Achieving the required objectives for alignment requires that a slightly more thoughtful preliminary survey be completed than would be done for a more conventionally designed road. There are two commonly accepted approaches for this type of survey: the grade or contour location method (used when grade is controlling), or the centerline location method (used when grades are light and alignment is controlling).
Figure 26 illustrates design adjustments that can be made in the field using the non-geometric design concept discussed earlier.

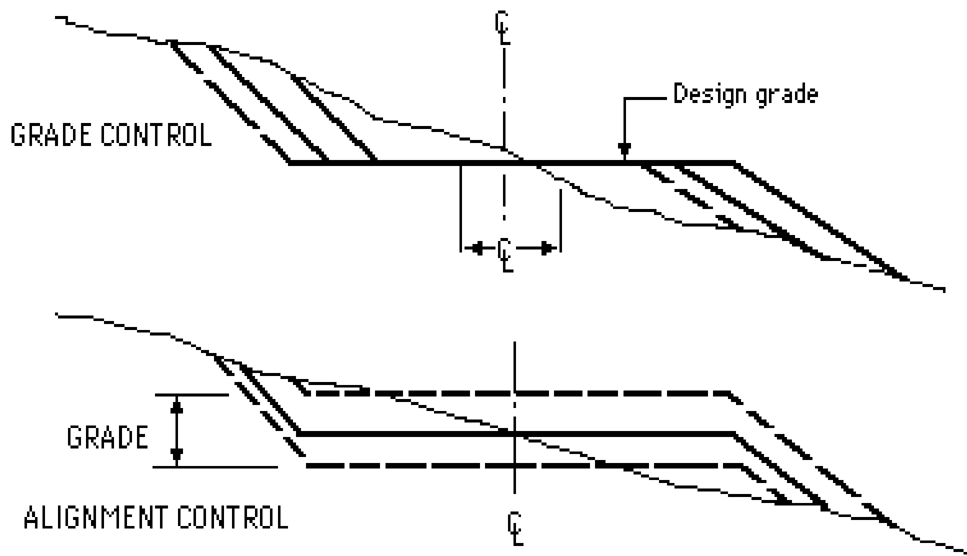


Figure 26. Design adjustments.

Equipment needed for either method may include a staff compass, two Abney levels or clinometers, fiberglass engineer's tape (30 or 50 m), a range rod, engineering field tables, notebook, maps, photos, crayons, stakes, flagging, and pencils. The gradeline or contour method establishes the location of the P-line by connecting two control points with a grade line. A crew equipped with levels or clinometers traverses this line with tangents that follow, as closely as possible, the contours of the ground. Each section is noted and staked for mass balance calculations. Centerline stakes should be set at even 25- and 50-meter stations when practicable and intermediate stakes set at significant breaks in topography and at other points, such as breaks where excavation goes from cut to fill, locations of culverts, or significant obstructions.

On gentle topography with slopes less than 30 percent and grade is not a controlling factor, the centerline method may be used. Controlling tangents are connected by curves established on the ground. The terrain must be gentle enough so that by rolling grades along the horizontal alignment, the vertical alignment will meet minimum requirements. In general, this method may be less practical than the gradeline method for most forested areas.

When sideslopes exceed 50 - 55 percent or when unstable slope conditions are present, it may be necessary to consider full bench construction shown in Figure 27. Excavated material in this case must be end hauled to a safe location. Normally, the goal of the road engineer is to balance earthwork so that the volume of fill equals the volume of cut plus any gain from bulking less any loss from shrinkage (Figure 28).

Road design, through its elements such as template (width, full bench/side cast), curve widening and grade affect the potential for erosion. Erosion rates are directly proportional to the total exposed area in cuts and fills. Road cuts and fills tend to increase with smooth, horizontal and vertical alignment. Conversely, short vertical and horizontal tangents tend to reduce cuts and fills. Erosion rates can be expected to be lower in the latter case. Prior to the design phase it.

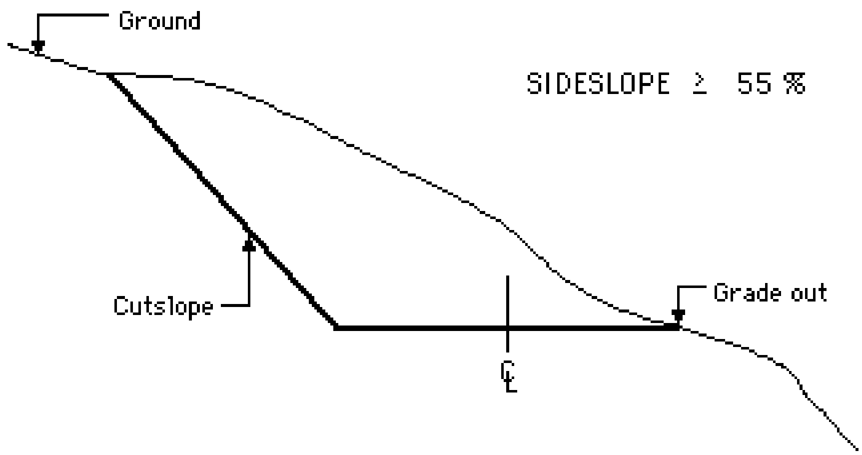


Figure 27. Full bench design.

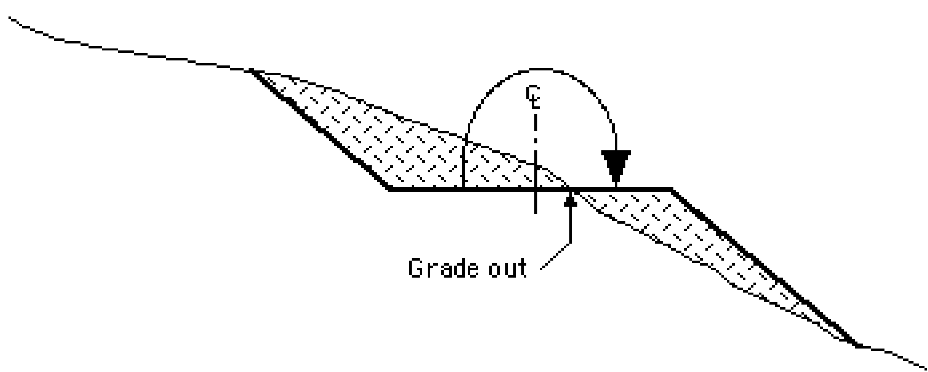


Figure 28. Self-balanced design

should be clearly stated which alignment, horizontal or vertical, takes precedence. For example, if the tag line has been located at or near the permissible maximum grade, the vertical alignment will govern. Truck speeds in this case are governed by grade and not curvature. Therefore, horizontal alignment of the centerline can follow the topography very closely in order to minimize earthwork. Self balancing sections would be achieved by shifting the template horizontally.

3.1.2 Curve Widening

Roadway safety will be in jeopardy and the road shoulders will be impacted by off-tracking wheels if vehicle geometry and necessary curve widening are not considered properly. Continually eroding shoulders will become sedimentation source areas and will eventually weaken the road. On the other hand, over design will result in costly excessive cuts and/or fills.

The main principle of off-tracking and hence curve widening, centers on the principle that all vehicle axles rotate about a common center. Minimum curve radius is vehicle dependent and is a function of maximum cramp angle and wheelbase length (see Figure 29).

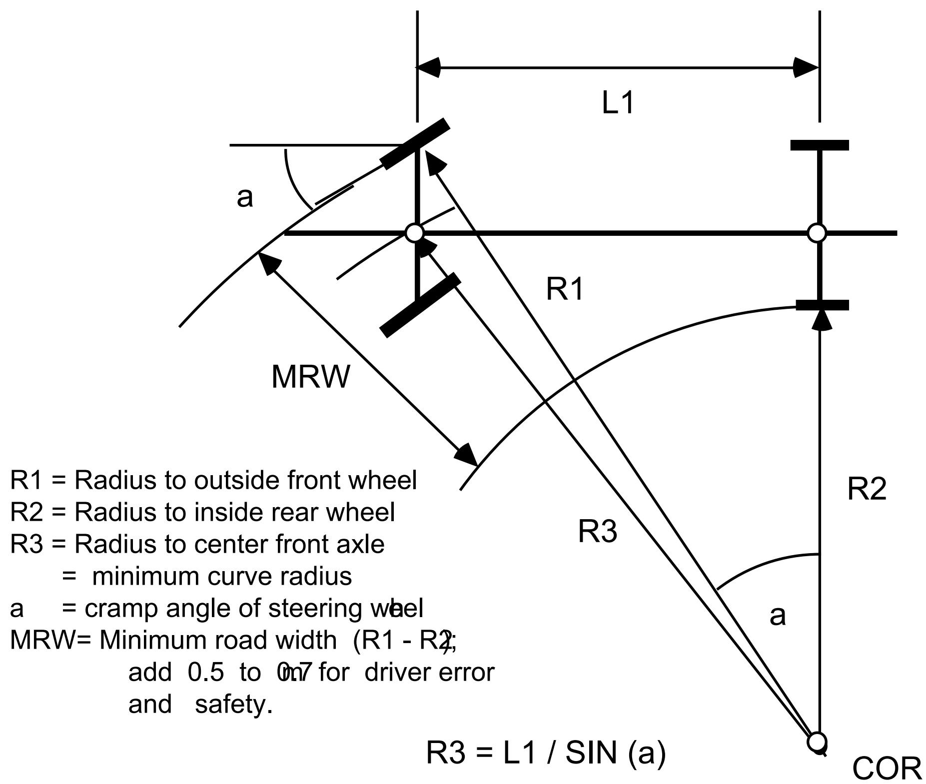


Figure 29. Basic vehicle geometry in off-tracking

Typical vehicle dimensions are shown in Figure 30 for single trucks, truck/ trailers, log truck (pole-type), and tractor/trailer combinations. These dimensions were used to develop Figures 35 to 38 for calculating curve widening in relation to curve radius and central angle.

Several different solutions to determine curve widening requirements are in use. Most mathematical solutions and their simplified versions give the maximum curve widening required. Curve widening is a function of vehicle dimensions, curve radius, and curve length (central angle).

A graphical solution to the problem is provided in Figures 31 to 33. This solution can be used for single trucks, truck-trailer combinations and vehicle overhang situations. This solution provides the maximum curve widening for a given curve radius.

TYPICAL TRUCK / TRAILER DIMENSIONS

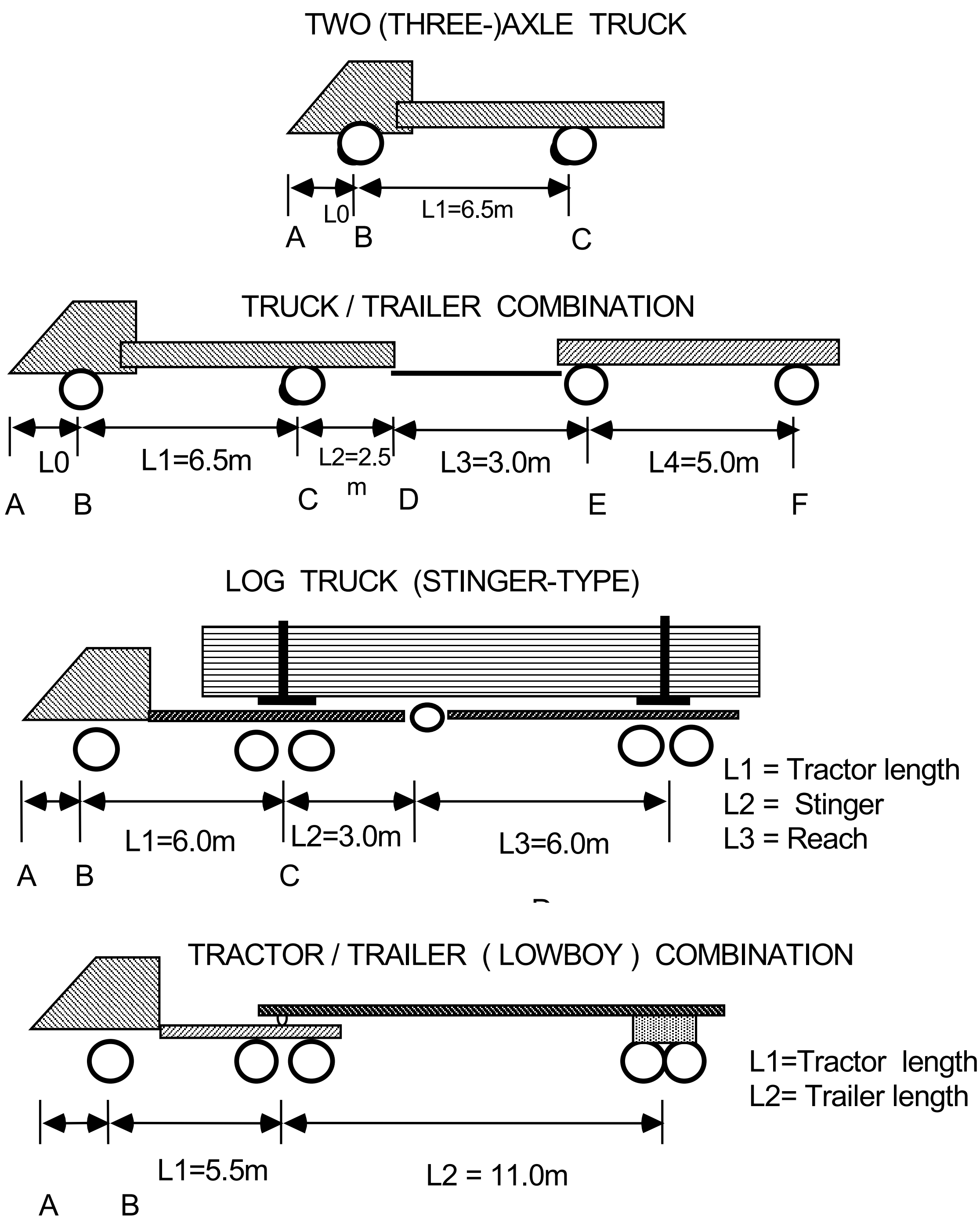
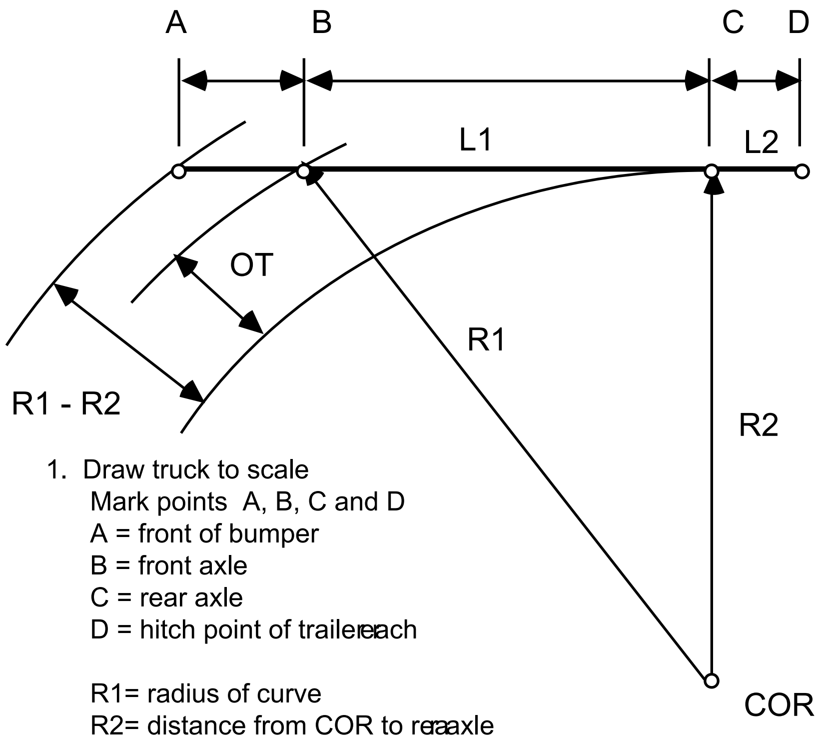


Figure 30. Example of truck-trailer dimensions.

Graphical Solution for Off-Tracking Truck / Trailer

STEP 1



1. Draw truck to scale
   Mark points A, B, C and D
   A = front of bumper
   B = front axle
   C = rear axle
   D = hitch point of trailer each

   R1= radius of curve
   R2= distance from COR to rear axle
   OT= off-tracking, front- to rear wheel
   COR= center of rotation

2. Extend rear axle (Pt. C); swing arc R1 from Pt.B
   Intersection of R1 and R2 locates COR (center of rotation)

3. The difference R1 - R2 is the off tracking of the rear axle

Figure 31. Step 1. Graphical solution of curve widening

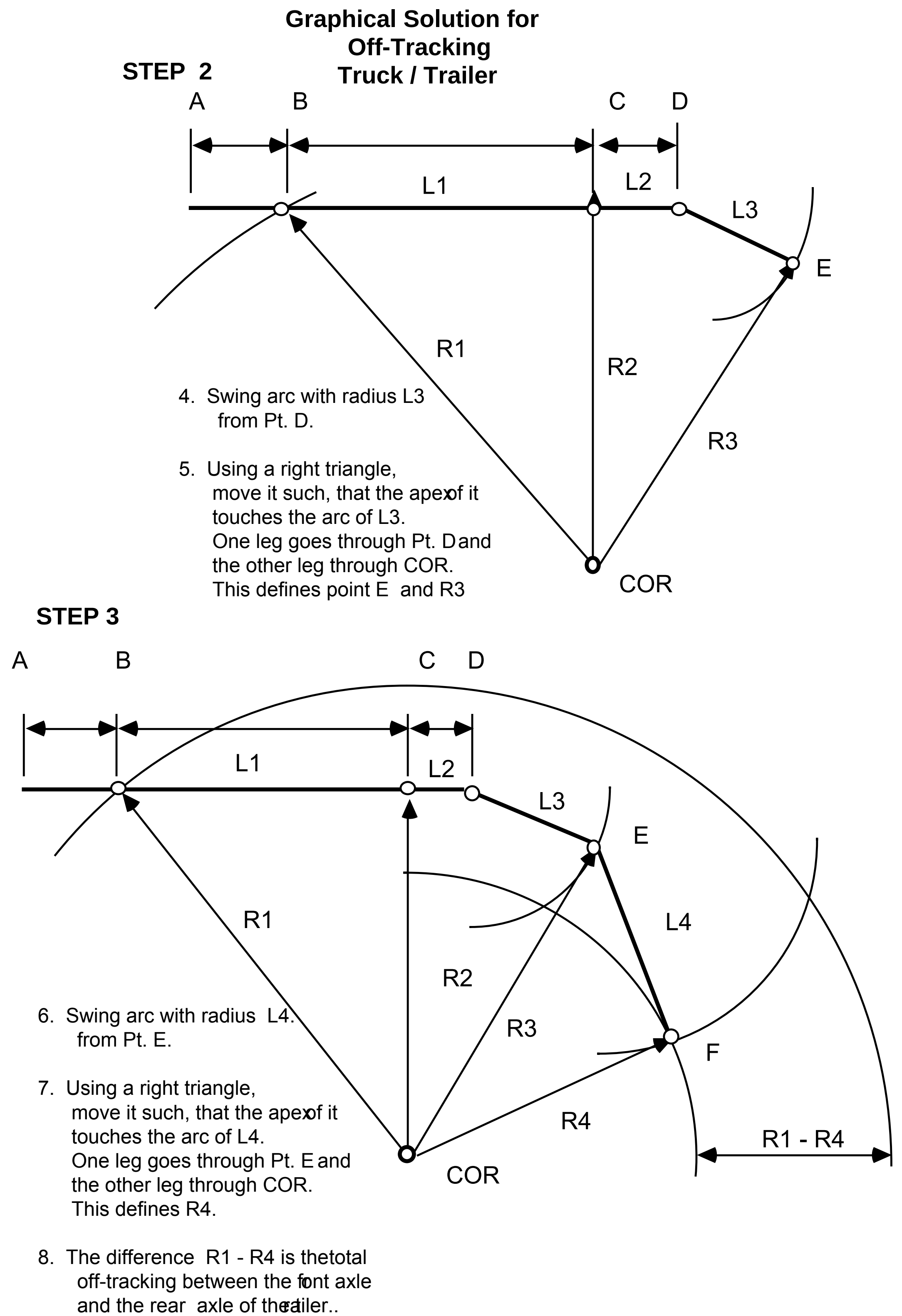


Figure 32. Steps 2 and 3; Graphical solution for curve widening.

The graphical solution for a stinger type log truck is shown in Figure 33. Here, an arc with the bunk length L2 plus L3 is drawn with the center at C. The log load swivels on the bunks, C and E and forms line C - E, with the trailer reach forming line D - E.

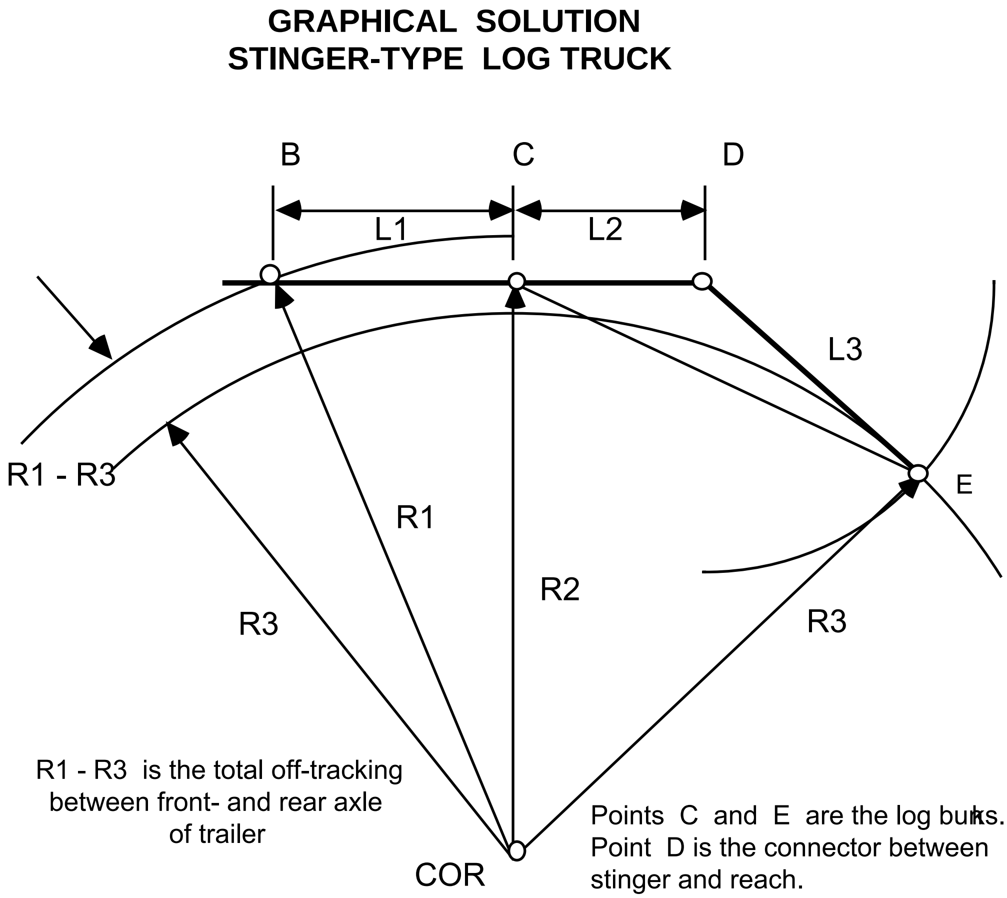


Figure 33. Graphical solution for off-tracking of a stinger-type log truck

Simple, empirical curve widening formulae have been proposed by numerous authors and government agencies. A common method used in North America is:

CW = 37/R For Tractor-trailer (low boy; units in meters)

CW =18.6/R For log-truck (units in meters)

The above equations are adapted for the typical truck dimensions used in the United States and Canada. In Europe, curve widening recommendations vary from 14/R to 32/R. Curve widening recommendations in Europe are given by Kuonen (1983) and Dietz et al. (1984). Kuonen defines the curve widening requirement for a two-axle truck (wheelbase = 5.5m) as

$$CW = 14/R$$

and a truck-trailer combination where

$$CW = 26/R$$

Dietz, et. al. (1984) recommend CW=32/R for any truck combination.

The approximation methods mentioned above are usually not satisfactory under difficult or critical terrain conditions. They typically overestimate curve widening requirements for wide curves (central angle < 45^o) and under estimate them for tight curves (central angle > 50^o).

Vehicle tracking simulation provides a better vehicle off-tracking solution because it considers vehicle geometry and curve elements, in particular the deflection angle (Kramer, 1982, Cain & Langdon, 1982). The following charts provide off-tracking for four common vehicle configurations--a single or two-axle truck, a truck-trailer combination, a stinger-type log-truck and a tractor-trailer (lowboy) combination. The charts are valid for the specified vehicle dimensions and are based on the following equation (Cain and Langdon, 1982):

$$OF = (R - (R2 - L2)\ 1/2) * (1 - e^{X})$$

where $x = (-0.015 * D * R/L + 0.216)$

OF = Off tracking (m)
R = Curve radius (m)
D = Deflection angle or central angle
e = Base for natural logarithm (2.7183)
L = Total combination wheelbase of vehicle
L = (Summation of $L_i^2)^{1/2}$

For a log-truck:

$$L= (L1^2 + L2^2 + L3^2)^{1/2}$$

where L1-3 are defined in Figure 30.

The maximum off-tracking for a given vehicle, radius and deflection angle occurs when the vehicle leaves the curve. Since vehicles travel both directions, the required curve widening, which consists of off-tracking (OT) plus safety margin (0.5 - 0.6m), should be added to the full curve length. One half of the required curve widening should be added to the inside and one half to the outside of the curve (Figure 34).

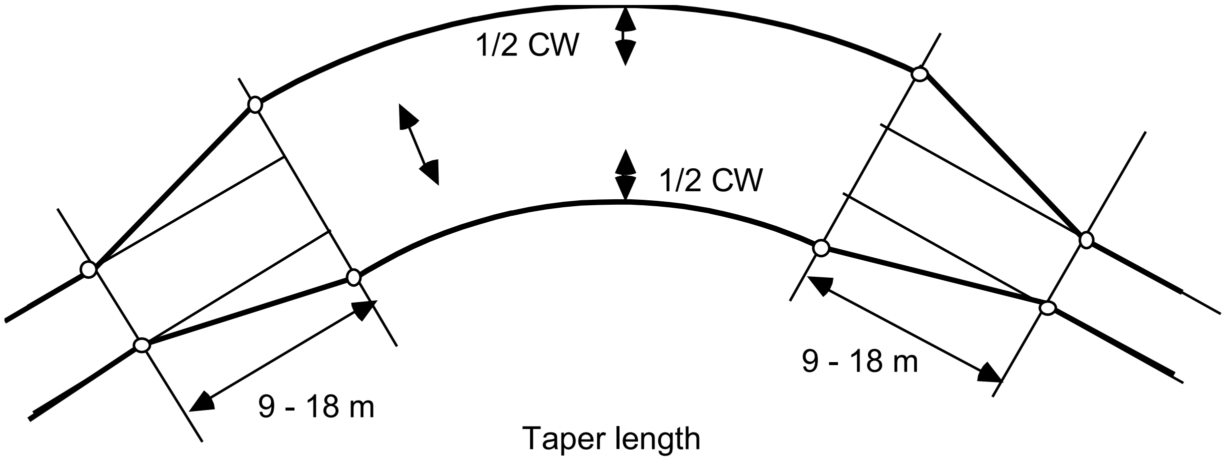


Figure 34. Curve widening and taper lengths.

Figures 35 through 38 provide vehicle off-tracking for a given vehicle, radius, and deflection (or central angle). To this value, 0.5 to 0.6m should be added to allow for formula and driver's error, grade, and road or super elevation variations.

Transition or taper length from tangent to curve vary from 9 to 18 m depending on curve radius. Recommended length of transition before and after a curve are as follows (Cain and Langdon, 1982):

Curve Radius (m)	Length of Taper (m)
20	18
20 - 25	15
25 - 30	12
30	10

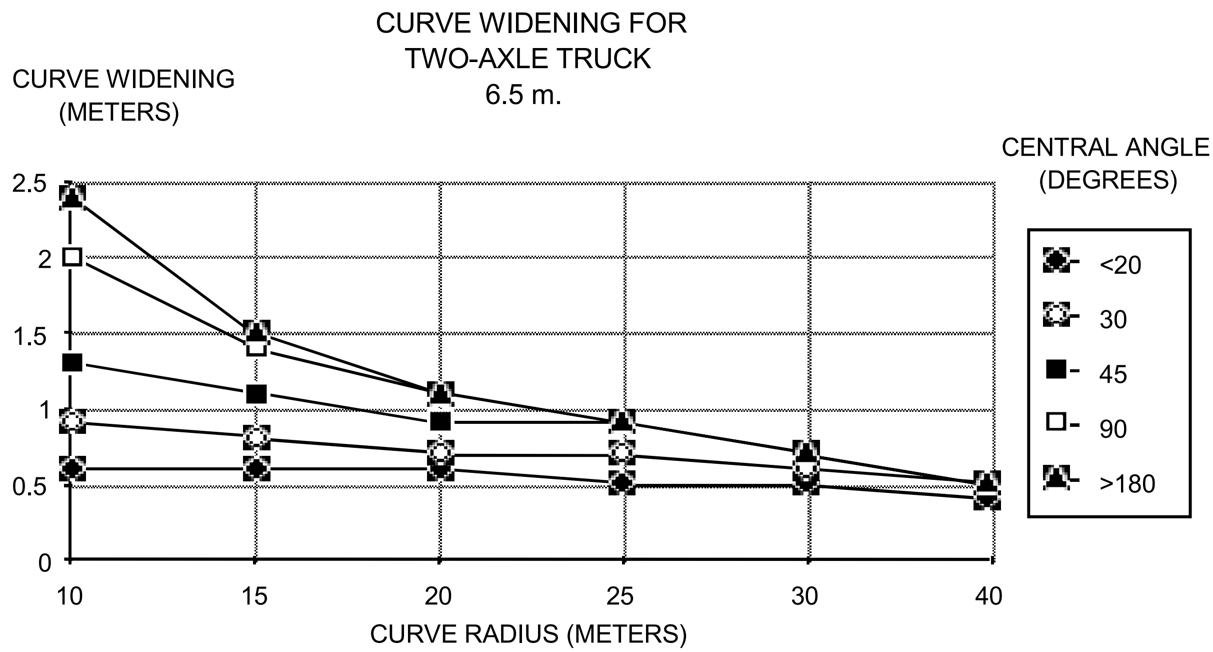
Example: Standard road width is 3.0 m. Design vehicle is a stinger-type log-truck with dimensions as shown in Figure 30. Curve radius is 22m, deflection angle equals 60^o.

From Figure 37 locate curve radius on the x-axis (interpolate between 20 and 25), go up to the corresponding 60^o- curve (interpolate between 45^o and 90^o), go horizontally to the left and read the vehicle off-tracking equal to 1.8 m.

The total road width is 4.80 m (3.0 m + 1.8 m).

Depending on conditions, a safety margin of 0.5 m could be added. The current 3.0 m road width already allows for safety and driver's error of 0.30 m on either side of the vehicle wheels (truck width = 2.40

m). Depending on the ballast depth, some additional shoulder width may be available for driver's error. Taper length would be 15 m.



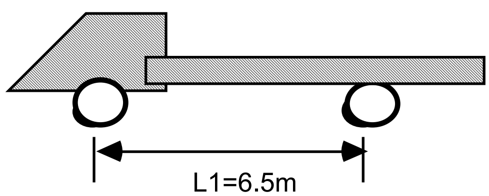
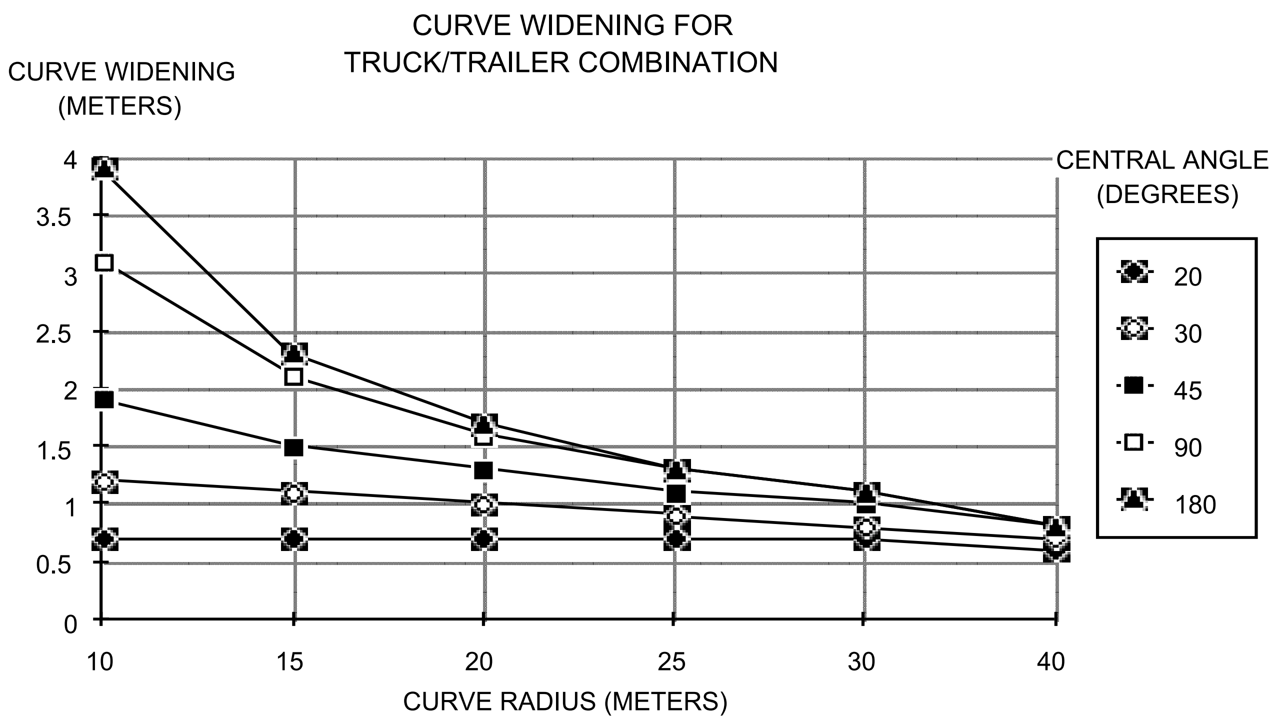


Figure 35. Curve widening guide for a two or three axle truck as a function of radius and deflection angle. The truck dimensions are as shown.



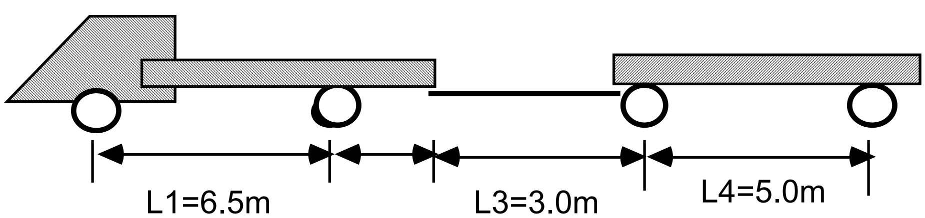
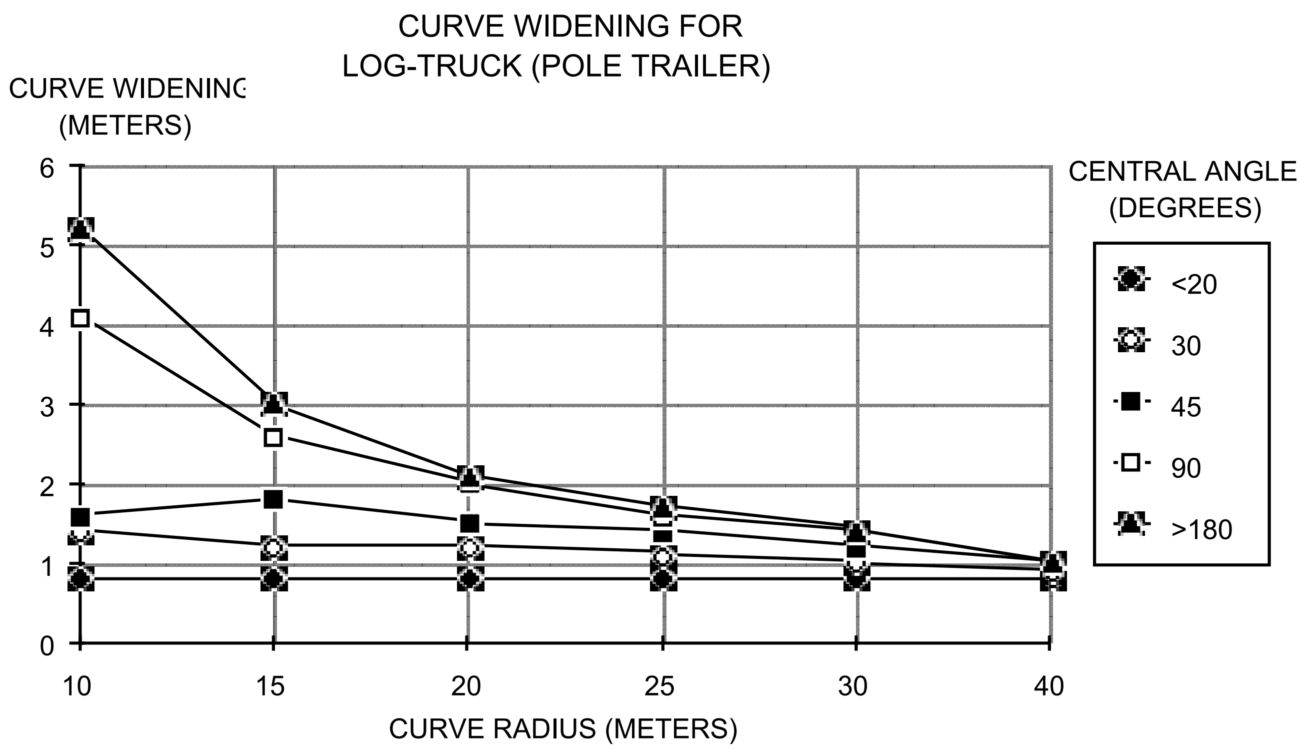


Figure 36. Curve widening guide for a truck-trailer combination as a function of radius and deflection angle. The dimensions are as shown.



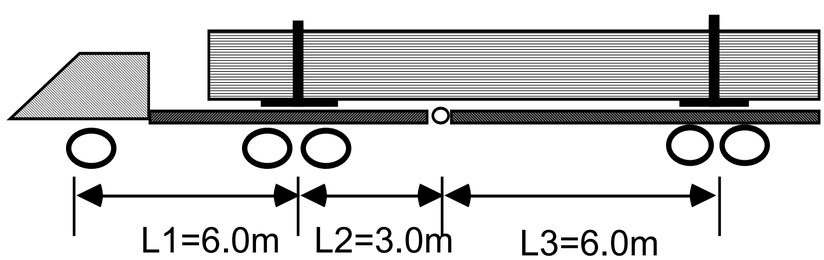
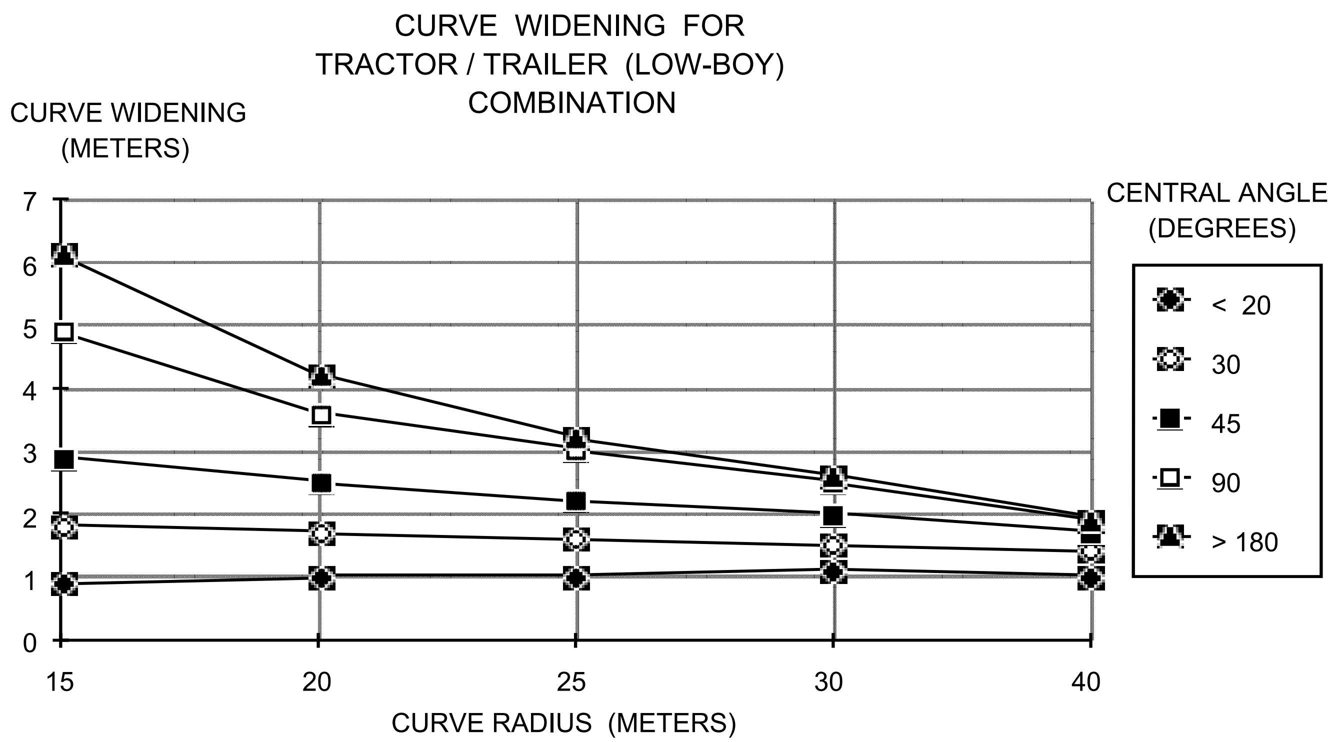


Figure 37. Curve widening guide for a log-truck as a function of radius and deflection angle. The dimensions are as shown.



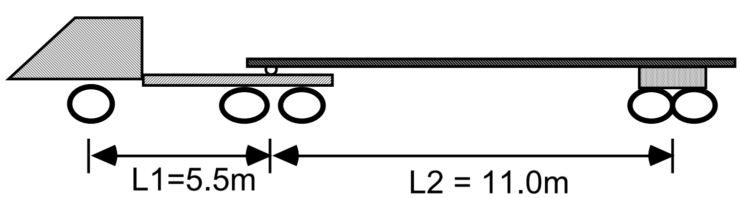


Figure 38. Curve widening guide for a tractor/trailer as a function of radius and deflection angle. The tractor-trailer dimensions are as shown.

3.1.3 Vertical Alignment

Vertical alignment is often the limiting factor in road design for most forest roads. Frequently grades or tag lines are run at or near the maximum permissible grade. Maximum grades are determined by either vehicle configuration (design/critical vehicle characteristic) or erosive conditions such as soil or precipitation patterns. Depending on road surface type, a typical logging truck can negotiate different grades. Table 16 lists maximum grades a log truck can start from. It should be noted that today's loaded trucks are traction limited and not power limited. They can start on grades up to 25 % on dry, well maintained, unpaved roads. Once in motion they can typically negotiate steeper grades.

Vertical curves or grade changes, like horizontal curves, require proper consideration to minimize earthwork, cost, and erosion damage. Proper evaluation requires an analysis of vertical curve requirements based on traffic characteristics (flow and safety), vehicle geometry, and algebraic difference of intersecting grades.

Vertical curves provide the transition between an incoming grade and an outgoing grade. For convenience in design, a parabolic curve (Figures 39 and 40) is used because the grade change is proportional to the horizontal distance. The grade change is the difference between incoming grade and outgoing grade. The shorter the vertical curve can be kept, the smaller the earthwork required.

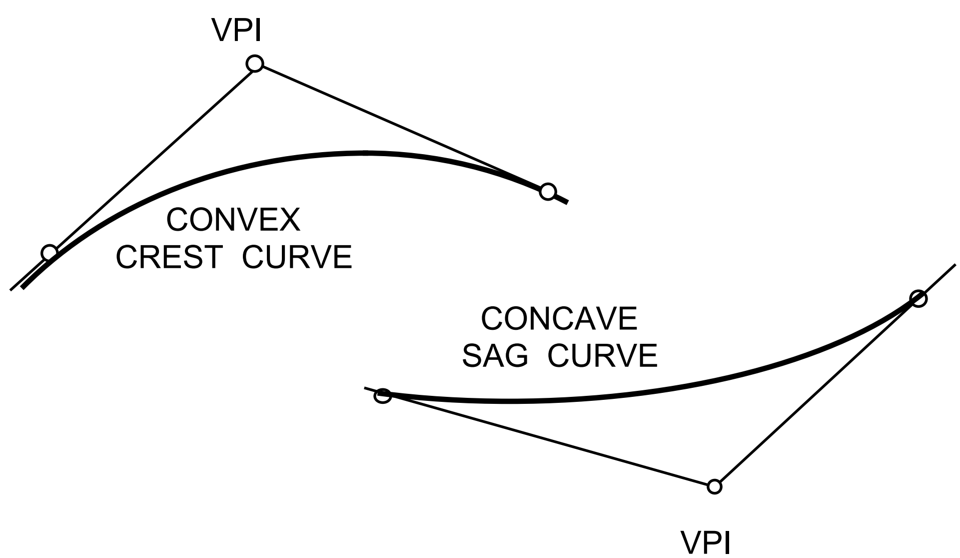


Figure 39. Typical vertical curves(VPI = Vertical Point of Intersection).

The grade change per unit length is defined as

(G1 -G2) / L (% / meter)

or more commonly its inverse, where the grade change is expressed in horizontal distance (meters) to effect a 1% change in grade.

(1) Surface	(2) Traction Coef. (f)	(3 Rolling Res. (r)	(4)* Starting Res. (s)	Maximum Starting Grades** (5) TR = .435 Loaded Truck[a]		(6) TR = .64 Empty Truck Piggyback[b]		(7) TR = .32 Empty Truck Trailor Extended[b]	
				From	To	From	To	From	To
Concrete-dry	.75-.90	.018	.10	21.6	28.1	47.0	61.4	17.6	24.8
Concrete-wet	.55-.70	.015	.10	13.1	19.6	29.6	42.5	9.0	15.5
Asphalt-dry	.55-.70	.020	.10	12.8	19.3	28.4	42.3	8.7	15.2
Asphalt-wet	.40-.70	.018	.10	6.4	19.4	17.4	42.4	2.8	15.3
Gravel-packed, oil, & dry	.50-.85	.022	.10	10.5	25.7	26.2	56.3	6.5	22.0
Gravel-packed, oil, & wet	.40-.80	.020	.10	6.3	23.7	17.4	51.6	2.7	19.8
Gravel-loose, dry	.40-.70	.030	.10	5.7	18.8	17.0	42.0	2.0	14.5
Gravel-loose, wet	.36-.75	.040	.10	3.4	20.4	13.6	46.2	-0.2	16.1
Rock-crushed, wet or dry	.55-.75	.030	.10	12.2	20.9	29.0	46.5	8.0	16.8
Earth-dry	.55-.65	.022-.03	.10	12.2	17.0	29.0	37.8	8.0	12.8
Earth-wet (excludes some clays)	.40-.50	.022-.03	.10	5.7	10.5	17.0	25.2	2.0	6.5
Dry packed snow	.20-.55	.025	.10	-2.7	12.5	2.5	29.2	-5.0	8.4
Loose snow	.10-.60	.045	.10	-8.2	13.6	-5.1	32.7	-9.8	9.1
Snow lightly sanded	.29-.31	.025	.10	1.2	2.1	8.9	10.4	-1.8	-1.0
Snow lightly sanded with chains	.34	.035	.10	2.8		12.3		0.6	
Ice without chains	.07-.12	.005	.10	-7.2	-5.1	-5.6	-2.3	-6.1	-6.4

Example

Given: earth surface road
Req'd: maximum adverse grades for the following:

1) landings
2) loaded log truck to start from rest
3) moving loaded log trucks

Assume hauling will be done during wet weather, but not ice or snow

Solution: Under Column (1), find earth-wet:

1) for landing, go across to Col. (7) truck, trailer extended, and read from 2.0 to 6.5 %.
2) for loaded log trucks starting from rest, go across to Col. (5) and read from 5.7 to 10.5 %.
3) add 10 % to part 2, which means a moving loaded log truck will 'spin out' somewhere between 15.7 to 20.5 %.

NOTE

Extreme caution is recommended in the use of steep grades, especially over 20 %. They may be impractical because of construction and maintenance problems and may cause vehicles that travel in the downhill direction to lose control.

*For vehicles with manual transmissions. Factor for wet clutches, hydraulic torque converters, freeshaft turbines, or hydrostatic transmissions would be .03 to .05.
**Add 10 % to these values to obtain the maximum grade a log truck may negotiate when moving.
[a]Based upon = f(TR) - r(1 - TR) - S
[b]Based upon = f(TR)/(1-(f(h)/b)) - r(1 - TR) - S
h = height of trailer coupling or center of gravity (1.2 m)
b = wheel base (5.5 m) formulas from source 2 values in Col. 2 & 3 are composite.

table 21 Maximum grades log-trucks can start on from rest (Cain, 1981).

VERTICAL CURVE ELEMENTS

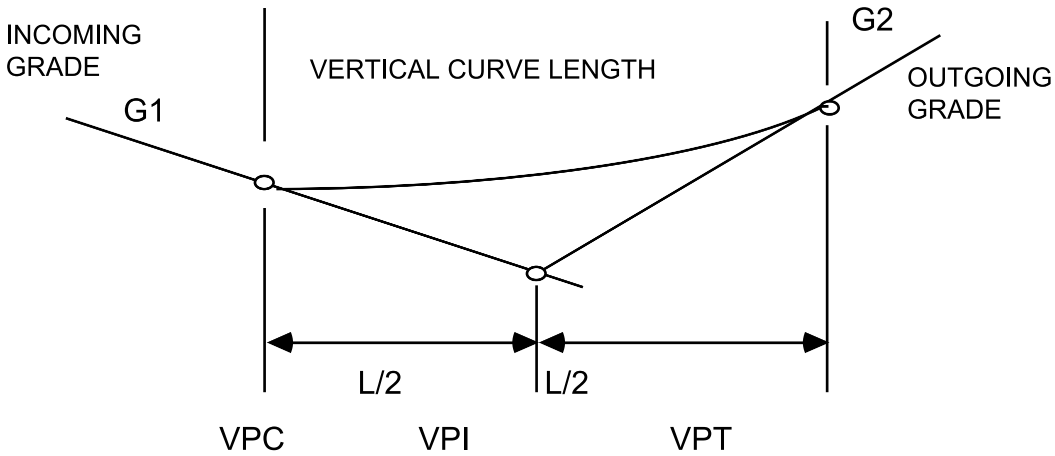


Figure 40. Vertical curve elements (VPC = Vertical Point of Curvature; VPT = Vertical Point of Tangency).

Factors to be considered in the selection of a vertical curve are:

Stopping Sight distance S: On crest curves, S is a function of overall design speed of the road and driver's comfort. On most forest roads with design speeds from 15 km/hr to 30 km/hr, the minimum stopping sight distance is 20 and 55 meters respectively (see Ch. 2.1.2.7). Kuonen(1983) provides an equation for minimal vertical curve length based on stopping distance:

$$L_{min} = S_{min}^2 / 800$$

Where L_{min} = minimum vertical curve length for each 1% change in grade (m/%)
S_{min} = minimum safe stopping sight distance (m).

Example: Determine the minimum vertical curve length for a crest curve that satisfies the safe stopping sight distance.

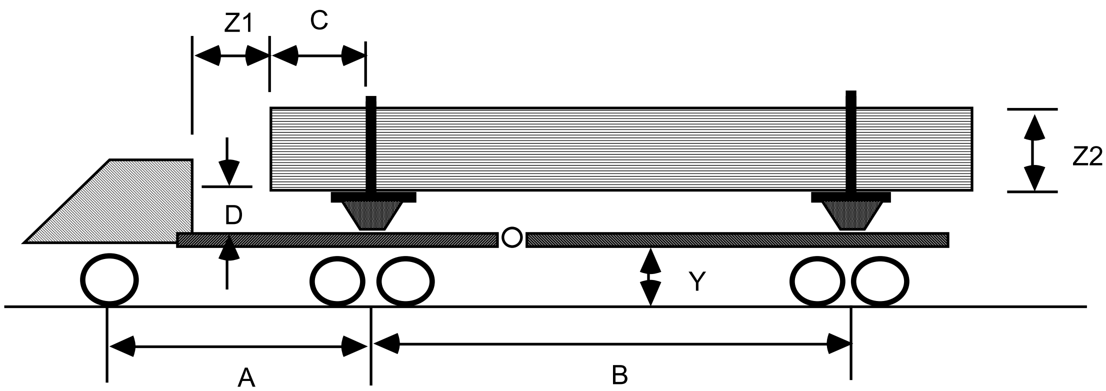
Design speed of road: 25 km/hr
Grade change (G1 - G2): 20 %

Solution: Stopping sight distance for 25 km/hr equals approximately 37 meters (from Ch. 2.1.2.7).

$L_{min} = (37^2 / 800)\ 20 = 34.2$ m

Vehicle geometry: Vehicle clearance, axle spacing, front and rear overhang, freedom of vertical movement at articulation points are all factors to be considered in vertical curve design.

Passage through a sag curve requires careful evaluation of the dimensions as illustrated in Figure 41.



D = Clearance between top of the frame of the truck and the bottom of the logs at the front
Y = Distance between the ground and the bottom of the trailer reach or stinger
Z1= Distance between the front of the logs and the cab of the truck which depend on C and Z2
Z2= Height of log load

Figure 41. Log truck geometry and dimensions for vertical curve analysis

The critical dimensions of a log truck when analyzing crest vertical curves are the length of the stinger and the vertical distance between the stinger and the bottom of the logs, x. A log truck as shown in Figure 41 with dimensions

A - Tractor length	=	4.8 m
B - Bunk to Bunk	=	7.2 m
C - Log overhang front	=	2.4 m
D - Clearance log - frame	=	0.39 m

could negotiate a grade change of 30% over a vertical curve length of 12 m without damage to the truck (Ohmstede, 1976).

As shown in the previous example, safety considerations typically require significantly longer, vertical curves than physical truck dimensions do. With the exception of special or critical vehicles, vertical curves can be kept very short, even for large grade changes. Road maintenance considerations are more important in such situations. Vehicle dimension considerations do become important, however, in special cases such as fords in creek crossings.

3.2 Road Prism

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. Overroading or misplacement of roads results from a poor land management or transportation plan; overdesign results from rigidly following design criteria with respect to curvature, width, gradient, and oversteepened cuts and fills or from designing roads to higher standards than are required for their intended use. As stated previously, allowing terrain characteristics to govern road design permits more flexibility and will be especially beneficial, both environmentally and economically, where it is possible to reduce cut and fill slope heights, slope angles, and roadway widths.

3.2.1 Road Prism Stability

Stability considerations as applied to natural slopes are also valid for stability analysis of road cuts and fills. Points to consider include

- Critical height of cut slope or fill slope
- Critical piezometric level in a slope or road fill
- Critical cut slope and fill slope angle.

The most common road fill or sidecast failure mode is a translational slope failure. Translational slope failure is characterized by a planar failure surface parallel to the ground or slope. Depth to length ratio of slides are typically very small. The following slopes would fall into this category:

1. Thin, residual soil overlaying an inclined bedrock contact
2. Bedrock slopes covered with glacial till or colluvium
3. Homogeneous slopes of coarse textured, cohesionless soils (road fills)

Fill slope failure can occur in two typical modes. Shallow sloughing at the outside margins of a fill is an example of limited slope failure which contributes significantly to erosion and sedimentation but does not directly threaten the road. It is usually the result of inadequate surface protection. The other is sliding of the entire fill along a contact plane which can be the original slope surface or may include some additional soil layers. It results from lack of proper fill compaction and/or building on too steep a side slope. Another reason could be a weak soil layer which fails under the additional weight placed on it by the fill.

Slope or fill failure is caused when forces causing or promoting failure exceed forces resisting failure (cohesion, friction, etc.). The risk of failure is expressed through the factor of safety (see Figure 2):

$$FS = \text{Shear strength} / \text{Shear stress}$$

where shear strength is defined as

$$T = C * A + N (\tan [f])$$

and shear stress, the force acting along the slope surface, is defined as

$$D = W * \sin[b]$$

where C = Cohesive strength (tonnes/m^2)
A = Contact area (m^2)
W = Unit weight of soil (tonne/m^3)
[b] = Ground slope angle
[f] = Coefficient of friction or friction angle (Table 17)
N = Normal force = W * cos[b].

table 23 Values of friction angles and unit weights for various soils. (from Burroughs, et. al., 1976)

Soil type	Density	Friction Angle Degrees	Unit Soil Weight Tonnes/m^3
Coarse Sand Gravel	Compact	45	2.24
	Firm	38	1.92
	Loose	32	1.44
Medium Sand	Compact	40	2.08
	Firm	34	1.76
	Loose	30	1.44
Fine Silty sands	Compact	32	2.08
	Firm	30	1.60
	Loose	28	1.36
Uniform Silts	Compact	30	1.76
	Loose	26	1.36
Clay- Silt	Medium	15 - 20	1.92
	Soft	15 - 20	1.44
Clay	Medium	0 - 10	1.92
	Soft	0 - 10	1.44

The friction angle is also referred to as the angle of repose. Sand or gravel cannot be used to form a steeper slope than the frictional angle allows. In other words, the maximum fill angle of a soil cannot exceed its coefficient of friction. Typical friction angles are given in Table 17. One should note the change in soil strength from "loose" to "compact" indicating the improvement in cohesion brought about by proper soil compaction.

Cohesionless soils such as sands or gravel without fines (clay) derive their strength from frictional resistance only

T = W * cos[b] * tan[f]

while pure clays derive their shear strength from cohesion or stickiness. Shear strength or cohesive strength of clay decreases with increasing moisture making clays very moisture sensitive.

The factor of safety against sliding or failure can be expressed as:

FS = {C * A + (W * cos[b] * tan [f]) } / { W * sin[b] }

In cases where the cohesive strength approaches zero (granular soils, high moisture content) the factor of safety simplifies to

$$FS = \tan[f] / \tan[b] \qquad (C = 0)$$

Road fills are usually built under dry conditions. Soil strength, particularly, cohesive strength is high under such conditions. If not planned or controlled, side cast fills are often built at the maximum slope angle the fill slope will stand (angle of repose). The fill slope, hence, has a factor of safety of one or just slightly larger than one. Any change in conditions, such as added weight on the fill or moisture increase, will lower the factor of safety, and the fill slope will fail. It is clear that the factor of safety must be calculated from "worst case" conditions and not from conditions present at the time of construction.

Failure can be brought about in one of two ways:

1. <u>Translational fill failure</u> (Figure 42) can be brought about by a build-up of a saturated zone. Frictional strength or grain-to-grain contact is reduced by a bouyancy force. Rainfall and/or ponded ditch water seeping into the fill are often responsible for this type of failure.

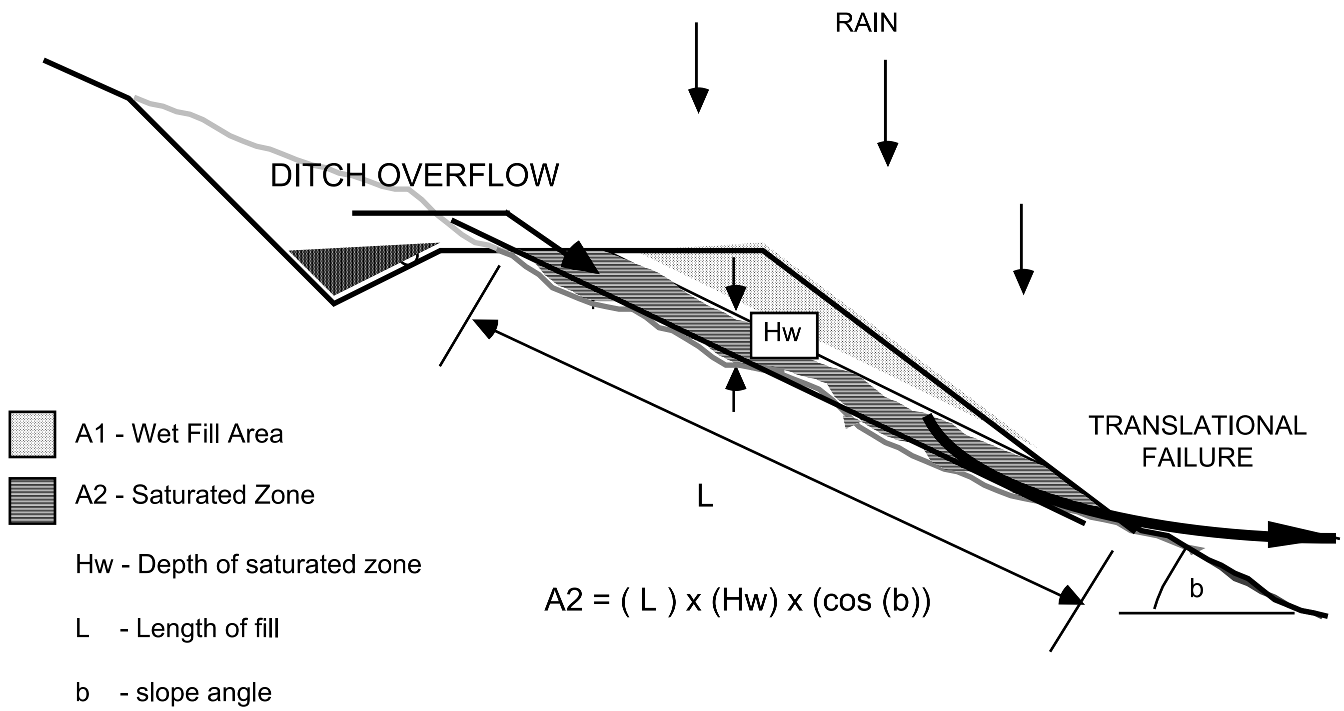


Figure 42. Translational or wedge failure brought about by saturated zone in fill. Ditch overflow or unprotected surfaces are often responsible.

The factor of safety against a translational failure can be shown to be:

$$FS = \{ [C^* A_1 + g_{buoy} * A_2] * \tan[f] \} / \{ [g^* A_1 + g_{sat} * A_2] * \tan[b] \}$$

where

g	= Wet or moist fill density
g_{sat}	= Saturated fill density
g_{bouy}	= $g_{sat} - g_{water}$ ($g_{water} = 1$)
A_1	= Cross sectional area of unsaturated fill
A_2	= Cross sectional area of saturated fill

2. Rotational or Slump fill failure brought about by seepage at the toe of the fill (Figure 43). The subsequent backward erosion of unprotected fill toes will result in a vertical face or bank prone to slumping. Eventually it will trigger a complete fill failure.

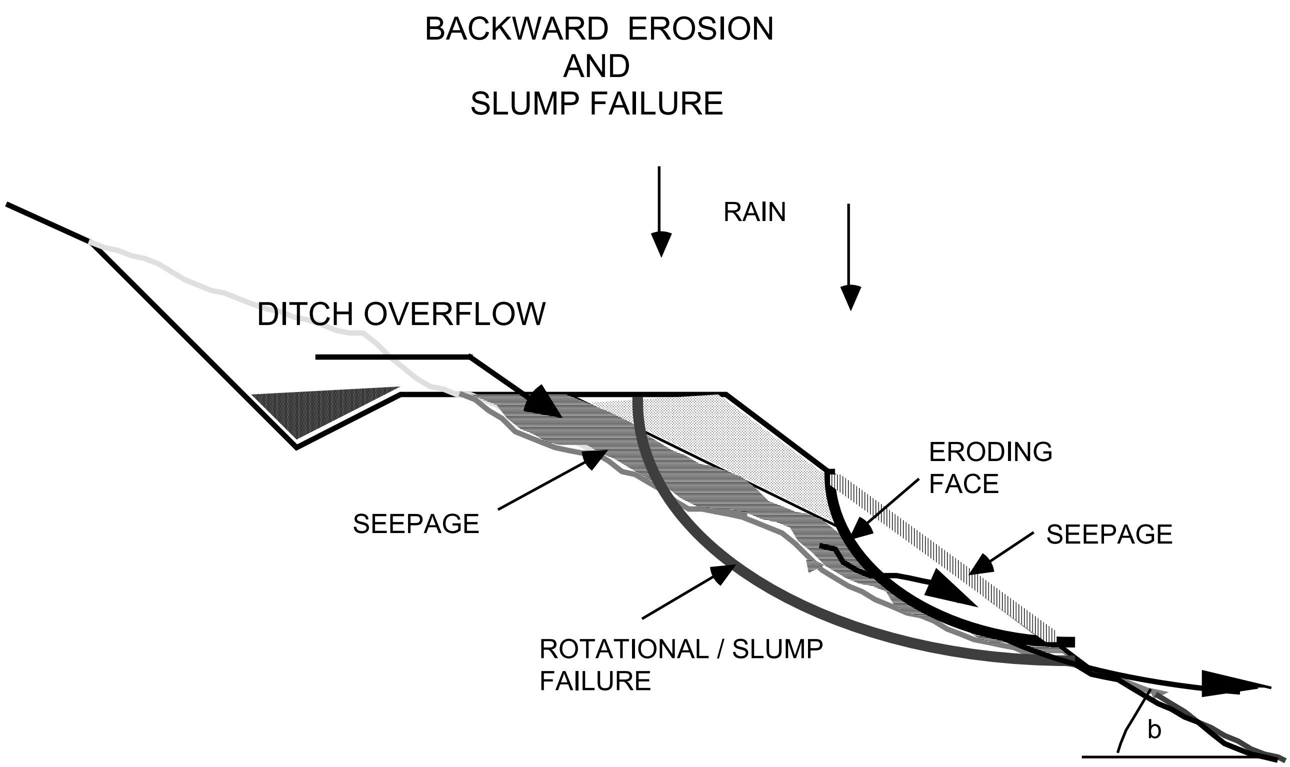


Figure 43. Fill failure caused by backward erosion at the toe of the fill due to excessive seepage and an unprotected toe.

Stability analysis can help in the determination and selection of proper road prism. Fill slope angle for common earth (a mixture of fragmented rock and soil) should typically not exceed 33.6^{o} which corresponds to a rise:run ratio of 1:1.5. Therefore, a road prism on side slopes steeper than 50 - 55% (26 - 29^{o}) should be built as "full-benched" because of the marginal stability of the fill section. Fill sections on steep side slopes can be used, if the toe of the fill is secured through cribbing or a rock wall which allows a fill slope angle of 33.6^{o} (1:1.5).

Practical considerations suggest that fill slope angle and ground slope angle should differ by at least 7^{o}. Smaller angles result in so-called "sliver-fills" which are difficult to construct and erode easily.

Example: Ground slope = 50%
Fill slope = 66.7%

Assuming zero cohesion and friction angle equals fill angle ([f] = [b])

FS = tan[f] / tan[b] = 0.667 / 0.50 = 1.33

The factor of safety is adequate. The fill slope stability becomes marginal if the same road prism (fill slope angle = 33.7^{o}) is built on a 60% side slope. The factor of safety becomes

FS = 0.667/0.600 = 1.11

The factor of safety in this case would be considered marginal. Here the difference between fill slope angle and ground slope is less than 7^{o}, a sliver fill.

Cut slope failures in road construction typically occur as a rotational failure. It is common in these cases to assume a circular slip surface. Rotational failures can be analyzed by the method of slices, probably the most common method for analyzing this type of failure (Bishop,1950; Burroughs, et. al.,1976).

Numerous stability charts have been developed for determining the critical height of a cut for a specific soil characterized by cohesion, friction angle, and soil density. The critical height, H_{crit}, is the maximum height at which a slope will remain stable. They are related to a stability number, N_s, defined as

$$N_s = H_{crit} (C/[g]).$$

Chen and Giger (1971) and Prellwitz (1975) published slope stability charts for the design of cut and fill slopes. Cut/fill slope and height recommendations in Section 3.2.3 are based on their work.

3.2.2 Side Cast - Full Bench Road Prism

Proper road design includes the selection of the appropriate road template as well as minimal earthwork by balancing the cuts and fills as shown in Figure 44.

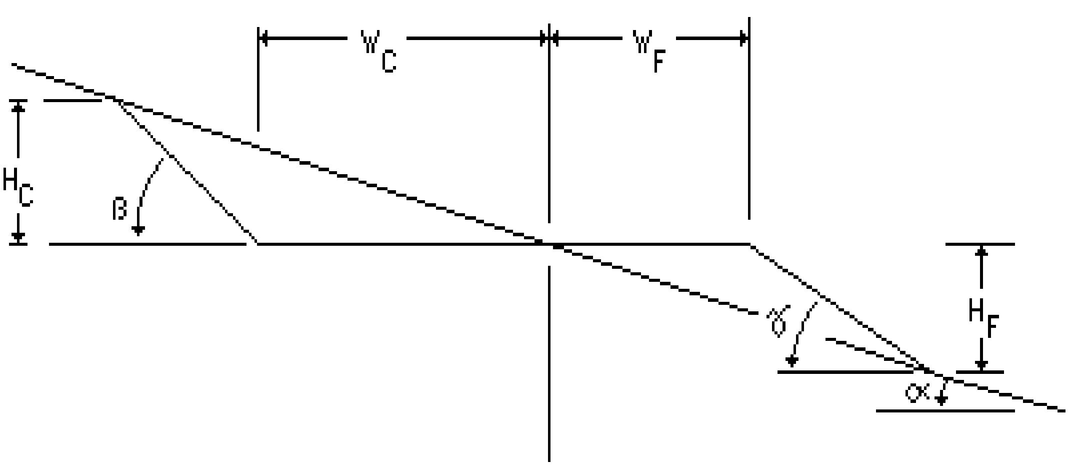


Figure 44. Elements of road prism geometry

The volume of cut and fill per meter of road can be calculated by the following formula:

$$\text{Volume of cut/meter} = W_C^{\,2} / (\cot\alpha - \cot\beta)$$

$$\text{Volume of fill/meter} = W_F^{\,2} / (\cot\alpha - \cot\gamma)$$

where

W_C = width of cut measured from grade-out point to hinge point
H_C = height of cut
W_F = width of fill, measured from grade-out point to shoulder
H_F = height of fill
α = angle of side slope
β = angle of cut slope
γ = angle of fill slope

For earthwork calculations, the required fill equals the cut, minus any loss from shrinkage, plus any gain from swell (rock).

Fill slopes can be constructed up to a maximum slope angle of 36° to 38°. Common practice is to restrict fill slopes to 34°. This corresponds to a ratio of 1 : 1.5 (run over rise). The maximum fill slope angle is a function of the shear strength of the soil, specifically the internal angle of friction. For most material, the internal angle of friction is approximately 36° to 38°.

Compacted side cast fills that must support part of the road become more difficult to construct with increasing side slopes. Sliver fills, as described in Section 3.2.1, result from trying to construct fills on steep side slopes. For side slopes in excess of 25° to 27° (50 to 55 %), the full road width should be moved into the hillside. Excavated material can be side cast or wasted, but should not form part of the roadbed or subgrade for the reasons discussed in Chapter 3.2.1.

The volume of excavation required for side cast construction varies significantly with slope. On side slopes less than 25° to 30° (50 to 60%) the volume of excavation for side cast construction is considerably less than the volume of excavation for full bench construction. However, as the side slope angle approaches 75% (37°), the volume of excavation per unit length of road for side cast construction approaches that required for full bench construction. Side cast fills, however, cannot be expected to remain stable on slopes greater than 75%.

This relationship of excavation volume for side cast and full bench construction is shown in Figure 45. The subgrade width is 6.6 meter, the fill angle is 37°, and a bulking factor of 1.35 is assumed (expansion due to fragmentation or excavation of rock).

A similar graph can be reconstructed by the following equation:

$$W_C = [-b \pm (b^2 - 4 * a * c)^{1/2}] / 2 * a$$

$$a = [(\cot\alpha - \cot\gamma)/K] - (\cot\alpha - \cot\beta)$$

$$b = 2 * W * (\cot\alpha - \cot\beta)$$

$$c = -W^2 (\cot\alpha - \cot\beta)$$

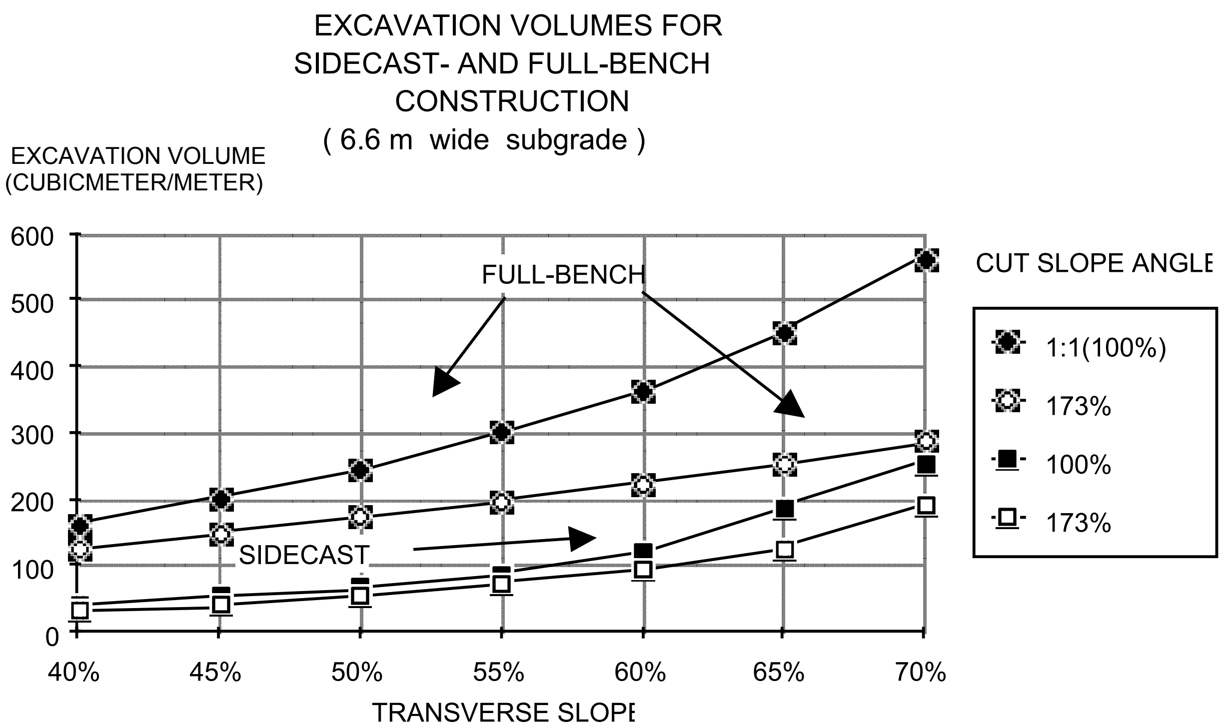


Figure 45. Required excavation volumes for side cast and full bench construction as function of side slope. Assumed subgrade width 6.6 m and bulking factor K = 1.35 (rock).

where W = total subgrade width

= $W_C + W_F$

K = bulking or compaction factor

(for rock, K = 1.3 - 1.4; for common earth compacted fills, K = 0.7 - 0.8. Other symbols are defined earlier in this section.)

The effect of careful template selection on overall width of disturbed area becomes more important with increasing side slope. Material side cast or "wasted" on side slopes steeper than 70 to 75% will continuously erode since the side slope angle exceeds the internal angle of friction of the material. The result will be continuous erosion and ravelling of the side cast material.

Another factor contributing to the instability of steeply sloping fills is the difficulty in revegetating bare soil surfaces. Because of the nature of the side cast material (mostly coarse textured, infertile soils) and the tendency for surface erosion on slopes greater than 70%, it is very difficult to establish a permanent protective cover. From that perspective, full bench construction combined with end haul of excavated material (removing wasted material to a safe area) will provide a significantly more stable road prism.

The relationship between erodible area per kilometer of road surface increases dramatically with increasing side slope where the excavated material is side cast (Figure 46). The affected area (erodible area), however, changes very little with increasing side slopes for full bench construction combined with end haul (Figure 47). The differences in affected area between the two construction methods are dramatic for side slopes exceeding 60%.

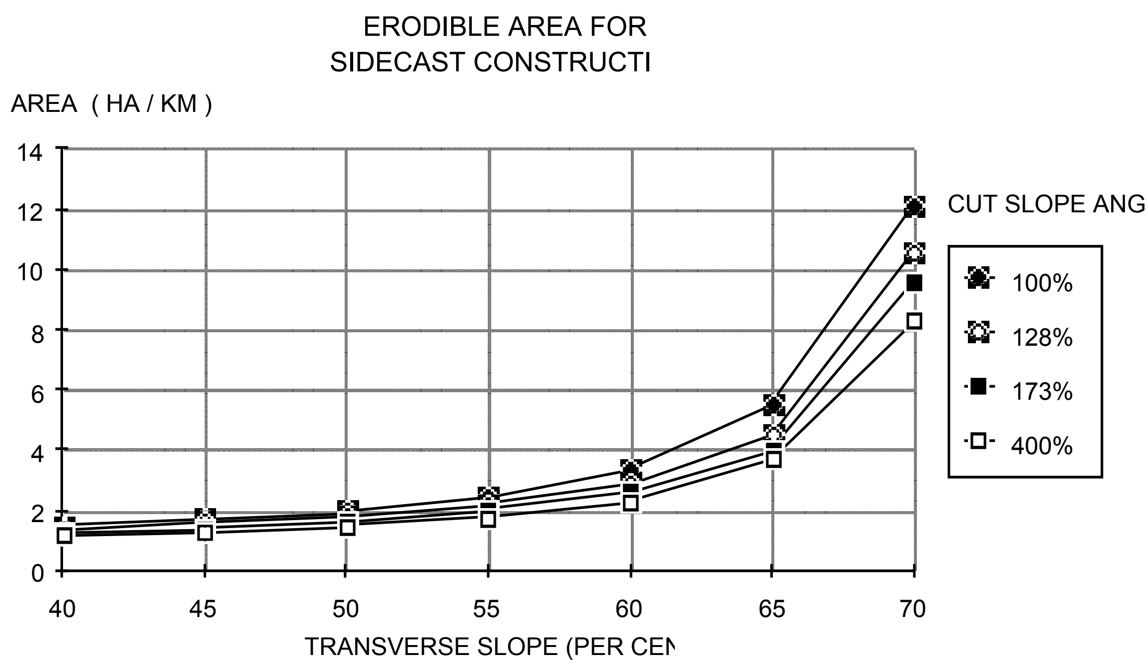


Figure 46. Erodible area per kilometer of road for side cast construction as a function of side slope angle and cut slope angle. The values shown are calculated for a 6.6 m wide subgrade. The fill angle equals 37°.

For example, the difference in affected area is over 8.8 km^2 per kilometer of road as the side slope angle approaches 65%. Also, as slope angle increases, the erosive power of flowing water increases exponentially. Obviously, careful consideration must be given when choosing between side cast construction and full bench construction with end haul.

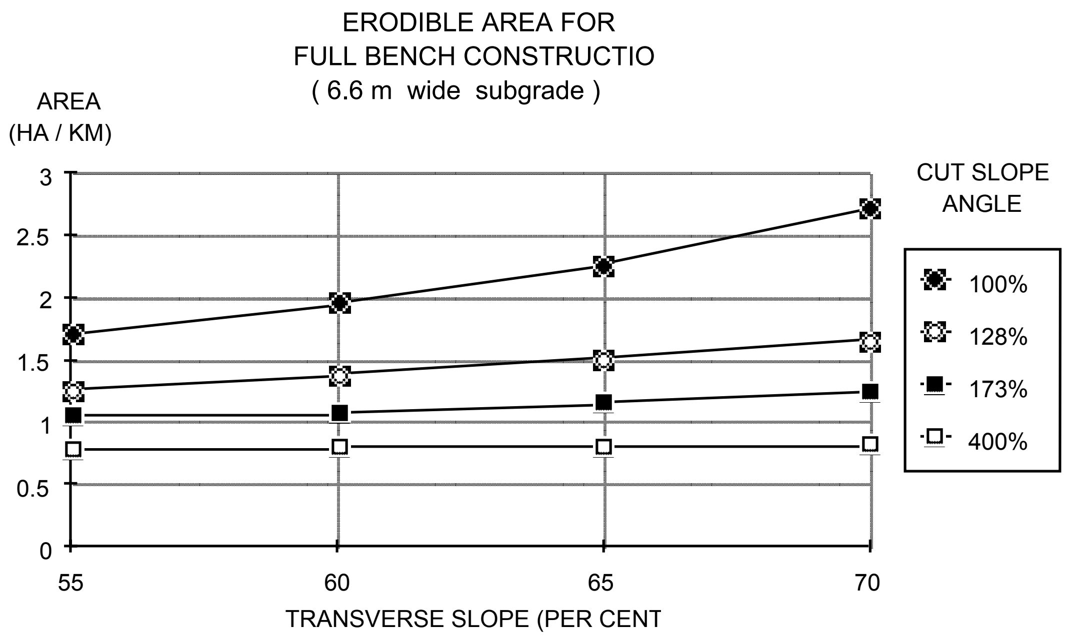


Figure 47. Erodible area per kilometer of road for full bench/end haul construction as a function of side slope angle and cut slope angle. The values shown are calculated for a 6.6 m wide subgrade..

3.2.3 Slope design

The U. S. Forest Service has developed guidelines for determining general values for maximum excavation and embankment slope ratios based on a combination of general field descriptions and the Unified Soil Classification of the material. Water table characteristics along with standard penetration and in-place density test values can further define the nature of the materials. Published information sources describing soils, geology, hydrology, and climate of the area should be carefully reviewed since certain of these reports often contain specific information relating to the engineering properties of materials in the area. These will also assist in the detailed characterization of soils, geologic, and bedrock conditions along the entire cross section of cut and/or fill area.

In general, the higher the cut or fill the more critical the need becomes for accurate investigation. The following consists of special limitations with regard to height of the cut or fill and the level of investigation required to adequately describe the entire cross section.

<u>0 to 15 meters (0 to 50 feet)</u> in vertical height requires a minimum of investigation for non-critical areas.The investigation would include soil classification, some hand or backhoe excavation, seismic data, and observations of nearby slopes to determine profile horizonation and relative stability.

15 to 30 meters (50 to 100 feet) in vertical height requires a more extensive investigation including all the items listed above plus test borings, either by hand auger or drill holes to identify soil horizons and the location of intermittent or seasonal water tables within the profile.

Over 30 meters (over 100 feet) in vertical height will require a slope designed by a specialist trained in soil mechanics or geological engineering. Under no circumstances should the following guides be used for slopes in excess of 30 meters in vertical height.

Special investigation may also be necessary when serious loss of property, extensive resource damage, or loss of life might result from a slope failure or when crossing areas where known instability exists or past slope failures have occurred. Soils containing excessive amounts of organic matter, swelling clays, layered schists or shales, talus, and pockets of loose water-bearing sands and silts may require special investigation as would fissured clay deposits or layered geologic strata in which subsurface conditions could not be determined for visual or seismic investigation.

The following list shows soil types and the pertinent design figures and tables for that soil:

SOIL TYPE (Unified)	TABLE / FIGURE
Coarse grained soils (≤ 50% passing #200 sieve)	
Sands and gravels with nonplastic fines (Plasticity Index ≤3); Unified Soil Classification: GW, GP, SW, SP, GM,and SM	Table 18
Sands and gravels with plastic fines (Plasticity Index > 3); Unified Soil Classification: GM, SM, respectively	Figures 48 & 49
Fine grained soils (> 50% passing #200 sieve)	
Unified Soil Classification: ML, MH, CL, AND CH slowly permeable layer at surface of cut and at some distance below cut, respectively	Figures 50 & 51
Unweathered rock	Table 19
Fill Slopes	Table 20

Curves generated in Figures 48 and 49 illustrating maximum cut slope angles for coarse grained soils are organized according to five soil types:

1. Well graded material with angular granular particles; extremely dense with fines that cannot be molded by hand when moist; difficult or impossible to dig with shovel; penetration test blow count greater than 40 blows per decimeter.

2. Poorly graded material with rounded or low percentage of angular granular particles; dense and compact with fines that are difficult to mold by hand when moist; difficult to dig with shovel; penetration test approximately 30 blows per decimeter.

3. Fairly well graded material with subangular granular particles; intermediate density and compactness with fines that can be easily molded by hand when moist (Plasticity Index > 10); easy to dig with shovel; penetration test blow count approximately 20 blows per decimeter.

4. Well graded material with angular granular particles; loose to intermediate density; fines have low plasticity (Plasticity Index < 10); easy to dig with shovel; penetration test blow count less than 10 blows per decimeter.

5. Poorly graded material with rounded or low percentage of granular material; loose density; fines have low plasticity (Plasticity Index < 10); can be dug with hands; penetration count less than 5 blows per decimeter.

Curves generated in Figures 50 and 51 illustrating maximum cut slope angles for fine grained soils are organized according to five soil types based on consistency. Complete saturation with no drainage during construction is assumed making the depth to a slowly permeable underlying layer such as bedrock or unweathered residual material the single most important variable to consider. Figure 50 assumes the critical depth to be at or above the bottom of the cut; Figure 51 assumes the critical depth to be at a depth three times the depth of excavation as measured from the bottom of the cut. Cut slope values for intermediate depths can be interpolated between the two charts:

1. Very stiff consistency; soil can be dented by strong pressure of fingers; ripping may be necessary during construction; penetration test blow count greater than 25 blows per decimeter.

2. Stiff consistency; soil can be dented by strong pressure of fingers; might be removed by digging with shovel; penetration test blow count approximately 20 blows per decimeter.

3. Firm consistency; soil can be molded by strong pressure of fingers; penetration test blow count approximately 10 blows per decimeter.

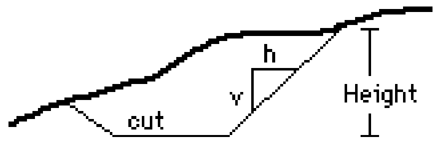
4. Soft consistency; soil can easily be molded by fingers; penetration test blow count approximately 5 blows per decimeter.

5. Very soft consistency; soil squeezes between fingers when fist is closed; penetration test blow count less than 2 blow s per decimeter

Fill slopes typically display weaker shear strengths than cut slopes since the soil has been excavated and moved from its original position. However, fill strengths can be defined with a reasonable degree of certainty, provided fills are placed with moisture and density control. The slope angle or angle of repose is a function of the internal angle of friction and cohesive strength of the soil material. Table 20 provides a recommended maximum fill slope ratio as a function of soil type, moisture content, and degree of compaction. Slopes and fills adjacent to culvert inlets may periodically become subjected to inundation when ponding occurs upstream of the inlet.

Compaction control, as discussed previously, is achieved through the manipulation of moisture and density and is defined by the standard Proctor compaction test (AASHTO 90). If no compaction control is obtained, fill slopes should be reduced by 25 percent.

table 24 Maximum cut slope ratio for coarse grained soils. (USFS, 1973)



Soil Type	Maximum Cut Slope Ratio (h:v)			
	Low groundwater (below bottom of excavation)		High groundwater 1/ (seepage from entire slope)	
	loose 2/	dense 3/	loose	dense
GW, GP	1.5 : 1	.85 : 1	3 : 1	1.75 : 1
SW	1.6 : 1	1 : 1	3.2 : 1	2 : 1
GM, SP, SM	2 : 1	1.5 : 1	4 : 1	3 : 1

1/ Based on material of saturated density approximately 19.6 kN/m^3. Flatter slopes should be used for lower density material and steeper slopes can be used for higher density material. For every 5 % change in density, change the ratio by approximately 5%.

2/ Approximately 85% of maximum density.

3/ Approximately 100% of maximum density.

table 25 Maximum cut slope ratio for bedrock excavation (USFS, 1973)

Rock type	Maximum Cut Slope Ratio Massive	Maximum Cut Slope Ratio Fractured
Igneous (granite, trap, basalt, and volcanic tuff)	0.25:1	0.50:1
Sedimentary (massive sandstone and limestone;	0.25:1	0.50:1
interbedded sandstone, shale, and limestone;	0.50:1	0.75:1
massive claystone and siltstone)	0.75:1	1:1
Metamorphic (gneiss, schist, and marble;	0.25:1	0.50:1
slate;	0.50:1	0.75:1
(serpentine)	Special investigation	

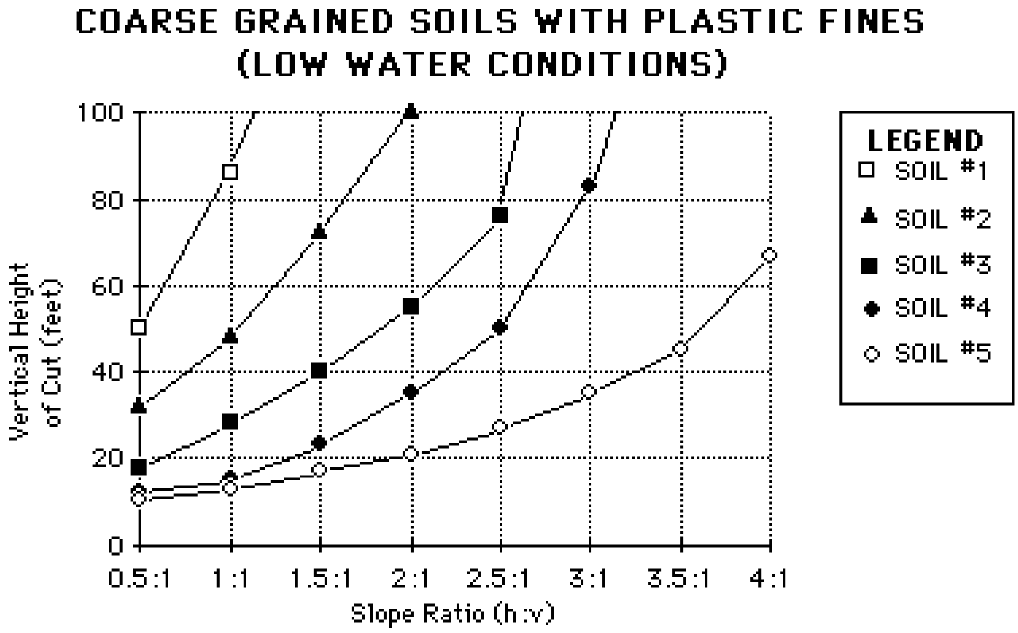


Figure 48. Maximum cut slope ratio for coarse grained soils with plastic fines (low water conditions). Each curve indicates the maximum height or the steepest slope that can be used for the given soil type. (After USFS,1973)

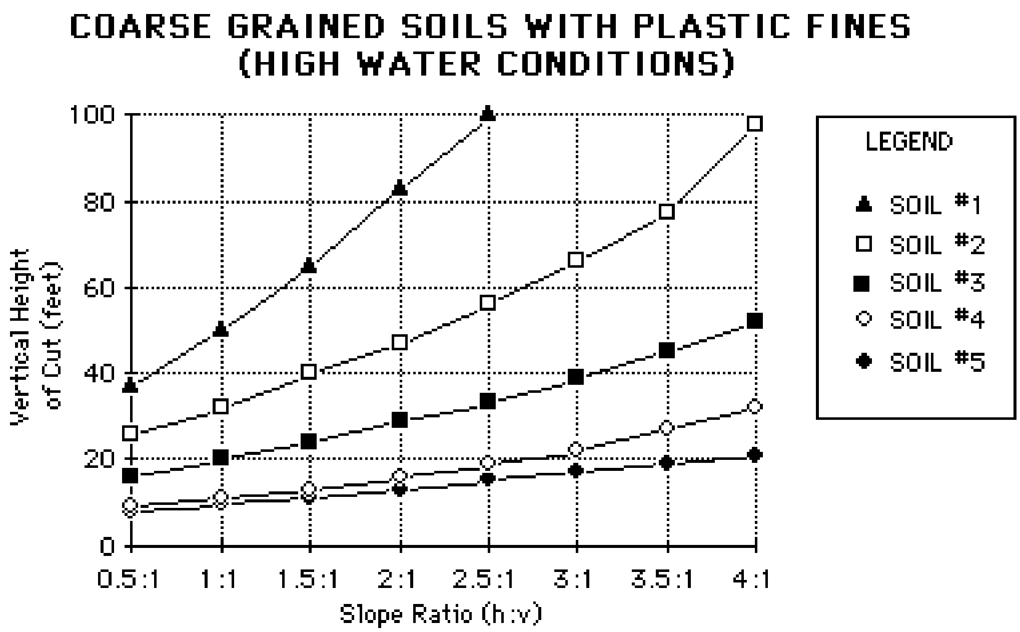


Figure 49. Maximum cut slope angle for coarse grained soils with plastic fines (high water conditions). Each curve indicates the maximum cut height or the steepest slopes that can be used for the given soil type. (After USFS,1973)

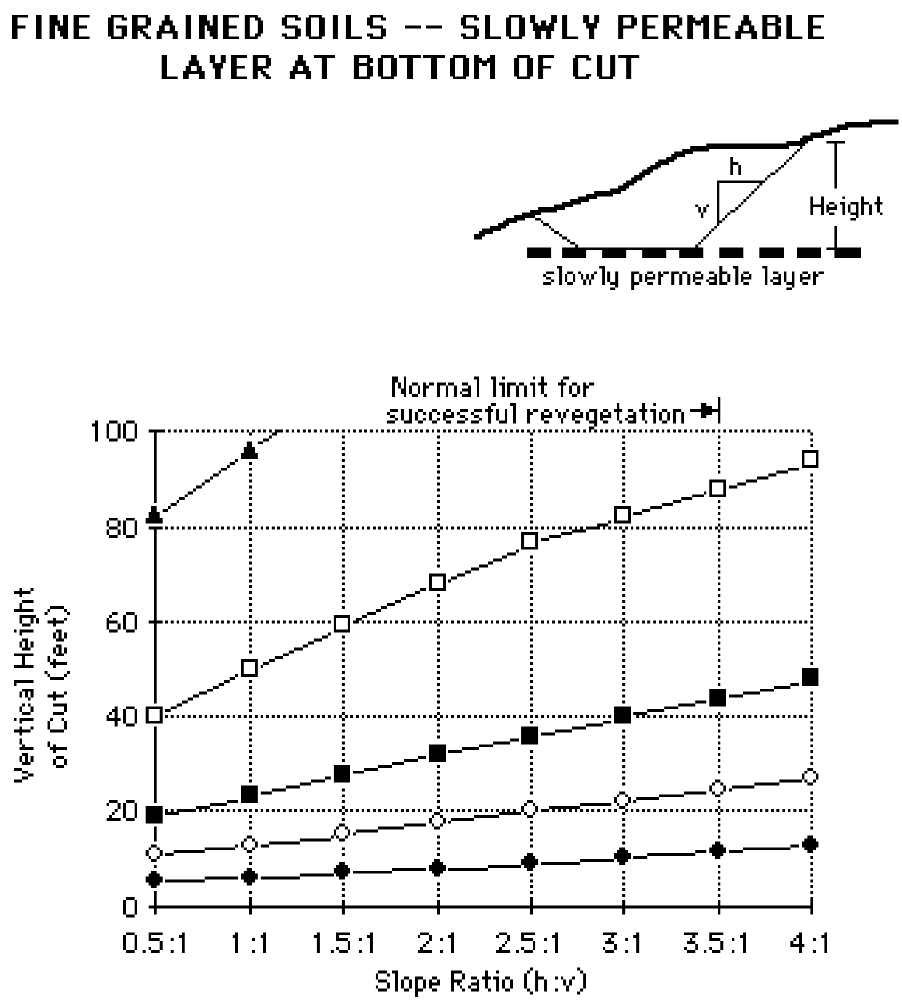
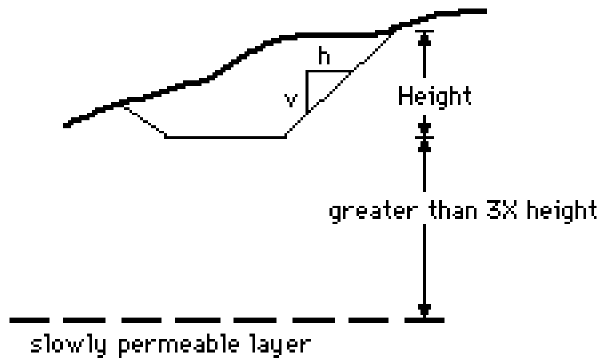


Figure 50. Maximum cut slope angle for fine grained soils with slowly permeable layer at bottom of cut. Each curve indicates the maximum vertical cut height or the steepest slope that can be used for the given soil type. (After USFS 1973)

FINE GRAINED SOILS WITH SLOWLY PERMEABLE LAYER BELOW CUT (GREATER THAN 3X HEIGHT OF CUT)



Soil Type Ratio (h:v)	Maximum Height (m)[1/]	Slope
1	24	0.5:1
2 [2/]	12	0.5:1
3 [2/]	6	0.5:1
4 [3/]	3	1:1
5 [3/]	1.5	1:1

[1/] If it is necessary to exceed this height consult with geologic or materials engineer.Benching will not improve stability as stability is nearly independent of slope ratio on these slopes.

[2/] If the slope of the natural ground exceeds 20° (36 percent), then the natural slope may be unstable. A detailed field investigation is necessary to check this condition prior to design or construction phases.

[3/] If the slope of the natural ground exceeds 10° (18 percent), then the natural slope may be unstable. A detailed field investigation is necessary to check this condition prior to design or construction phases.

Figure 51. Maximum cut slope angle for fine grained soils with slowly permeable layer at great depth (≥ 3 times height of cut) below cut. (After USFS, 1973).

table 26 Minimum fill slope ratio for compacted fills. (US Forest Service, 1973)

Soil type to inundation	Slope not subject to inundation	Slope subject compaction	Minimum percent
Hard, angular rock, blasted or ripped	1.2:1	1.5:1	--
GW	1.3:1	1.8:1	90[1]
GP, SW	1.5:1	2:1	90[1]
GM, GC, SP	1.8:1	3:1	90[1]
SM, SC[2]	Figure 48, Soil 3	Figure 49, Soil 3	90
Figure 48, Soil 4	Figure 49, Soil 4	no control	
ML, CL[2]	Figure 48, Soil 4	Figure 49, Soil 4	90
Figure 48, Soil 5	Figure 49, Soil 4	no control	
MH, OH[2]	Figure 50, Soil 3	Figure 50, Soil 4	90
Figure 50, Soil 4	Figure 50, Soil 5	no control	

[1] With no compaction control flatten slope by 25 percent.

[2] Do not use any slope steeper than 1.5:1 for these soil types.

3.2.4 Road Prism Selection

In the planning stage (Chapter 2) basic questions such as road uses, traffic volume requirements and road standards have been decided. The road standard selected in the planning stage defines the required travel width of the road surface. The road design process uses the travel width as a departure point from which the necessary subgrade width is derived. The road design process which deals with fitting a road template into the topography uses the subgrade width for cut and fill calculations. Therefore, ditch and ballast requirements need to be defined for a given road segment in order to arrive at the proper subgrade width or template to be used.

<u>Example</u> (see also Figure 52.): Travelled road width is established at 3.0 meters. Ballast material is pit-run rock. Shoulder slope of ballast is 2:1. Soil and traffic characteristics require 0.45 m layer of ballast. The ditch line is to be 0.30 m deep with slopes of 1:1 and 2:1. Fill widening of 0,6 m is added because of fill slope height.

Total subgrade width is therefore:

3.0 m	traveled width
+ (2.9 m)	shoulder
+ (0.9 m)	ditch line
+ (0.6 m)	fill widening
= 6.3 m	total subgrade width

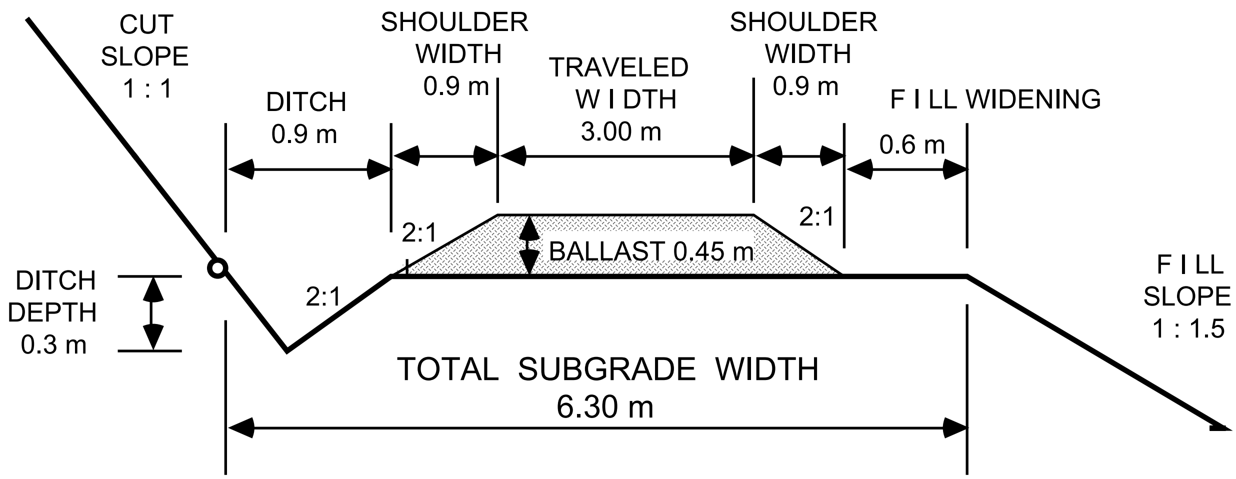


Figure 52. Interaction of subgrade dimension, roadwidth, ballast depth, ditch width and fill widening

Table 21 lists various subgrade width for a 3.00 m traveled road width and different ballast depth requirements.

table 27 Required subgrade width (exclusive fo fill widening) as a function of road width, ballast depth and ditch width. Roadwidth = 3.0 m, ditch = 0.9 m (1:1 and 2:1 slopes), shoulder-slopes 2:1.

Ballast Depth	Subgrade Width Ditch	Subgrade and Ditch	Through-cut on both Sides
	- - - - - - - - - - - - - - - - - - meters	- - - - - - - - - - - - - - - -	
0.30	4.2	5.1	6.0
0.45	4.8	5.7	6.0
0.60	5.4	6.3	7.2
0.75	6.0	6.9	7.8

Fill widening is another factor which modifies the subgrade or template width independent of traveled road width or ballast depth. Fill widening should be considered in cases where fills cannot be compacted with proper equipment and where no compaction control is performed. In such cases fill widening of 0.30 m are recommended where fill slope height is less than 2.00 m. Fill slope height in excess of 2.00 m should have 0,60 m of fill widening (see Figure 53). Fill slope height in excess of 6.00 m should be avoided altogether because of potential stability problems.

Fill slope height H_f =< 2 m add 0.30 m fill widening.
H_f => 2 m add 0.60 m.

Maximum Fill slope height H_f =< 6.00 m (unless engineered)

Cut slopes are inherently more stable than fill slopes. The road designer should try to minimize fill slope length by "pushing the alignment into the hill side in order to minimize erosion. Typically this will result in longer cut slopes and add slight to moderate cuts at the center line. The result will be a moderate fill slope (see Figure 54) with no additional fill widening required.

Toe walls are often a feasible alternative on steep side slopes to reduce excavation and avoid end hauling. Toe walls can be built of log cribs, gabions or large rocks (Figure 55). A proper base foundation is excavated at the toe of the fill on which the retaining wall is constructed. Approximately two-thirds of the subgrade would be projected into the hill side and one third would be supported by the fill resting on the retaining structure.The reduction in excavation material, exposed cut slope and avoided end haul is significant.

FILL WIDENING

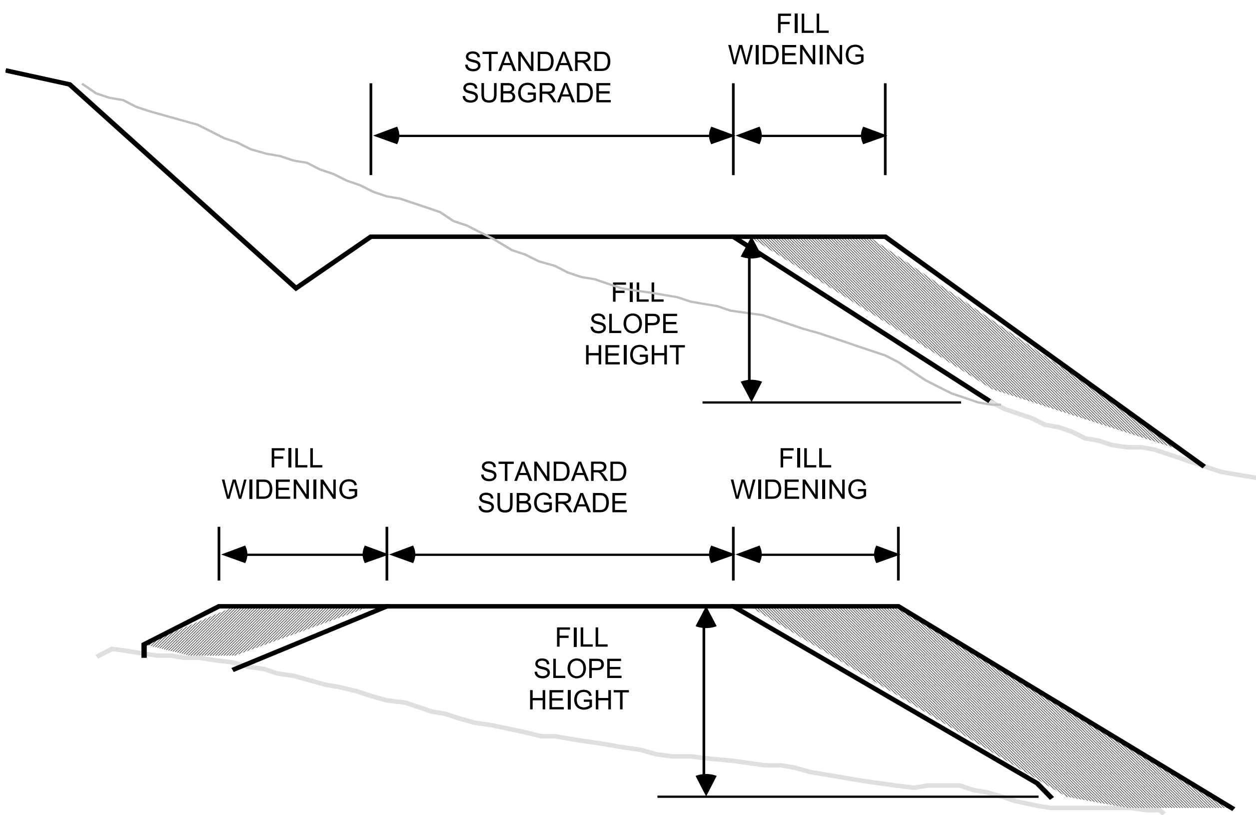


Figure 53. Fill widening added to standard subgrade width where fill height at centerline or shoulder exceeds a critical height. Especially important if sidecast construction instead of layer construction is used.

FILL SLOPE LENGTH REDUCTION

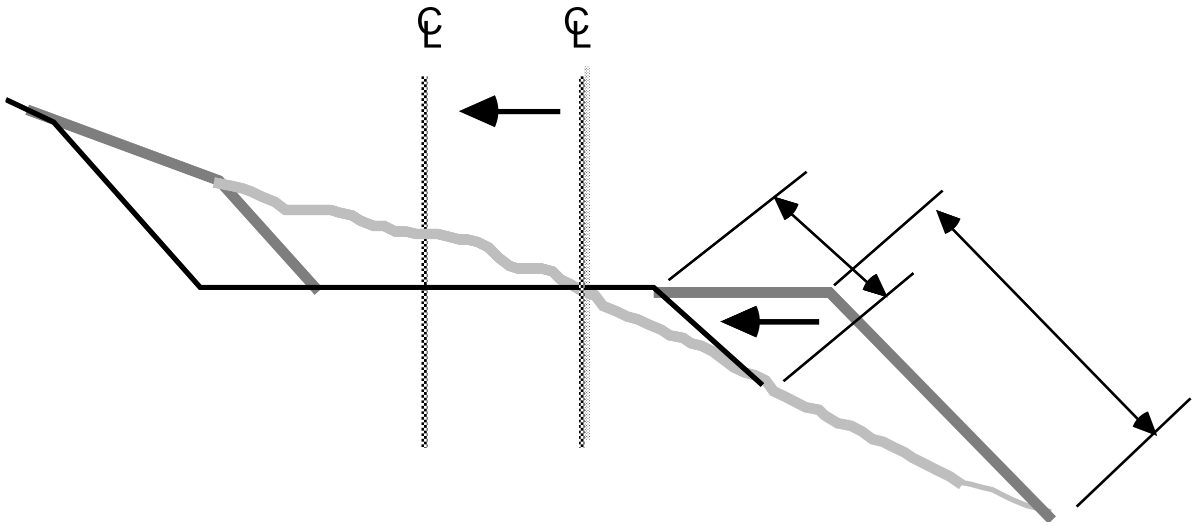


Figure 54. Template and general road alignment projected into the hill favoring light to moderate cuts at centerline in order to minimize fill slope length. Fill slopes are more succeptible to erosion and sloughing than cut slopes.

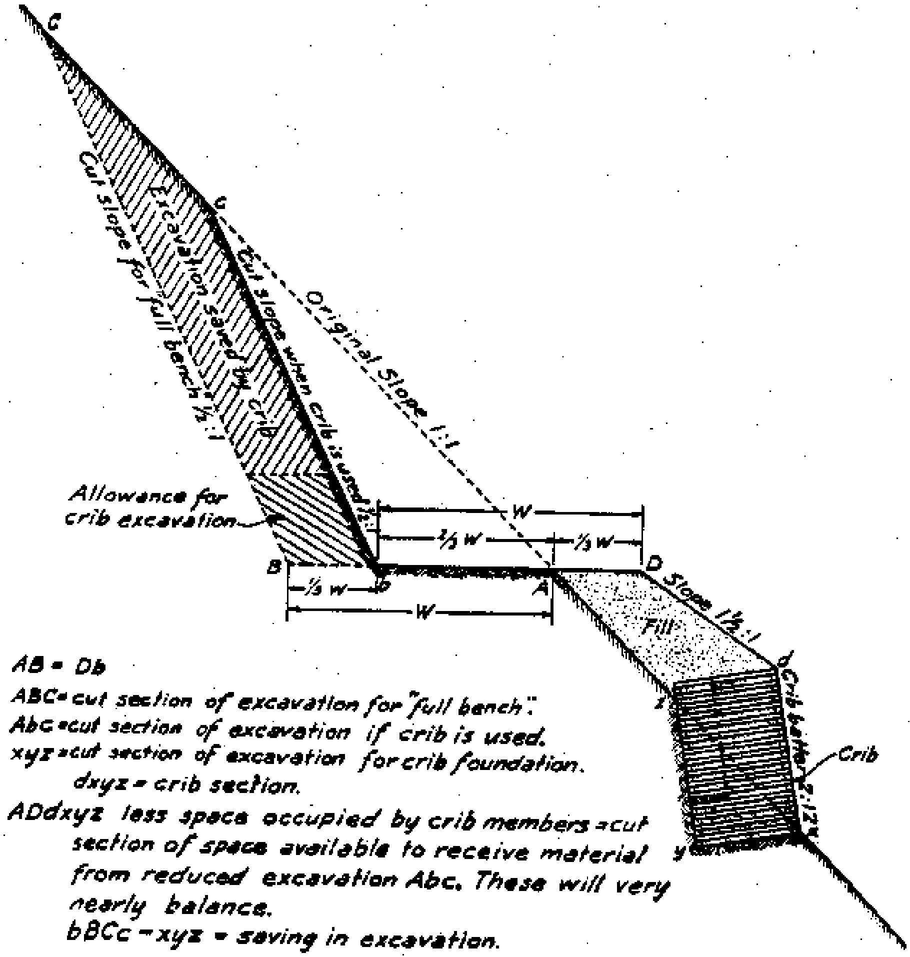


Figure 55. Illustration of the very considerable reduction in excavation made possible on a steep slope by the use of cribbing. Crib proportions shown are suitable for log construction; if crib was built of concrete or steel, shorter spreaders could be used in upper 3 m as indicated by the dashed line (Kraebel, 1936).

3.3 Road Surfacing

Properly designed road surfaces serve a dual purpose. First, they provide a durable surface on which traffic can pass smoothly and safely. If heavy all-season use is anticipated, the surface should be designed to withstand the additional wear. Second, the road surface must protect the subgrade by distributing surface loads to a unit pressure the subgrade can support, minimizing frost action, and providing good surface drainage. A crowned surface of 3 to 5 cm/m of half-width will ensure adequate movement of surface water and reduce the potential for subgrade saturation.

Improper road surfacing or ballasting affects water quality in two ways: 1) Surface material is ground up into fines that are easily eroded. It has been demonstrated that surface loss is related to traffic levels and time in addition to erosional forces. Larger gravels present in the road surface must be mechanically ground up by traffic before they can be acted upon by surface erosion processes (Armstrong, 1984).

Reid and Dunne (1984) have demonstrated the significance of traffic intensity in the mobilization of sediment in an area of the Pacific Northwest Region of the United States which receives an average annual rainfall of 3900 mm/yr (150 in/yr). The results from their study demonstrated that although road segments receiving "heavy" use accounted for only a small proportion of total road length in the basin study area (6 percent), 70 percent of the total amount of sediment generated from road surfaces could be attributed to those segments during periods of heavy use. Reid and Dunne found the sediment production for a paved and gravelled road to be 2.0 and 500 tonnes/km/year, respectively (Table 22).

table 28 Calculated sediment yield per kilometer of road for various road types and use levels (Reid, 1981; Reid and Dunne, 1984)*

Road Type	Average Sediment Yield tonnes/km/year
Heavy use (gravel)	500
Temporary non-use (gravel)	66
Moderate use (gravel)	42
Light use (gravel)	3.8
Paved, heavy use	2.0
Abandoned	0.51

* Road width 4 m, average grade 10%
6 culverts/km, annual precipitation 3900 mm/year

Heavy use consisted of 4 to 16, 30 tonne log-trucks per day. Temporary non-use occurred over weekends with no log-truck traffic but occassional light vehicles. Light use was restricted to light vehicles (less than 4 tonnes GVW).

A road surface in its simplest form consists of a smoothed surface, in effect the subgrade. Dirt roads would fall into this category. Obviously, dirt roads are only useful where the road is expected to receive intermittent, light use and is not affected by climate. Sediment production from dirt road surfaces is high. Significant erosion rate reductions can be achieved by applying a rock or ballast layer. Even a minimal rock surface of 5 to 10 cm effectively reduces erosion and sediment yield by a factor of 9. Kochenderfer and Helvey (1984) documented soil loss reduction from 121 down to 14 tonnes per hectare per year by applying a 7.5cm rock surface on a dirt road.

Inadequate ballast or rock layers will not provide wheel load support appropriate for the subgrade strength except in cases where the subgrade consists of heavily consolidated materials. As a result, the ballast material is pushed into the subsoil and ruts begin to form. Ruts prevent effective transverse drainage, and fine soil particles are brought to the surface where they become available for water transport. Water is channeled in the ruts and obtains velocities sufficient for effective sediment transport. Sediment yield from rutted surfaces is about twice that of unrutted road surfaces (Burroughs et. al., 1984).

An improvement over a simple dirt road consists of a ballast layer over a subgrade, with or without a wearing course. The function of the ballast layer is to distribute the wheel load to pressures the subsoil can withstand. The wearing course provides a smooth running surface and also seals the surface to protect the subgrade from surface water infiltration. The wearing course can either be a crushed gravel layer with fines or a bitumenous layer.

Required ballast depth or thickness not only depends on subgrade strength but also on vehicle weight and traffic volume. The time or service life a road can support traffic without undue sediment delivery depends on:

- soil strength
- ballast depth
- traffic volume (number of axles)
- vehicle weight (axle load)

Surface loading from wheel pressure is transmitted through the surface to the subgrade in the form of a frustrum of a cone. Thus, the unit pressure on the subgrade decreases with increasing thickness of the pavement structure. Average unit pressure across the entire width of subgrade for any wheel load configuration can be calculated from the following formula:

$$P = L / \pi(r+d)^2$$

Where: P = unit pressure (kN/cm^2)
L = wheel load (kN)
r = radius of circle, equal in area to tire contact area (cm)
d = depth of pavement structure (cm)

A useful parameter for determining the strength of subgrade material is the California Bearing Ratio (CBR). CBR values are indices of soil strength and swelling potential. They represent the ratio of the resistance of a compacted soil to penetration by a test piston to penetration resistance of a "standard material", usually compacted, crushed rock (Atkins, 1980). The range of CBR values for natural soils is listed in Table 23 together with their suitability as subgrade material. Poorer subgrade material requires a thicker ballast layer to withstand traffic load and volume.

Factors other than CBR values must be considered when determining the thickness of the pavement structure. Subgrade compaction will depend upon construction methods used and the control of moisture during compaction. Subgrade drainage effectiveness, frost penetration and frost heave, and subgrade soil swell pressure are associated with water content in the soil and will also affect final design thickness. To counteract these factors, a thicker, heavier pavement structure should be designed.

table 29 Engineering characteristics of soil groups for road construction (Pearce, 1960).

Unified Soil Classification	Suitability Subgrade**	Field CBR Value	Dry weight* (kN/m3)	Frost Action
GW	60-80	19.6-22	none to very slight	excellent
GP	25-60	17.3-20.4	none to very slight	good to excellent
GM-d***	40-80	20.4-22.8 medium	slight to	good to excellent
GM-u***	20-40	18.9-22	slight to medium	good
GC	20-40	18.9-22	slight to medium	good
SW	20-40	17.3-20.4	none to very slight	good
SP	10-25	15.7-18.9	none to very slight	fair
SM-d***	20-40	18.9-21.2	slight to high	good
SM-u***	10-20	16.5-20.4	slight to high	fair to good
SC	10-20	16.5-20.4	slight to high	fair to good
ML	5-15	15.7-19.6	medium to very high	fair to poor
CL	5-15	15.7-19.6	medium to high	fair to poor
OL	4-8	14.1-16.5	medium to high	poor
MH	4-8	12.6-15.7	medium to very high	poor
CH	3-5	14.1-17.3	medium	poor to very poor
OH	3-5	12.6-16.5	medium	poor to very poor
PT			slight	unsuitable

* Unit dry weight for compacted soil at optimum moisture content for modified AASTO compactive effort.
** Value as subgrade, foundation or base course (except under bituminous) when not subject to frost action.
*** "d" = liquid limit ≤ 28 and plasticity index < 6; "u" = liquid limit > 28.

An alternative to the cost of a heavier pavement structure is the use of geotextile fabrics. Fabrics have been found to be an economically acceptable alternative to conventional construction practices when dealing with less than desirable soil material. The U. S. Forest Service has successfully used fabrics as filters for surface drainage, as separatory features to prevent subgrade soil contamination of base layers, and as subgrade restraining layers for weak subgrades. A useful guide for the selection and utilization of fabrics in constructing and maintaining low volume roads is presented by Steward, et al. (1977). This report discusses the current knowledge regarding the use of fabrics in road construction and contains a wealth of information regarding physical properties and costs of several brands of fabric currently marketed in the United States and abroad.

Proper thickness design of ballast layer not only helps to reduce erosion but also reduces costs by requiring only so much rock as is actually required by traffic volume (number of axles) and vehicle weight (axle weight-wheel loads). The principle of thickness design is based on the system developed by AASHO (American Association of State Highway Officials) and adapted by Barenberg et al. (1975) to soft soils. Barenberg developed a relationship between required ballast thickness and wheel or axle loads. Soil strength can be simply measured either with a cone penetrometer or vane shear device, such as a Torvane.

Thickness design for soft soil is based on the assumption of foundation shear failure where the bearing capacity of the soil is exceeded. For rapid loading, such as the passage of a wheel, the bearing capacity q is assumed to depend on cohesion only.

$$q = N_C * C$$

where q = Bearing capacity of a soil (kg/cm^2).
C = Cohesive strength of soil (kg/cm^2).
N_C = Dimensionless bearing capacity factor

Based on Barenberg's work, Steward, et al. (1977) proposed a value of 2.8 to 3.3 and 5.0 to 6.0 for N_C. The significance of the bearing capacity q is as follows:

A. q = 2.8 C is the stress level on the subgrade at which very little rutting will occur under heavy traffic (more than 1000 trips of 8,160 kg axle equivalencies) without fabric.

B. q = 3.3 C is the stress level at which heavy rutting will occur under light axle loadings (less than 100 trips of 8,160 kg axle equivalencies) without fabric.

C. q = 5.0 C is the stress level at which very little rutting would be expected to occur at high traffic volumes (more than 1000 trips of 8,160 kg equivalency axles) using fabric.

D. q = 6.0 C is the stress level at which heavy rutting will occur under light axle loadings (less than 100 trips of 8,160 kg axle equivalencies) using fabric.

(Heavy rutting is defined as ruts having a depth of 10 cm or greater. Very little rutting is defined as ruts having a depth of less than 5 cm extending into the subgrade.)

Charts relating soil strength (as measured with a vane shear device) to axle load and ballast thickness are shown in Figure 56 through 58. Figure 56 is based on a single axle, single wheel load. Figure 57 is based on a double wheel, single axle load, and Figure 58 is based on a tandem wheel configuration typical of 3 axle dump trucks or stinger type log-trucks.

It should be noted that axle and wheel configuration have a tremendous impact on the load bearing capacity of a road. The relationship between axle load and subgrade failure is not linear. Allowing 16,000 kg axle load vehicle to use a road designed for a standard axle load of 8,200 kg, is equivalent to 15 trips with the 8,200 kg axle load vehicle. Premature rut formation and its prevention depend on the selection of the proper axle load and strict enforcement of the selected load standard.

Some typical truck configurations, gross vehicle weights (GVW), and axle or wheel loads are given in Table 24. Vehicles under 3 tonnes GVW have no measurable effect on subgrade stress and deterioration.

Design Example

A road is to be constructed to access a watershed. Because of erosive conditions and traffic volumes, only 5 cm of rutting can be tolerated. Expected traffic volume is high (greater than 1,000 axle loads).

Three vehicle types are using the road:

1. Utility truck - 10 tonnes GVW; 4,500 kg single wheel load
   (9,000 kg axle load on rear axle, loaded)

2. Dump truck - 15 tonnes GVW with two axles;
   (11 tonnes rear axle load or 5.5 tonnes per dual wheel)

3. Log truck - 36 tonnes GVW with 5 axles, rear tandem axle load
   equals 15.9 tonnes or 7.95 tonnes per tandem wheel set.

Soil tests: Visually segment the road into logical construction segments based on soil type. Take soil strength measurements with vane shear device. Measurements should be taken during wet soil conditions, its weakest state. Take at least 10 vane shear readings at approximately 10 cm and 40 cm below the surface (in mineral soil). Tabulate readings in descending order from largest to smallest value. Your design shear strength is the 25th percentile shear strength--the value at which 75 percent of the soil strength readings are higher.

Vane Shear Readings

1. 0.58 kg/cm^2
2. 0.58
3. 0.50
4. 0.46
5. 0.45
6. 0.45
7. 0.40
8. 0.37
9. 0.36 => 0.36 (25 percentile) -Design strength to be 10.
0.32 used in calculation.
11. 0.32
12. 0.30

Subgrade strength for design purpose is taken as 0.36 kg/cm^2.

Ballast Depth Calculation: Calculate the soil stress value without fabric for little rutting (less than 5 cm for more than 1,000 axle loads).

$$q = 2.8 * 0.36 = 1.01 \ kg/cm^2$$

(Conversion factor: Multiply kg/cm^2 by 14.22 to get psi. This gives a value of 14.33 psi in this example).

In the case of the utility truck with 4,500 kg (10,000 lbs) wheel load, enter Figure 56 at 14.33 on the bottom line and read upwards to the 4,500 kg (10,000 lbs) single wheel load. A reading of 42 cm (16.5 in.) is obtained. Since 7 -12 cm additional ballast is needed to compensate for intrusion from the soft subgrade, a total of 49 - 54 cm of ballast is required. When fabric is used, a factor of 5.0 (little rutting for high traffic volumes) is applied to determine the ballast depth.

$$q = (5.0 * 0.36 * 14.22) + 25.6 \text{ psi}$$

A reading of 28 cm (11 in.) is obtained from Figure 56 indicating a saving of 21-26 cm of rock when fabric is used.

The same analysis is carried out for the other vehicles. Ballast depth required to support the other vehicles is shown in Table 24.

table 30 Required depth of ballast for three design vehicles. Road designed to withstand large traffic volumes (> 1,000 axle loads) with less than 5 cm of rutting.

Bearing Capacity q $C*N_C*14.22$	Utility truck 10 t GVW 4,500 kg (10,000 lbs) Single wheel	Dump truck 15 t GVW 5,500 kg (12,000 lbs) Dual wheel	Log truck 36 t GVW 7,850 kg (17,500 lbs) Tandem wheel	
	Ballast Thickness			
Without* Fabric 14.33 psi kg/cm2)	41+10=51 cm	42+10=52 cm	40+10=50 cm	(1.01
With ** Fabric 25.6 psi (1.80 kg/cm2)	31 cm	28 cm	24 cm	

*7 -12 cm additional rock is needed to compensate for contamination from the soft subgrade.
**The fabric separates the subgrade from the ballast. No intrusion of fines into subgrade.

The dump truck (15 t GVW) represents the most critical load and requires 52 cm of rock over the subgrade to provide an adequate road surface. If fabric were to be used, the utility truck (10 t GVW) would be the critical vehicle. The rock requirement would be reduced by 21 cm to 31 cm. A cost analysis would determine if the cost of fabric is justified.

The above example shows that a simple, 2 axle truck can stress the subgrade more than a 36 tonne log truck. The engineer should consider the possibility and frequency of overloading single-axle, single-wheel trucks. Overloading a 4,500 kg (10,000 lbs) single wheel load truck to 6,750 kg (15,000 lb) increases the the rock requirements from 51 to 62 cm.

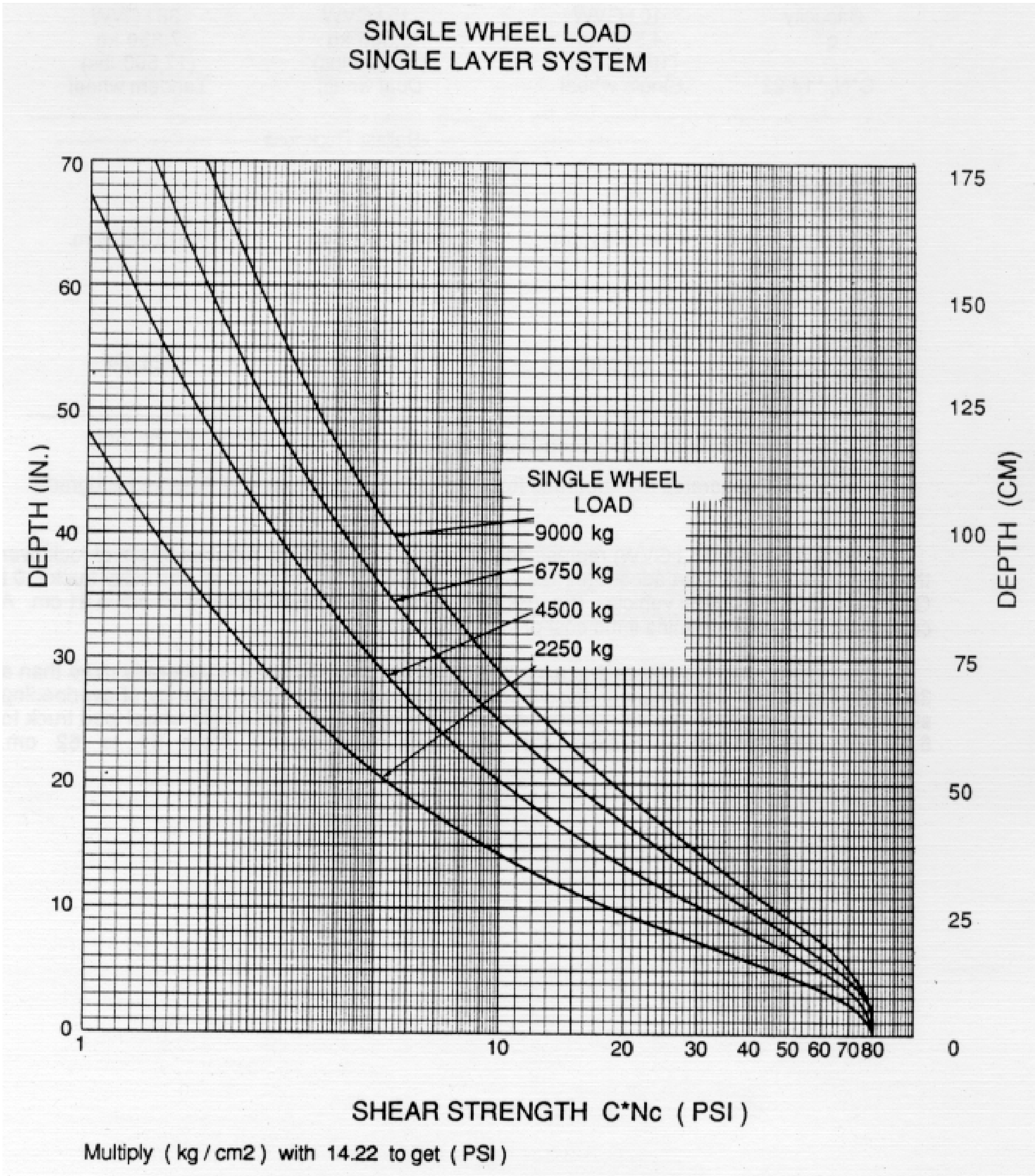


Figure 56. Ballast thickness curves for single wheel loads (from Steward, et al., 1977). Conversion factors: 1 inch = 2.5 cm ; 1kg/cm^2 = 14.22 psi.

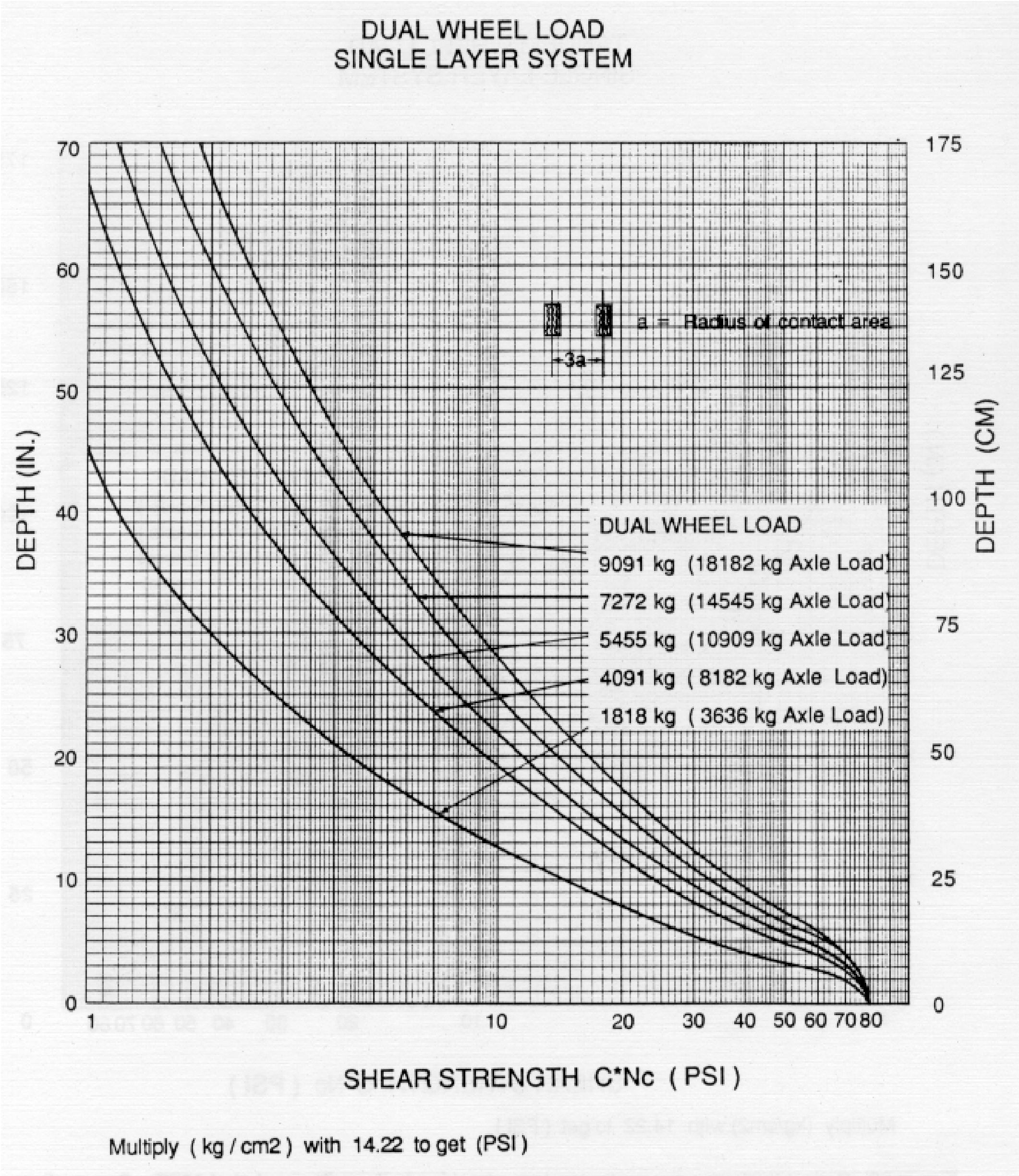


Figure 57. Ballast thickness curves for dual wheel loads (from Steward, et al., 1977). Conversion factors: 1 inch = 2.5 cm ; 1kg/cm^2 = 14.22 psi.

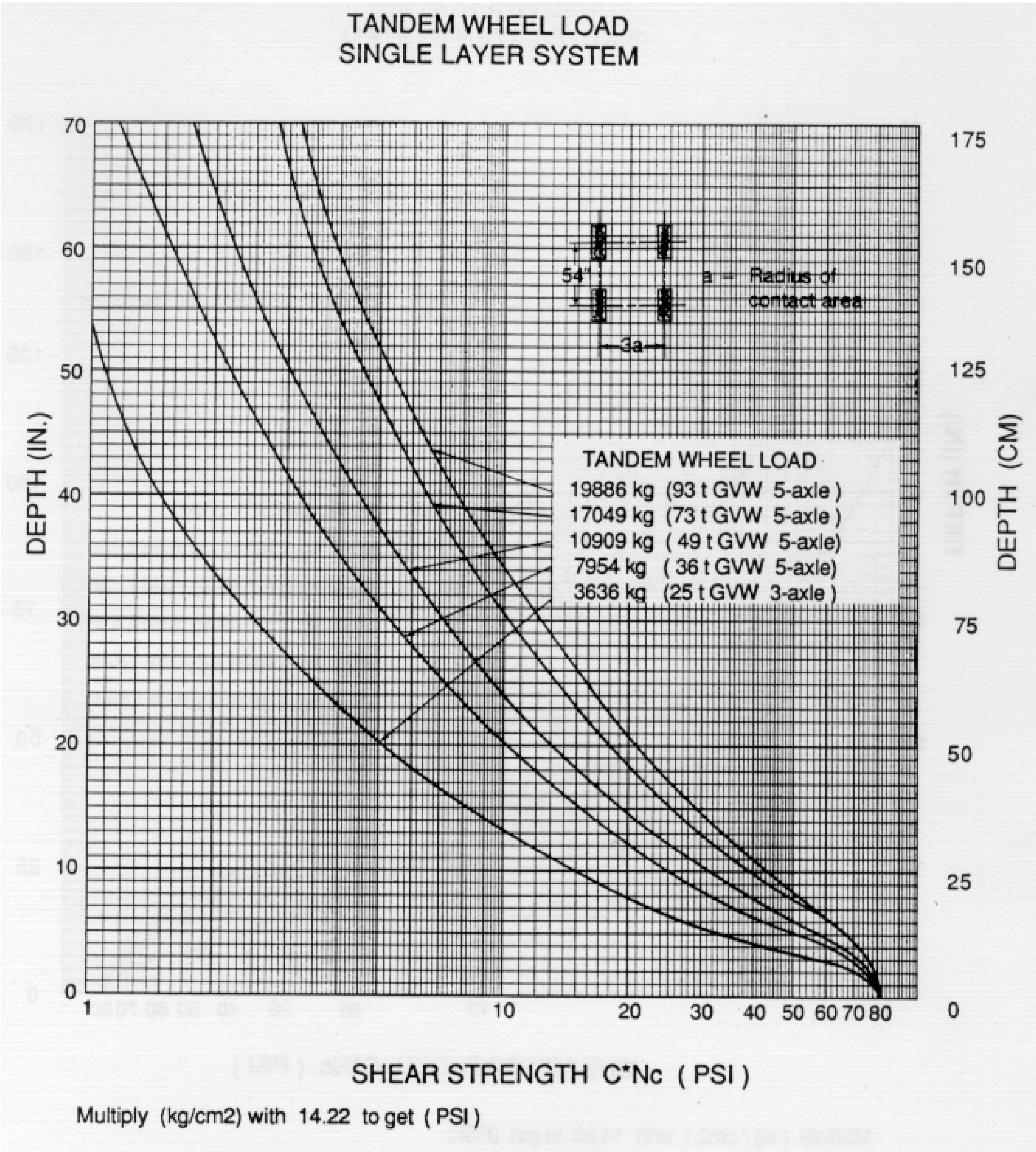


Figure 58. Ballast thickness curves for tandem wheel loads (form Steward et al 1977). Conversion factors: 1inch = 2.5 cm; 1kg/cm^2 = 14.22 psi.

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CHAPTER 4

DRAINAGE DESIGN

4.1 General Considerations

Roads will affect the natural surface and subsurface drainage pattern of a watershed or individual hillslope. Road drainage design has as its basic objective the reduction and/or elimination of energy generated by flowing water. The destructive power of flowing water, as stated in Section 3.2.2, increases exponentially as its velocity increases. Therefore, water must not be allowed to develop sufficient volume or velocity so as to cause excessive wear along ditches, below culverts, or along exposed running surfaces, cuts, or fills.

Provision for adequate drainage is of paramount importance in road design and cannot be overemphasized. The presence of excess water or moisture within the roadway will adversely affect the engineering properties of the materials with which it was constructed. Cut or fill failures, road surface erosion, and weakened subgrades followed by a mass failure are all products of inadequate or poorly designed drainage. As has been stated previously, many drainage problems can be avoided in the location and design of the road. Drainage design is most appropriately included in alignment and gradient planning.

Hillslope geomorphology and hydrologic factors are important considerations in the location, design, and construction of a road. Slope morphology impacts road drainage and ultimately road stability. Important factors are slope shape (uniform, convex, concave), slope gradient, slope length, stream drainage characteristics (e.g., braided, dendritic), depth to bedrock, bedrock characteristics (e.g., fractured, hardness, bedding), and soil texture and permeability. Slope shape (Figure 59 gives an indication of surface and subsurface water concentration or dispersion. Convex slopes (e.g., wide ridges) will tend to disperse water as it moves downhill. Straight slopes concentrate water on the lower slopes and contribute to the buildup of hydrostatic pressure. Concave slopes typically exhibit swales and draws. Water in these areas is concentrated at the lowest point on the slope and therefore represents the least desirable location for a road.

Hydrologic factors to consider in locating roads are: number of stream crossings, side slope, and moisture regime. For example, at the lowest point on the slope, only one or two stream crossings may be required. Likewise, side slopes generally are not as steep, thereby reducing the amount of excavation. However, side cast fills and drainage requirements will need careful attention since water collected from upper positions on the slope will concentrate in the lower positions. In general, roads built on the upper one-third of a slope have better soil moisture conditions and, therefore, tend to be more stable than roads built on lower positions on the slope.

Natural drainage characteristics of a hillslope, as a rule, should not be changed. For example, a drainage network will expand during a storm to include the smallest depression and draw in order to collect and transport runoff. Therefore, a culvert should be placed in each draw so as not to impede the natural disposition of stormflow. Culverts should be placed at grade and in line with the centerline of the channel. Failure to do this often results in excessive erosion of soils above and below the culvert. Also, debris cannot pass freely through the culvert causing plugging and oftentimes complete destruction of the road prism. Headwater streams are of particular concern (point A, Figure 60) since it is common to perceive that measurable flows cannot be generated from the moisture collection area above the crossings. However, little or no drainage on road crossings in these areas is notorious for causing major slide and debris torrents, especially if they are located on convex slope breaks.

Increased risks of road failures are created at points A and B. At point A, water will pond above the road fill or flow downslope through the roadside ditch to point B. Ponding at A may cause weakening and/or erosion of the subgrade . If the culvert on Stream 1 plugs, water and debris will flow to point A and from A to B. Hence, the culvert at B is handling discharge from all

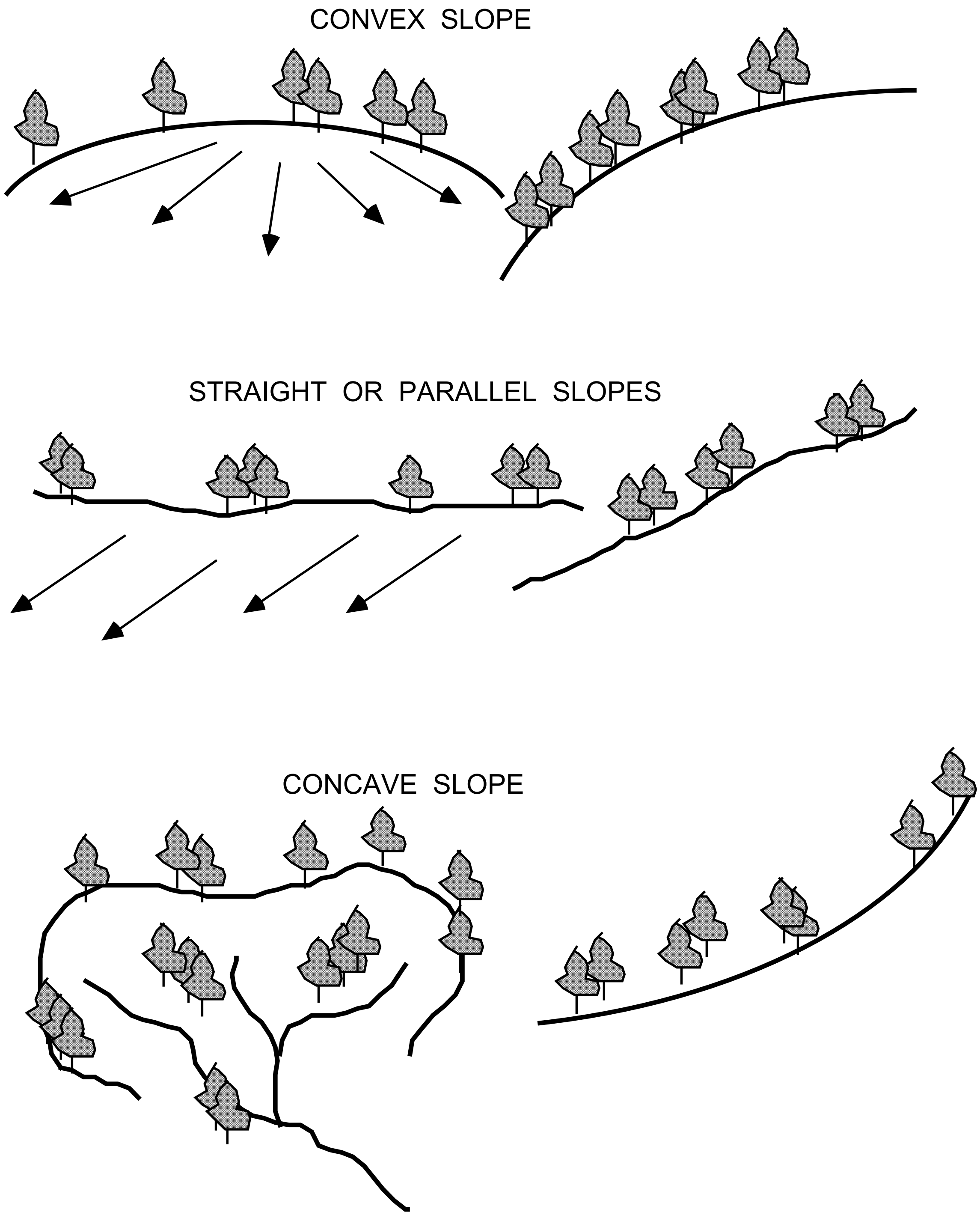


Figure 59. Slope shape and its impact on slope hydrology. Slope shape determines whether water is dispersed or concentrated. (US Forest Service,1979)

three streams. If designed to minimum specifications, it is unlikely that either the ditch or the culvert at B will be able to efficiently discharge flow and debris from all three streams resulting in overflow and possible failure of the road at point B.

A road drainage system must satisfy two main criteria if it is to be effective throughout its design life:

1. It must allow for a minimum of disturbance of the natural drainage pattern.

2. It must drain surface and subsurface water away from the roadway and dissipate it in a way that prevents excessive collection of water in unstable areas and subsequent downstream erosion.

The design of drainage structures is based on the sciences of hydrology and hydraulics--the former deals with the occurrence and form of water in the natural environment (precipitation, streamflow, soil moisture, etc.) while the latter deals with the engineering properties of fluids in motion.

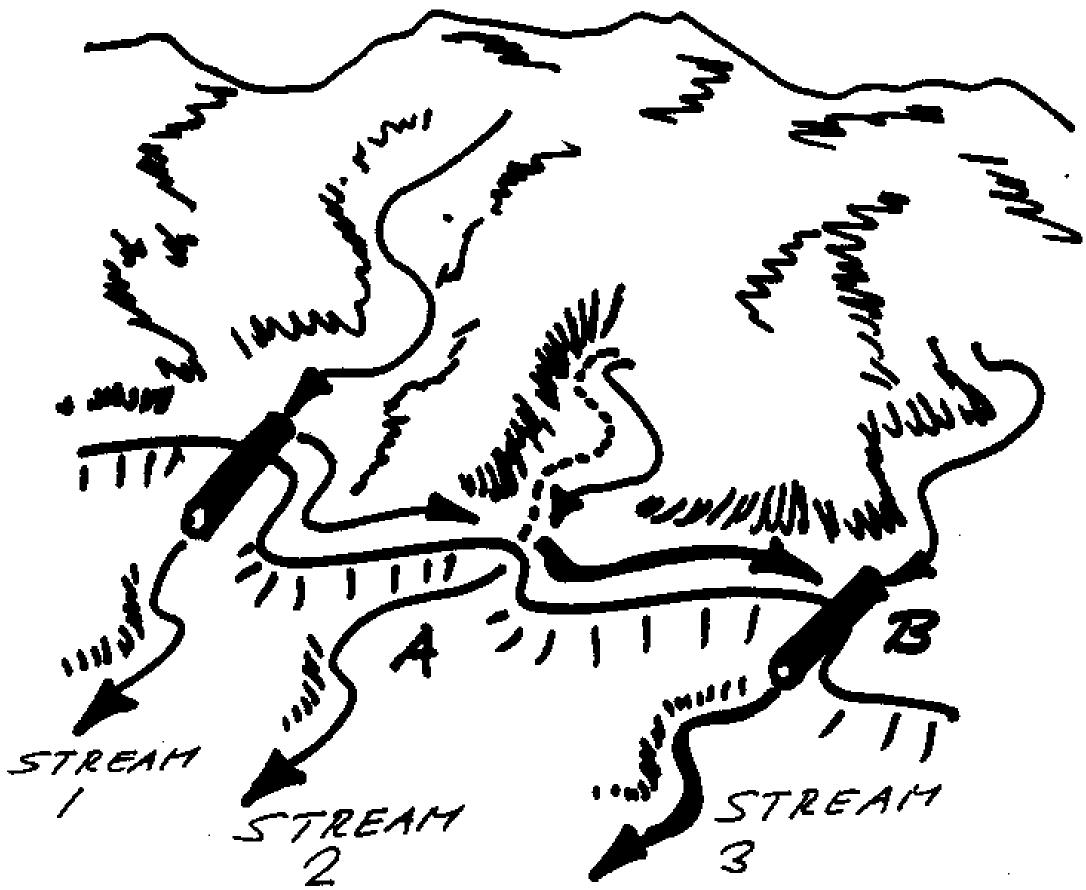


Figure 60. Culvert and road locations have modified drainage patterns of ephemeral streams 2 and 3. Locations A and B become potential failure sites. Stream 3 is forced to accept more water below B due to inadequate drainage at A.

4.2 Estimating runoff

Any drainage installation is sized according to the probability of occurrence of an expected peak discharge during the design life of the installation. This, of course, is related to the intensity and duration of rainfall events occurring not only in the direct vicinity of the structure, but also upstream of the structure. In snow zones, peak discharge may be the result of an intense warming period causing rapid melting of the snowpack.

In addition to considering intensity and duration of a peak rainfall event, the frequency, or how often the design maximum may be expected to occur, is also a consideration and is most often based on the life of the road, traffic, and consequences of failure. Primary highways often incorporate frequency periods of 50 to 100 years, secondary roads 25 years, and low volume forest roads 10 to 25 years.

Of the water that reaches the ground in the form of rain, some will percolate into the soil to be stored until it is taken up by plants or transported through pores as subsurface flow, some will evaporate back into the atmosphere, and the rest will contribute to overland flow or runoff. Streamflow consists of stored soil moisture which is supplied to the stream at a more or less constant rate throughout the year in the form of subsurface or groundwater flow plus water which is contributed to the channel more rapidly as the drainage net expands into ephemeral channels to incorporate excess rainfall during a major storm event. The proportion of rainfall that eventually becomes streamflow is dependent on the following factors:

1. <u>The size of the drainage area</u>. The larger the area, the greater the volume of runoff. An estimate of basin area is needed in order to use runoff formulas and charts.

2. <u>Topography</u>. Runoff volume generally increases with steepness of slope. Average slope, basin elevation, and aspect, although not often called for in most runoff formulas and charts, may provide helpful clues in refining a design.

3. <u>Soil</u>. Runoff varies with soil characteristics, particularly permeability and infiltration capacity. The infiltration rate of a dry soil, by nature of its intrinsic permeability, will steadily decrease with time as it becomes wetted, given a constant rainfall rate. If the rainfall rate is greater than the final infiltration rate of the soil (infiltration capacity), that quantity of water which cannot be absorbed is stored in depressions in the ground or runs off the surface. Any condition which adversely affects the infiltration characteristics of the soil will increase the amount of runoff. Such conditions may include hydrophobicity, compaction, and frozen earth.

A number of different methods are available to predict peak flows. Flood frequency analysis is the most accurate method employed when sufficient hydrologic data is available. For instance, the United States Geological Survey has published empirical equations providing estimates of peak discharges from streams in many parts of the United States based on regional data collected from "gauged" streams. In northwest Oregon, frequency analysis has revealed that discharge for the flow event having a 25-year recurrence interval is most closely correlated with drainage area and precipitation intensity for the 2-year, 24-hour storm event. This is, by far, the best means of estimating peak flows on an ungauged stream since the recurrence interval associated with any given flow event can be identified and used for evaluating the probability of failure.

The probability of occurrence of peak flows exceeding the design capacity of a proposed stream crossing installation should be determined and used in the design procedure. To incorporate this information into the design, the risk of failure over the design life must be specified. By identifying an acceptable level of risk, the land manager is formally stating the desired level of success (or failure) to be achieved with road drainage structures. Table 25 lists flood recurrence intervals for installations in relation to their design life and probability of failure.

table 31 Flood recurrence interval (years) in relation to design life and probability of failure.* (Megahan, 1977)

Design Life (years)	Chance of Failure (%) 10	20	30	40	50	60	70
	-------------------Recurrence Interval (years)----------------------						
5	48	23	15	10	8	6	5
10	95	45	29	20	15	11	9
15	100+	68	43	30	22	17	13
20	100+	90	57	40	229	22	17
25	200+	100+	71	49	37	28	21
30	200+	100+	85	59	44	33	25
40	300+	100+	100+	79	58	44	34
50	400+	200+	100+	98	73	55	42

*Based on formula $P = 1 - (1 - 1/T)^n$, where n = design life (years), T = peak flow recurrence interval (years), P = chance of failure (%).

<u>EXAMPLE.</u> If a road culvert is to last 25 years with a 40% chance of failure during the design life, it should be designed for a 49-year peak flow event (i.e., 49-year recurrence interval).

When streamflow records are not available, peak discharge can be estimated by the "rational" method or formula and is recommended for use on channels draining less than 80 hectares (200 acres):

$$Q = 0.278\ C\ i\ A$$

where Q = peak discharge, (m^3/s)
i = rainfall intensity (mm/hr) for a critical time period
A = drainage area (km^2).

(In English units the formula is expressed as:

$$Q = C\ i\ A$$

where Q = peak discharge (ft^3/s)
i = rainfall intensity (in/hr) for a critical time period, t_c
A = drainage area (acres).)

The runoff coefficient, C, expresses the ratio of rate of runoff to rate of rainfall and is shown below in Table 26. The variable t_c is the time of concentration of the watershed (hours).

table 32 Values of relative imperviousness for use in rational formula. (American Iron and Steel Institute, 1971)

Type of Surface	Factor C
Sandy soil, flat, 2%	0.05-0.10
Sandy soil, average, 2-7%	0.10-0.15
Sandy soil, steep, 7	0.15-0.20
Heavy soil, flat, 2%	0.13-0.17
Heavy soil, average, 2-7%	0.18-0.22
Heavy soil, steep, 7%	0.25-0.35
Asphaltic pavements	0.80-0.95
Concrete pavements	0.70-0.95
Gravel or macadam pavements	0.35-0.70

Numerous assumptions are necessary for use of the rational formula: (1) the rate of runoff must equal the rate of supply (rainfall excess) if t_{rain} is greater than or equal to t_c; (2) the maximum discharge occurs when the entire area is contributing runoff simultaneously; (3) at equilibrium, the duration of rainfall at intensity **I** is $t = t_c$; (4) rainfall is uniformly distributed over the basin; (5) recurrence interval of Q is the same as the frequency of occurrence of rainfall intensity **I**; (6) the runoff coefficient is constant between storms and during a given storm and is determined solely by basin surface conditions. The fact that climate and watershed response are variable and dynamic explain much of the error associated with the use of this method.

Manning's formula is perhaps the most widely used empirical equation for estimating discharge since it relies solely on channel characteristics that are easily measured. Manning's formula is:

$$Q = n^{-1} A R^{2/3} S^{1/2}$$

where Q = discharge (m^3/s)
A = cross sectional area of the stream (m^2)
R = hydraulic radius (m), (area/wetted perimeter of the channel)
S = slope of the water surface
n = roughness coefficient of the channel.

(In English units, Manning's equation is : $Q = 1.486 \; n^{-1} A R^{2/3} S^{1/2}$

where Q = discharge (cfs)
A = cross sectional area of the stream (ft^2)
R = hydraulic radius (ft)
S = slope of the water surface
n = roughness coefficient of the channel.)

Values for Manning's roughness coefficient are presented in Table 27.

table 33 Manning's n for natural stream channels (surface width at flood stage) less than 30 m (Highway Task Force, 1971).

Natural stream channels	-- n --
1. Fairly regular section:	
Some grass and weeds, little or no brush	0.030 - 0.035
Dense growth of weeds, depth of flow materially greater than weed height	0.035 - 0.050
Some weeds, light brush on banks	0.035 - 0.050
Some weeds, heavy brush on banks	0.050 - 0.070
Some weeds, dense willows on banks	0.060 - 0.080
For trees within channel, with branches submerged at high stage, increase above values by	0.010 - 0.020
2. Irregular sections, with pools, slight channel meander; increase values given above by	0.010 - 0.020
3. Mountain streams, no vegetation in channel, banks usually steep, trees and brush along banks submerged at high stage:	
Bottom of gravel, cobbles, and few boulders	0.040 - 0.050
Bottom of cobbles with large boulders	0.050 - 0.070

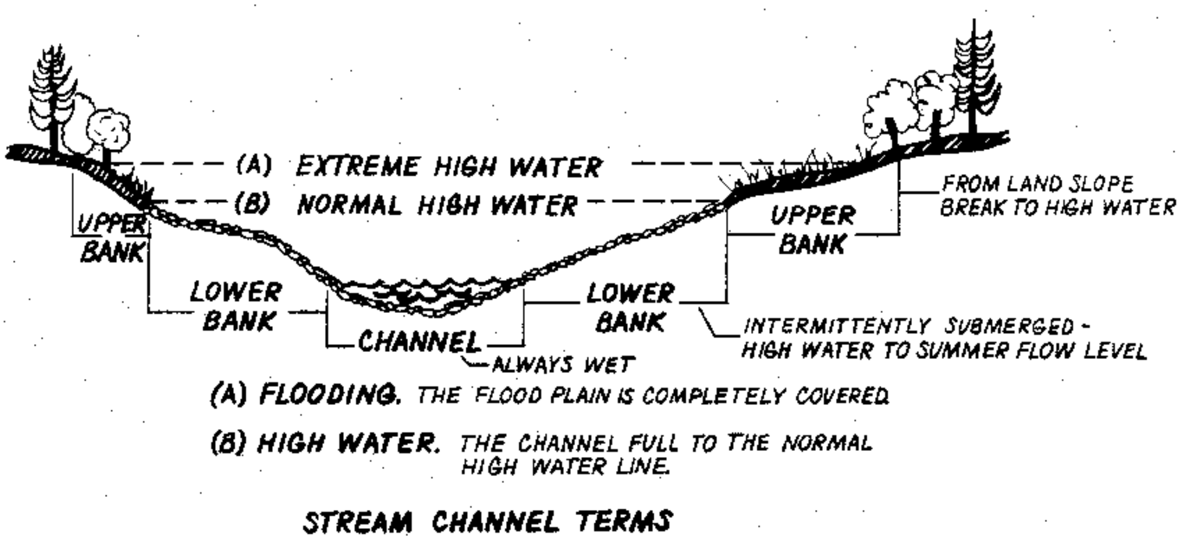


Figure 61. Determining high water levels for measurement of stream channel dimensions.

Area and wetted perimeter are determined in the field by observing high water marks on the adjacent stream banks (Figure 61). Look in the stream bed for scour effect and soil discoloration. Scour and soil erosion found outside the stream channel on the floodplains may be caused by the 10-year peak flood. Examining tree trunks and brush in the channel and floodplain may reveal small floatable debris hung up in the vegetation. Log jams are also a good indication of flood marks because their age can be estimated and old, high log jams will show the high watermark on the logs. The difficulty in associating high water marks with flow events of a specified recurrence interval makes values obtained by this method subject to gross inaccuracy. If the 10-year flood can be determined, flow levels for events with a higher recurrence interval can be determined roughly from Table 28.

table 34 Relationship of peak flow with different return periods. (Nagy, et al, 1980)

Peak flow return period (years)	Factor of flood intensity (10-year peak flow = 1.00)
10	1.00
25	1.25
50	1.50
100	1.80

A key assumption in the use of Manning's equation is that uniform steady flow exists. It is doubtful that high gradient forested streams ever exhibit this condition. (Campbell, et al., 1982) When sufficient hydrologic data is lacking, however, Manning's equation, together with observations of flow conditions in similar channels having flow and/or precipitation records, provide the best estimate of stream discharge for purposes of designing stream crossings. An example illustrating the use of Manning's equation to calculate peak discharge is as follows:

EXAMPLE: A trapezoidal channel of straight alignment and uniform cross section has a bottom width of 10 meters, side slopes 1:1, channel slope 0.003, and high water depth (25-year event) of 5 meters. The channel has weeds and heavy brush along its banks.

1. The wetted perimeter is equal to $10 + 2(5 / \cos 45°) = 24.1$ m.

2. The cross sectional area is equal to 1/2 x sum of parallel sides x perpendicular height = $0.5(10 + 20)(5) = 75$ m^2.

3. The hydraulic radius is cross sectional area ÷ wetted perimeter = $75 \div 24.1 = 3.1$ m.

4. Manning's n from Table 25 is n = 0.06

5. Discharge, Q, from Manning's equation= $(0.06)^{-1}(75)(3.1)^{2/3}(0.003)^{1/2} = 146$ m^3/sec (Velocity, if needed, can be computed by $Q \div A = 1.9$ m/sec.)

4.3 Channel Crossings

4.3.1 Location of Channel Crossings

Channel crossings require careful design and construction. Functionally, they must (1) allow for passage of the maximum amount of water which can reasonably be expected to occur within the lifetime of the structure and (2) not degrade water quality or endanger the structure itself or any downstream structures. It should be pointed out that most road failures are related to inadequate water passage structures and fill design and placement as well as poor construction practices in such locations.

Accelerated erosion brought about by failure of channel crossing structures can be caused by:

1 Inadequate design to handle peak flow and debris. Water will back up behind structure, sat- [1]urating the fill and creating added hydrostatic pressure. Water will overflow the structure and the fill may be washed out.

2. Inadequate outlet design. By constricting flow through a small area, water velocity (along with its erosive power) will increase. Outlets need to be properly designed in order to withstand high flow velocities and thus avoid excessive downstream erosion and eventual road failure.

3. Poor location of crossing. Crossings need to be located along relatively stable stretches where stream bottoms and banks exhibit little signs of excessive erosion or deposition. Meandering and/or multiple channels often indicate unstable conditions. If there is no choice but to use a poor location, careful consideration of the type of crossing selected, along with bank and stream bottom stabilization and protection measures, should be given.

There are three generally accepted methods used to cross channels on low volume roads--bridges, fords, and culverts. The selection is based on traffic volume and characteristics, site conditions (hydrologic/hydraulic conditions of channel), and management needs such as occasional closure, continuous use, safety considerations, resource impact (fish, wildlife, sediment). Factors to consider when selecting a crossing type are listed as follows:

1. Bridges: high traffic volume, large and variable water volume, high debris potential, sensitive channel bottom and banks, significant fish resource, large elevation difference between channel and road grade

2. Culvert: Medium to low water volume, medium to low debris potential, fish resource not significant, elevation difference between channel and road grade less than 10 meters, high traffic volume

3. Ford: low to intermittent water flow, high debris potential, no fish resource, road grade can be brought down to channel bottom, low traffic volume

All three channel crossing types require a careful analysis of both vertical and horizontal alignment. In particular, careful analysis of curve widening requirements is imperative in relation to the specified critical vehicle. Channel crossings are fixed structures where the road way width cannot be temporarily widened. Road width, curvature, approach, and exit tangents govern the vehicle dimensions which can pass the crossing.

Except for bridge locations, roads should climb away from channel crossings in both directions wherever practical so high water will not flow along the road surface. This is particularly true for ford installations.

4.3.2 Fords

Fords are a convenient way to provide waterway crossing in areas subject to flash floods, seasonal high storm runoff peaks, or frequent heavy passage of debris or avalanches. Debris will simply wash over the road structure. After the incident, some clearing may be necessary to allow for vehicle passage. Figure 62 shows a very simple ford construction where rock-filled gabions are used to provide a road bed through the stream channel.

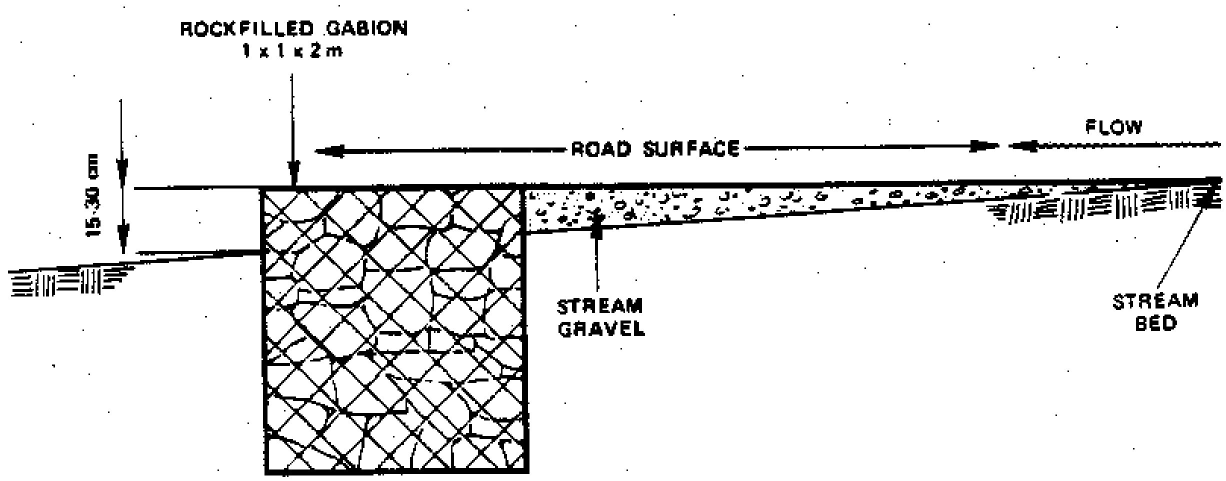


Figure 62. Ford construction stabilized by gabions placed on the downstream end. (Megahan, 1977)

There are some design considerations which need careful attention:

1. The ford should allow for passage of debris and water without diverting it onto the road surface. The ford results in a stream bed gradient reduction. Therefore, debris has a tendency to be deposited on top of a ford because of reduced flow velocity.

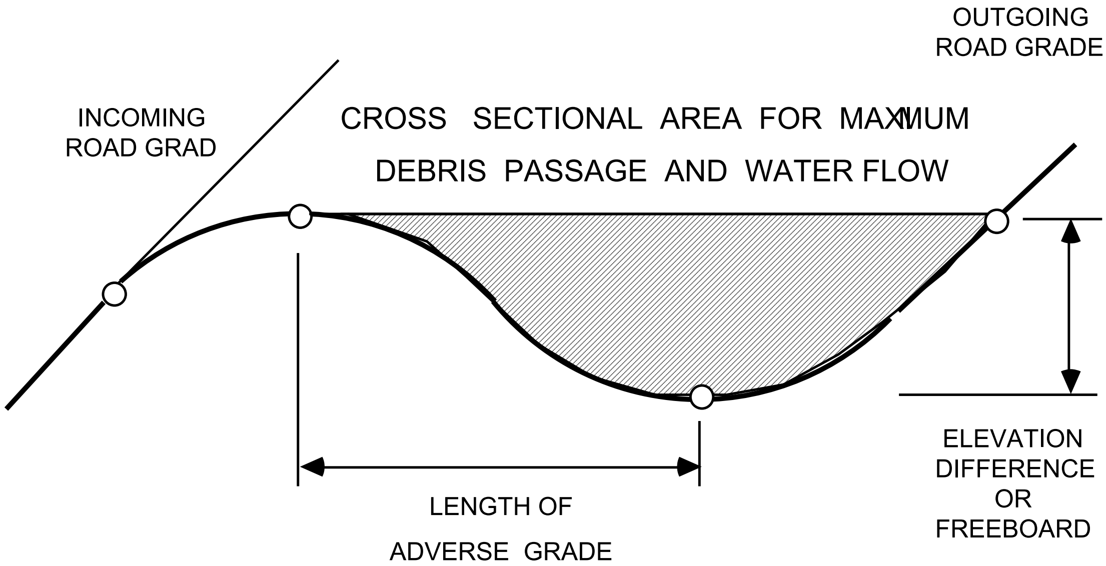


Figure 63. Profile view of stream crossing with a ford. A dip in the adverse grade provideschanneling preventing debris accumulation from diverting the stream on to and along the road surface. The profile of the ford along with vehicle dimensions must be considered to insure proper clearance and vehicle passage. (After Kuonen, 1983)

2. Fords should be designed with steep, short banks, which help to confine and channel the stream (Figure 63). The steepness and length of the adverse grade out of ford depends on

the anticipated debris and water handling capacity required as well as vehicle geometry (See Chapter 3.1.3). Typically, the design vehicle should be able to pass the ford without difficulty. Critical vehicles (vehicles which have to use the road, but only very infrequently) may require a temporary fill to allow passage.

An alternative to the above described ford is a "hardened" fill with culvert (Figure 64). This approach is an attractive alternative for crossing streams that are prone to torrents. The prevailing low flow conditions are handled by a small culvert and the occasional flash flood or debris avalanche will simply wash over the road surface. The fill surface has to be hardened either by concrete or large rock able to withstand the tremendous kinetic energy associated with floods and torrents. Vertical curve design through the stream has to include an adverse grade as discussed for the typical ford.

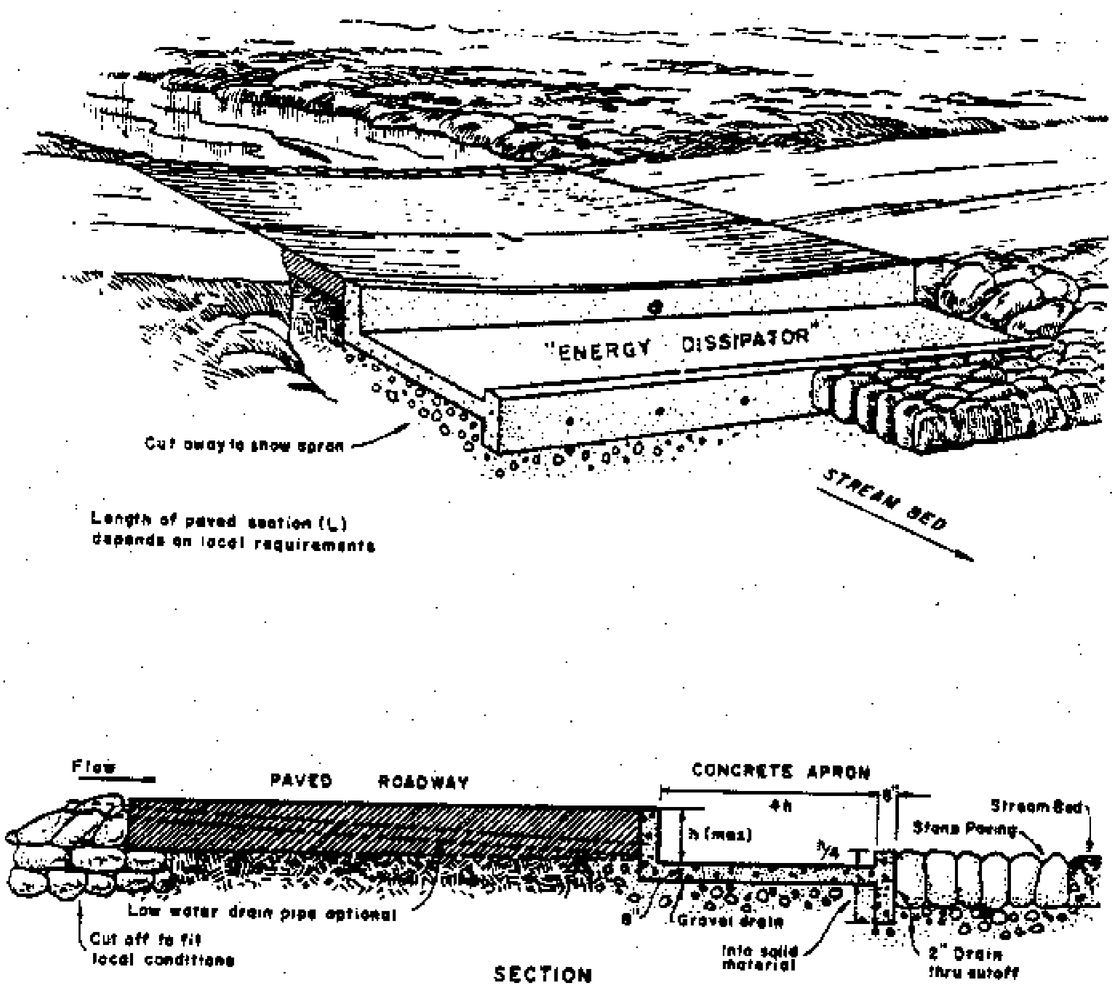


Figure 64. Hardened fill stream crossings provide an attractive alternative for streams prone to torrents or debris avalanches (Amimoto, 1978).

4.3.3 Culverts

Culverts are by far the most commonly used channel crossing structure used on forest roads. Culvert types normally used, and the conditions under which they are used, are as follows:

Corrugated metal pipe (CMP) ....All conditions except those noted below

CMP with paved invert ....Water carries sediments erosive to metal

CM pipe-arch ....Low fills; limited head room

Multi-plate..Large sizes (greater than 1.8 meters)

Reinforced concrete pipe (RCP) ....Corrosive soil or water, as salt water; short haul from plant; unloading and placing equipment available

Reinforced concrete box ....Extra large waterway; migratory fish way

Although more expensive than round culverts, pipe-arch or plate arch types are preferred over ordinary round pipes. Pipe-arch culverts, besides having a more efficient opening per unit area than round pipe for a given discharge, will collect bottom sediments over time when it is installed slightly below the stream grade. They also require lower fills. However, during periods of low flow, water in pipes with this shape may be spread so thin across the bottom that fish passage is impossible. A plate-arch set in concrete footings is the most desirable type from a fish passage standpoint since it has no bottom. The stream can remain virtually untouched if care is exercised during its installation. (Yee and Roelofs, 1980)

Regardless of the type of culvert, they should all conform to proper design standards with regards to alignment with the channel, capacity, debris control, and energy dissipation. They should all perform the following functions:

1. The culvert with its appurtenant entrance and outlet structures should efficiently discharge water, bedload, and floating debris at all stages of flow.

2. It should cause no direct or indirect property damage.

3. It should provide adequate transport of water, debris, and sediment without drastic changes in flow patterns above or below the structure.

4. It should be designed so that future channel and highway improvements can be made without much difficulty.

5. It should be designed to function properly after fill has settled.

6. It should not cause objectionable stagnant pools in which mosquitoes could breed.

7. It should be designed to accommodate increased runoff occasioned by anticipated land development.

8. It should be economical to build, hydraulically adequate to handle design discharge, structurally durable, and easy to maintain.

9. It should be designed to avoid excessive ponding at the entrance, which may cause property damage, accumulation of sediment, culvert clogging, saturation of fills, or detrimental upstream deposits of debris.

10. Entrance structures should be designed to screen out material which will not pass through the culvert, reduce entrance losses to a minimum, make use of velocity of approach insofar as practical, and by use of transitions and increased slopes, as necessary, facilitate channel flow entering the culvert.

11. The outlet design should be effective in re-establishing tolerable non-erosive channel flow within the right-of-way or within a reasonably short distance below the culvert, and should resist undercutting and washout.

12. Energy dissipators should be simple, easy to build, economical and reasonably self-cleaning during periods of low flow.

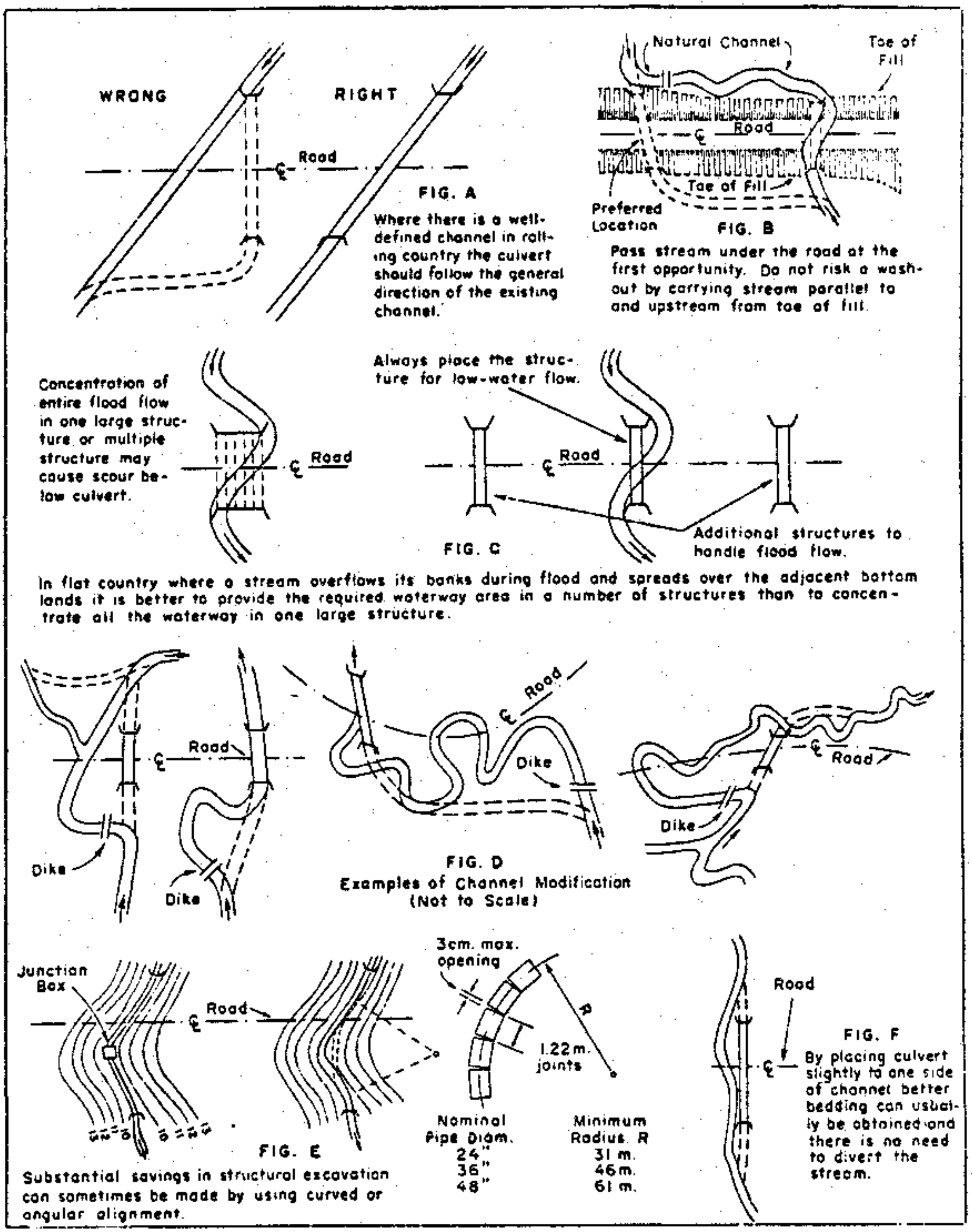


Figure 65. Possible culvert alignments to minimize channel scouring. (USDA, Forest Service, 1971)

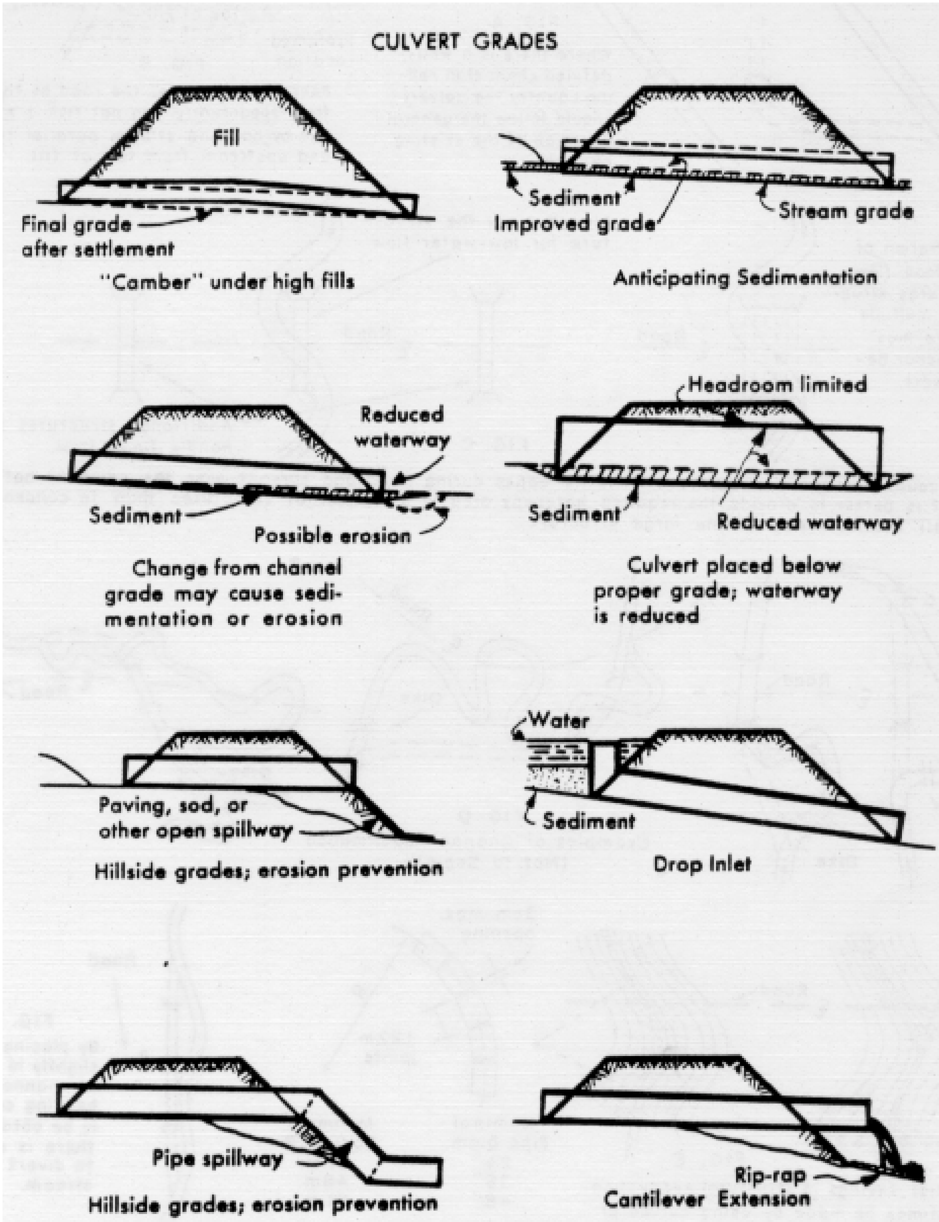


Figure 66. Proper culvert grades. (Highway Task Force, 1971)

13. Alignment should be such that water enters and exits the culvert directly. Any abrupt change in direction at either end will retard flow and cause ponding, erosion, or a buildup of debris at the culvert entrance. All of these conditions could lead to failure. (See Figure 65 for suggested culvert-channel alignment configurations and Figure 66 for suggested culvert grades. In practice, culvert grade lines generally coincide with the average streambed above and below the culvert.)

If there are existing roads in the watershed, examination of the performance of existing culverts often serves as the best guide to determining the type, size, and accompanying inlet/outlet improvements needed for the proposed stream crossing. For estimating streamflow on many forest watersheds, existing culvert installations may be used as "control sections". Flow can be calculated as the product of water velocity (V) and cross-sectional area (A):

$$Q = A * V.$$

Cross-sectional area of water flowing in a round culvert is difficult to measure, however a rough estimate can be calculated from the following equation:

$$A = [(\pi r^2 ß) / 180] - (r^2 - rd) \sin ß$$

where r = culvert radius

d = measured depth of flow

ß = angle (°) between radial lines to the bottom of the culvert and to the water surface (Figure 62)

$$= \cos^{-1} [(r-d) / r].$$

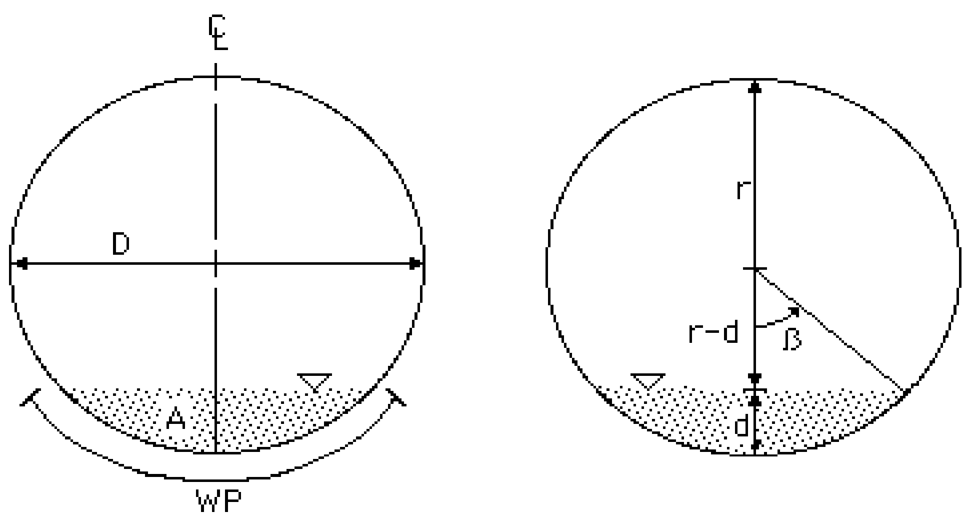


Figure 67. Definition sketch of variables used in flow calculations.

Velocity can be calculated using Manning's equation:

$$V = Q / A = (n^{-1})(R^{2/3})(S^{1/2})$$

where S = slope
n = Manning's roughness coefficient
R = hydraulic radius (meters)

$= 0.5r - [(\pi ß)^{-1}(90(r - d) \sin ß)]$ (see Figure 67)

Values for coefficient of roughness (n) for culverts are given in Table 29.

table 35 Values for coefficient of roughness (n) for culverts. (Highway Task Force,1971)

	Culvert diameter (ft)*	Annular corrugations (in)*	n
corrugated metal	1 to 8	2-2/3 x 1/2	0.024
	3 to 8	3 x 1	0.027
concrete	all diameters	---	0.012

*1 ft = 0.30 m, 1 in. = 2.54 cm

The types of flow conditions found in conventional circular pipes and pipe-arch culverts are illustrated in Figure 68. Under inlet control, the cross-sectional area of the barrel, the inlet configuration or geometry, and the amount of headwater or ponding are of primary importance. Under outlet control, tailwater depth in the outlet channel and slope, roughness, and length of the barrel are also considered. The flow capacity of most culverts installed in forested areas is usually determined by the characteristics of the inlet since nearly any pipe that has a bottom slope of 1.5% or greater will exhibit inlet control. At slopes of 3% or greater, the culvert can become self-cleaning of sediment.

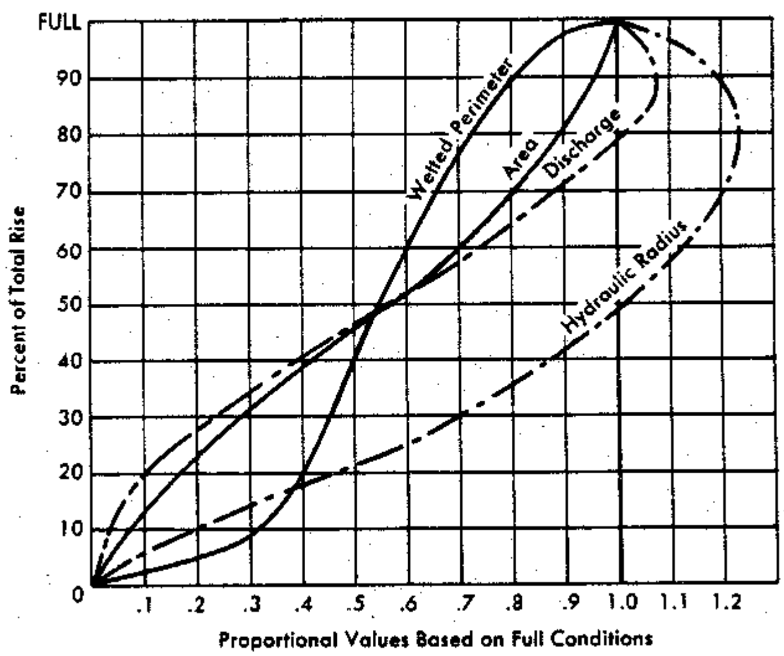


Figure 68. Hydraulics of culverts (Highway Task Force, 1971)

Once the design peak discharge has been determined by one of the methods discussed above, the size of pipe required to handle the discharge can be determined from available equations, charts, tables, nomographs, etc., such as the ones presented in Figures 70, 71, 72, 73 and 74. Figure 69 provides an example of a work sheet which can be used for diameter and flow capacity calculations. If outlet control is indicated (for example, in a low gradient reach where "backwater effects" may be created at the outlet end), the reader is referred to Handbook of Steel Drainage and Highway Construction Products (1971) or Circular No. 5 published by the U. S. Department of Commerce (1963). Outlet control conditions are shown in Figure 74 for a corrugated metal pipe.

It is important to keep in mind that in addition to discharge from areas upstream of the installation, the culvert must be able to handle accumulated water from roadside ditches recalling that roadside ditches on roads lower on the slope intercept more subsurface water than those on roads higher on the slope. Sudden surges from rapid snowmelt (if applicable) must also be allowed for. Organic debris and bedload sediments can plug a culvert and can greatly reduce culvert efficiency. For these reasons, an "oversized" culvert design may be indicated.

Inlet characteristics can greatly influence flow efficiency through the culvert. The end either (1) projects beyond the fill, (2) is flush with a headwall, or (3) is supplemented with a manufactured mitered steel end section. Inlets with headwalls are generally the most efficient followed by culverts with mitered inlets and finally culverts with projecting entrances. When headwater depths are 1 to 2 times greater than culvert diameter, culverts with headwalls have an increase in flow capacity of approximately 11 and 15%, respectively, over culverts with projecting entrances.

Procedure for Selection of Culvert Size

Note: Culvert design sheets, similar to Figure 69 should be used to record design data.

Step 1: List given data:

a. Design discharge Q, in m^3/sec.

b. Approximate length of culvert, in meters.

c. Allowable headwater depth, in meters. Headwater depth is defined as the vertical distance from the culvert invert (flow line) at the entrance to the water surface elevation permissible in the approach channel upstream from the culvert.

d. Type of culvert, including barrel material, barrel cross-sectional shape and entrance type.

e. Slope of culvert. (If grade is given in percent, convert to slope in meters per meter).

f. Allowable outlet velocity (if scour or fish passage is a concern).

g. Convert metric units to english units for use with the nomographs.

Volume flow Q(m^3/sec) to Q(cfs) : 1 m^3/sec = 35.2 cfs (cubic ft/sec). Multiply Q(m^3/sec) by 35.2 to get Q(cfs)

Length, Diameter (meter): 1 meter = 3.3 ft.; 1 cm = 0.4 inches. Multiply (cm) by 0.4 to get (inches). Multiply (meter) by 3.3 to get (feet)

Step 2: Determine a trial size culvert:

a. Refer to the inlet control nomograph for the culvert type selected.

b. Using an HW / D (Headwater depth/Diameter)) of approximately 1.5 and the scale for the entrance type to be used, find a trial size culvert by following the instructions for use of the nomographs. If a lesser or greater relative headwater depth should be needed, another value of HW/D may be used.

c. If the trial size for the culvert is obviously too large because of limited height of embankment or size availability, try different HW/D values or multiple culverts by dividing the discharge equally for the number of culverts used. Raising the embankment height or using a pipe arch and box culvert which allow for lower fill heights is more efficient hydraulically than using the multiple culvert approach. Given equal end areas, a pipe arch will handle a larger flow than two round culverts. Selection should be based on an economic analysis.

<u>Step 3:</u> Find headwater (HW) depth for the trial size culvert:

a. Determine and record HW depth by use of the appropriate inlet control nomograph. Tailwater (TW) conditions are to be neglected in this determination. HW in this case is found by simply multiplying HW/D (obtained from the nomograph) by D.

<u>Step 4:</u> Check outlet velocities for size selected:

a. If inlet control governs, outlet velocity can be assumed to equal normal velocity in open-channel flow as computed by Manning's equation for the barrel size, roughness, and slope of culvert selected.

Note: In computing outlet velocities, charts and tables such as those provided by U.S. Army Corp of Engineers, Bureau of Reclamation, and Department of Commerce are helpful (see Literature Cited).

<u>Step 5:</u> Try a culvert of another type or shape and determine size and HW by the procedure above.

<u>Step 6:</u> Record final selection of culvert with size, type, outlet velocity, required HW and economic justification. A good historical record of culvert design, installation, and performance observations can be a valuable tool in planning and designing future installations.

<u>Instructions for Using Inlet Control Nomographs</u>

<u>1. To determine headwater (HW):</u>

a. Connect with a straight edge the given culvert diameter or height (D) and the discharge Q, or Q/B for box culverts; mark intersection of straightedge on HW/D scale mark (1).

b. If HW / D scale mark (1) represents entrance type used, read HW / D on scale (1). If some other entrance type is used, extend the point of intersection found in (a) horizontally to scale (2) or (3) and read HW/D.

c. Compute HW by multiplying HW / D by D.

<u>2. To determine culvert size:</u>

a. Given an HW / D value, locate HW / D on scale for appropriate entrance type. If scale (2) or (3) is used, extend HW / D point horizontally to scale (1).

b. Connect point on HW / D scale (1) as found in (a) above to given discharge and read diameter of culvert required.

3. To determine discharge (Q):

a. Given HW and D, locate HW / D on scale for appropriate entrance type. Continue as in 2a.

b. Connect point on HW / D scale (1) as found in (a) above, the size of culvert on the left scale, and read Q or Q/B on the discharge scale.

c. If Q/B is read in (b) multiply by B to find Q.

Good installation practices are essential for proper functioning of culverts, regardless of the material used in the construction of the culvert (Figure 75). Flexible pipe such as aluminum, steel, or polyethylene, requires good side support and compaction, particularly in the larger sizes. It is recommended that the road be constructed to grade or at least a meter above the top of the pipe, the fill left to settle and then excavated to form the required trench.

The foundation dictates if bedding is needed or not. Proper foundation maintains the conduit on a uniform grade. Most times, the culvert can be laid without bedding, however, a few centimeters of bedding helps in installation of the culvert. When bedding is required, the depth should be 8 cm if the foundation material is soil and 30 cm if it is rock.

Backfilling is the most important aspect of culvert installation. Ten percent of the loading is taken by the pipe and 90 percent is taken by the material surrounding the pipe if backfilling is done correctly. Backfill material should consist of earth, sand, gravel, rock or combinations thereof, free of humus, organic matter, vegetative matter, frozen material, clods, sticks and debris and containing no stones greater than 8 cm (3 in) in diameter. It should be placed in layers of no greater than 15 cm (6 in) and compacted up to 95% of Proctor density at or near the optimum moisture content for the material.

PROJECT: 432 (1)

DESIGNER: J.A.F.

DATE: 8-20-61

HYDROLOGIC AND CHANNEL INFORMATION

Q_1 = 180 cfs. = Q_{25}

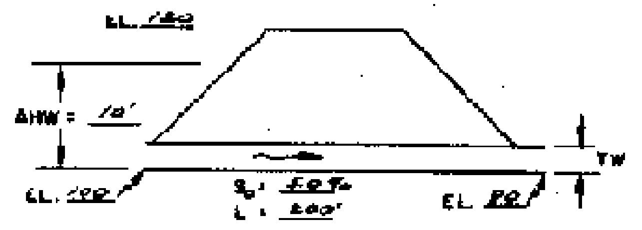
Q_2 = 225 = Q_{50}

TW_1 = 3'

TW_2 = 4'

SKETCH

STATION: 5+30



ALLOWABLE OUTLET VELOCITY: 20'/sec.

CULVERT TYPE	Q	SIZE	HEADWATER COMPUTATION: INLET CONT. $\frac{HW}{D}$	INLET CONT. HW	OUTLET CONTROL K_e	d_c	$\frac{d_c+D}{2}$	h_o	H	LS_o	HW	CONTROLLING HW	OUTLET VELOCITY	COST	COMMENTS
CMP Proj. Inlet	180	60"	1.51	7.55	.9	Try smaller size @ HW = 10									
"	180	54"	2.2	9.9'	.9	4.0	4.25	4.25	5.8	10'	4.65	9.9'	16.5		Inlet Control Governs
"	225	54	3.15	14.2	.9	4.5	4.5	4.5	14.8	10	9.3	14.2	17		Try 60"
"	180	60	1.51	7.55	.9	3.9	4.4	4.4	5.9	10	0.3	7.55	16.5		
"	225	60	2.05	10.3	.9	4.2	4.6	4.6	9.0	10	3.6	10.5	17		

SUMMARY & RECOMMENDATIONS: Velocities read from chart 46, 47 - "Design Charts for Open Channel Flow". (see p. 5-14). Velocities as read from charts are about the same for each size, indicating change in size has little effect. Size selected would be based on accuracy of flood estimate. If 180 c.f.s. is conservative, select 54". Note that TW must be greater than 10.1' for outlet control to govern for 54" pipe flowing 180 c.f.s. This points out that accuracy in estimating TW depths is unnecessary in some cases.

Figure 69. Sample work sheet for culvert dimension determination

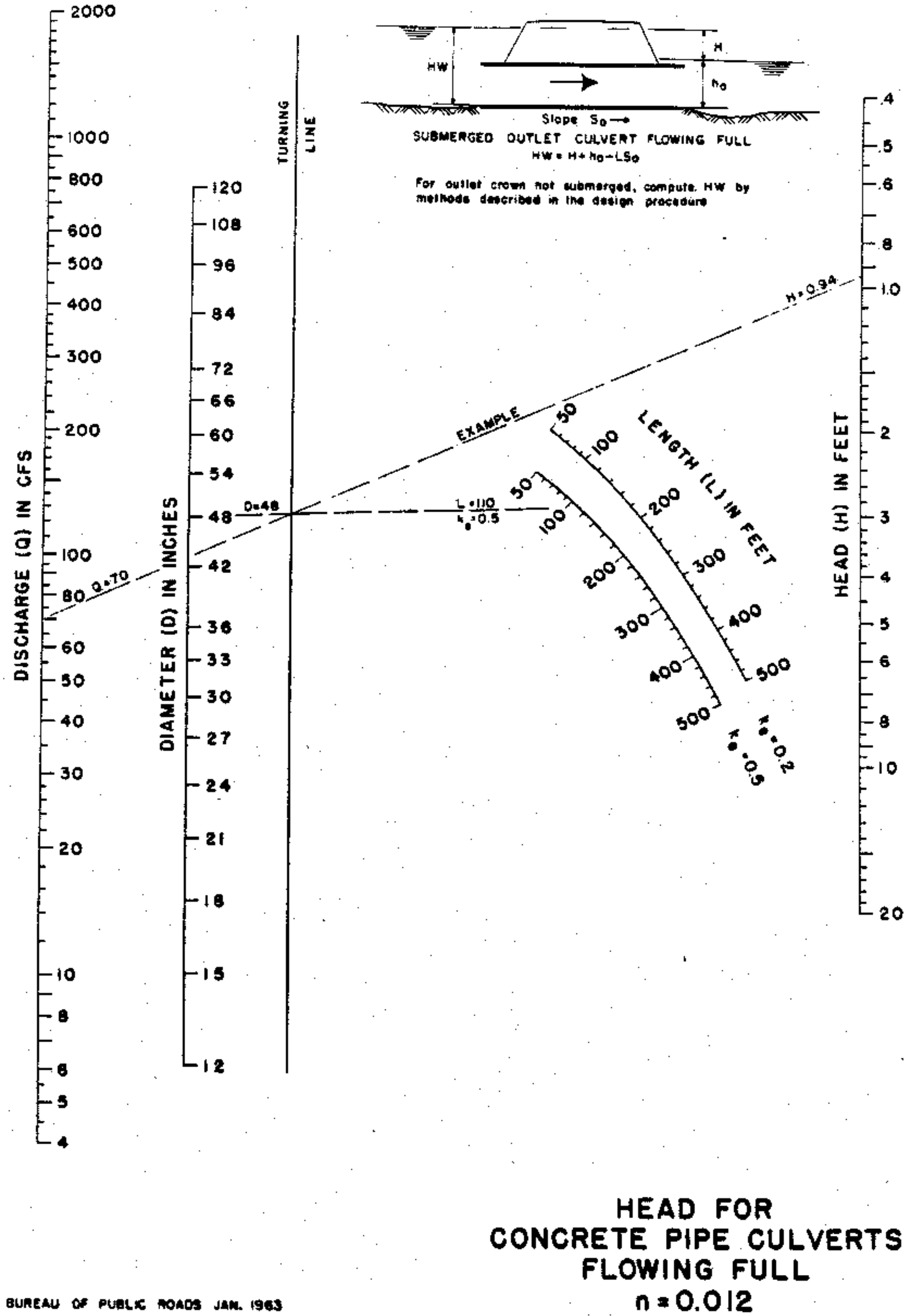


Figure 70. Nomograph for *concrete pipes*, inlet control (U.S. Dept. of Commerce, 1963)

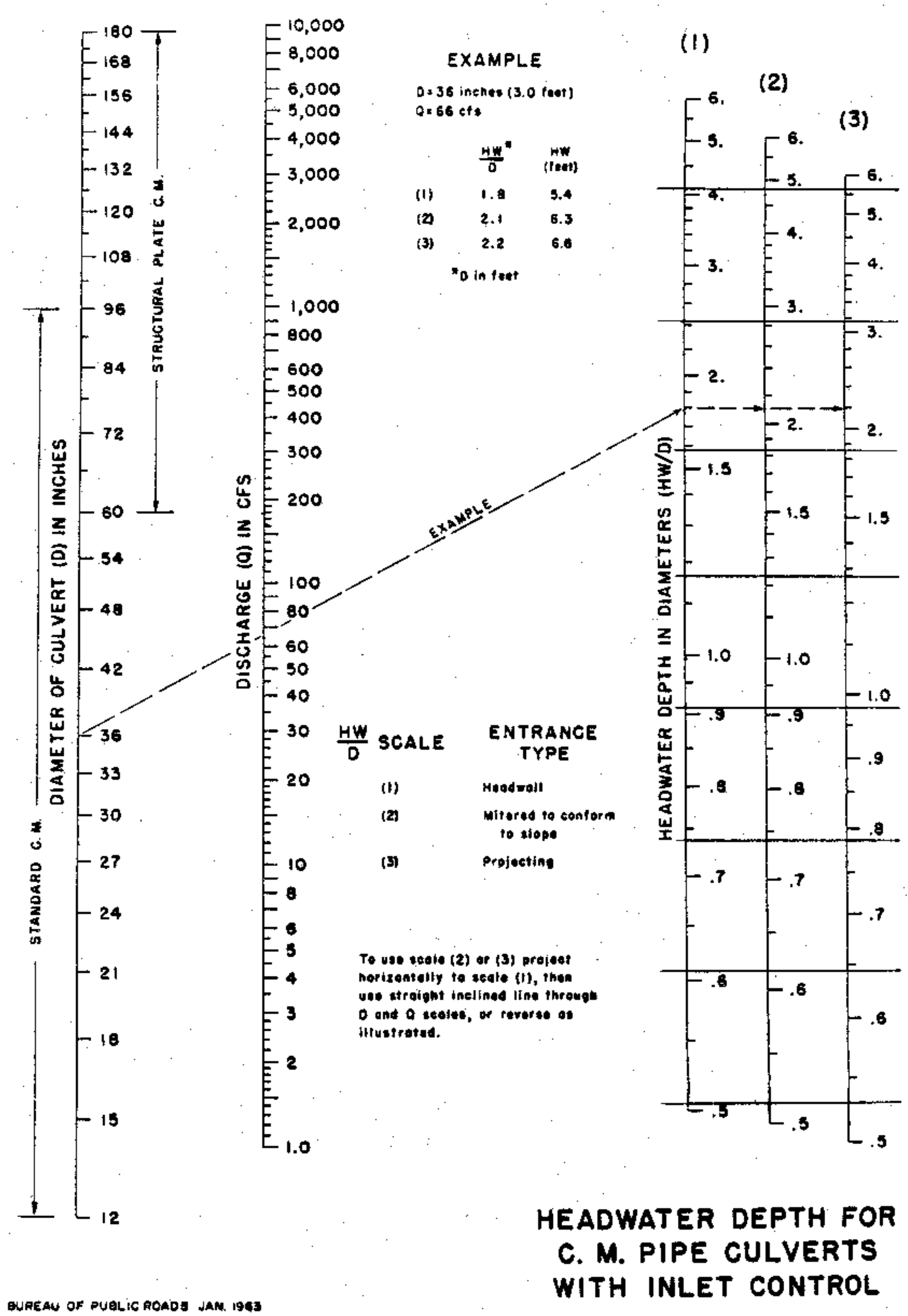


Figure 71. Nomograph for *corrugated metal pipe (CMP)*, inlet control. (U.S. Dept. of Commerce,1963).

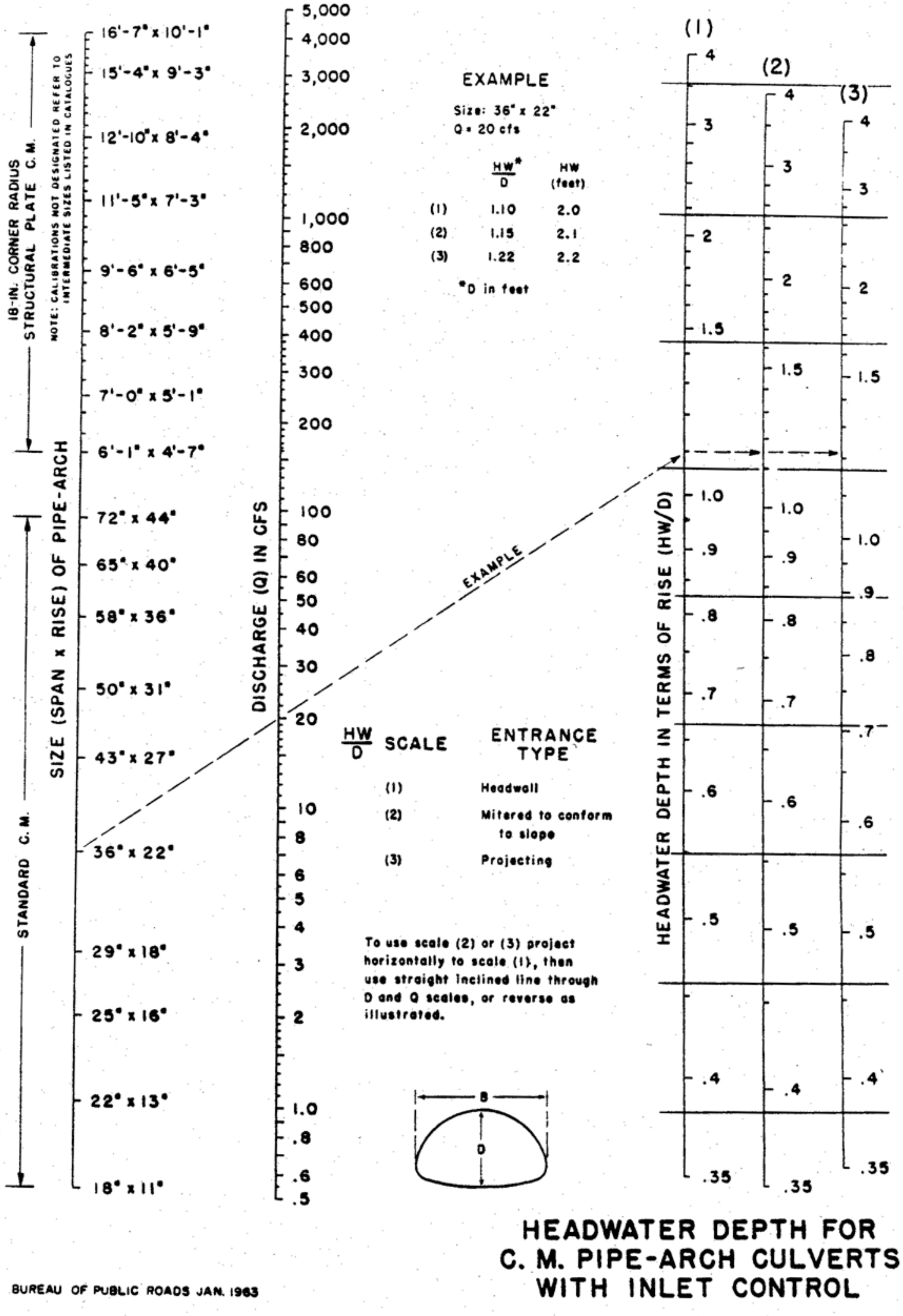


Figure 72. Nomograph for *corrugated metal arch pipe (CMP)*, inlet control. (U.S. Dept. of Commerce, 1963).

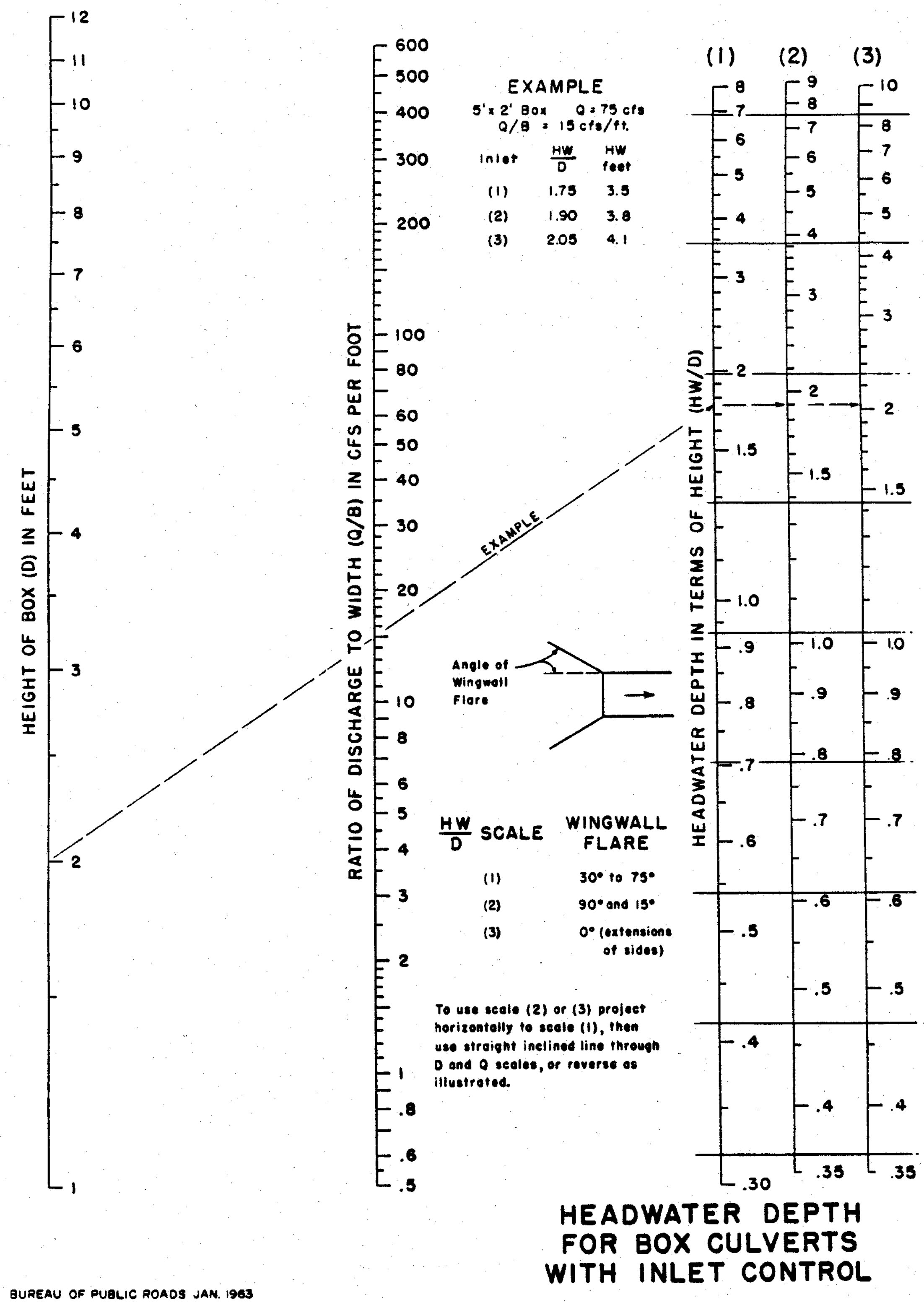


Figure 73. Nomograph for *box - culvert*, inlet control. (U.S. Dept. of Commerce, 1963).

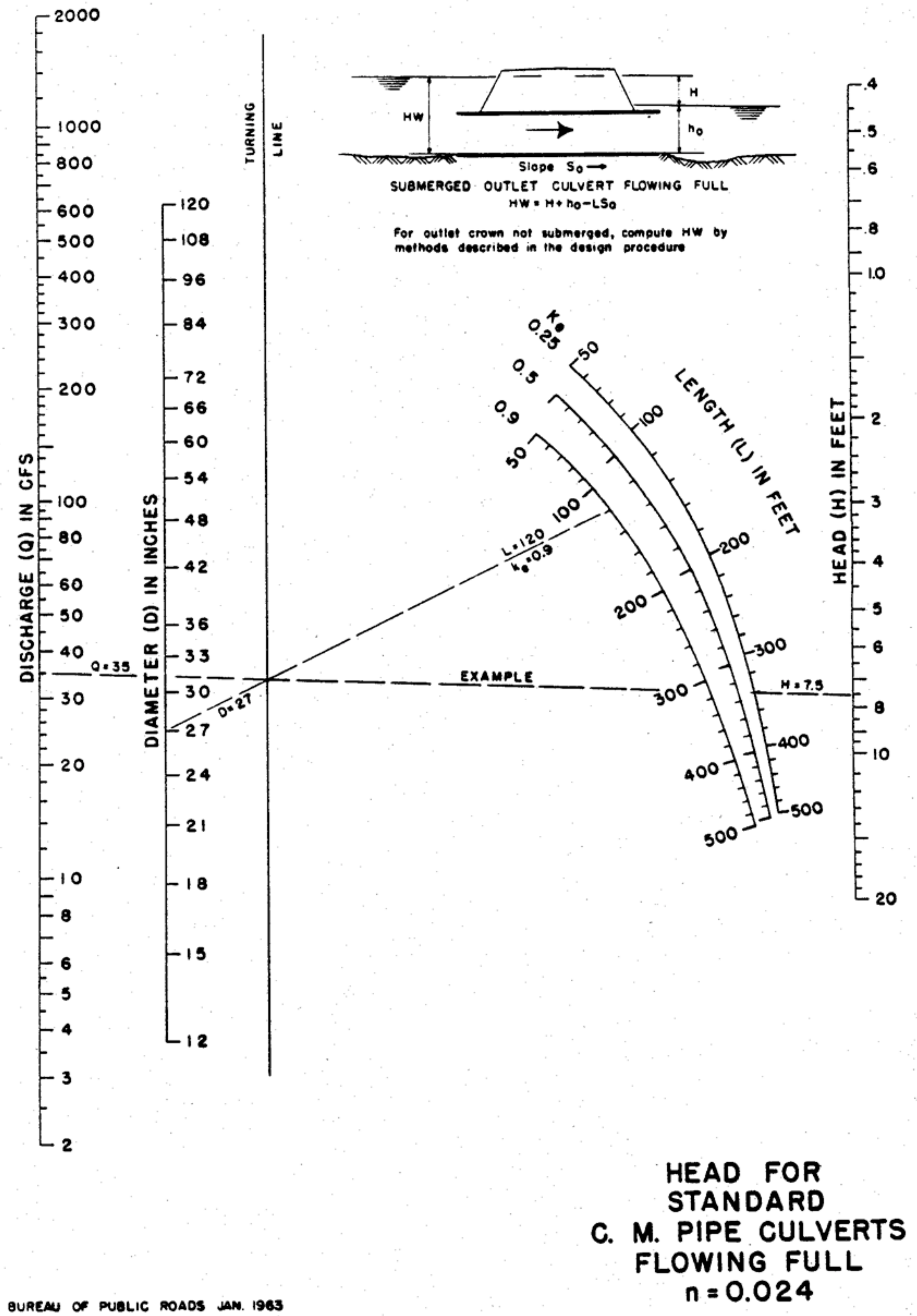


Figure 74. Nomograph for corrugated metal pipe (CMP), *outlet control*. (U.S. Dept. of Commerce, 1963).

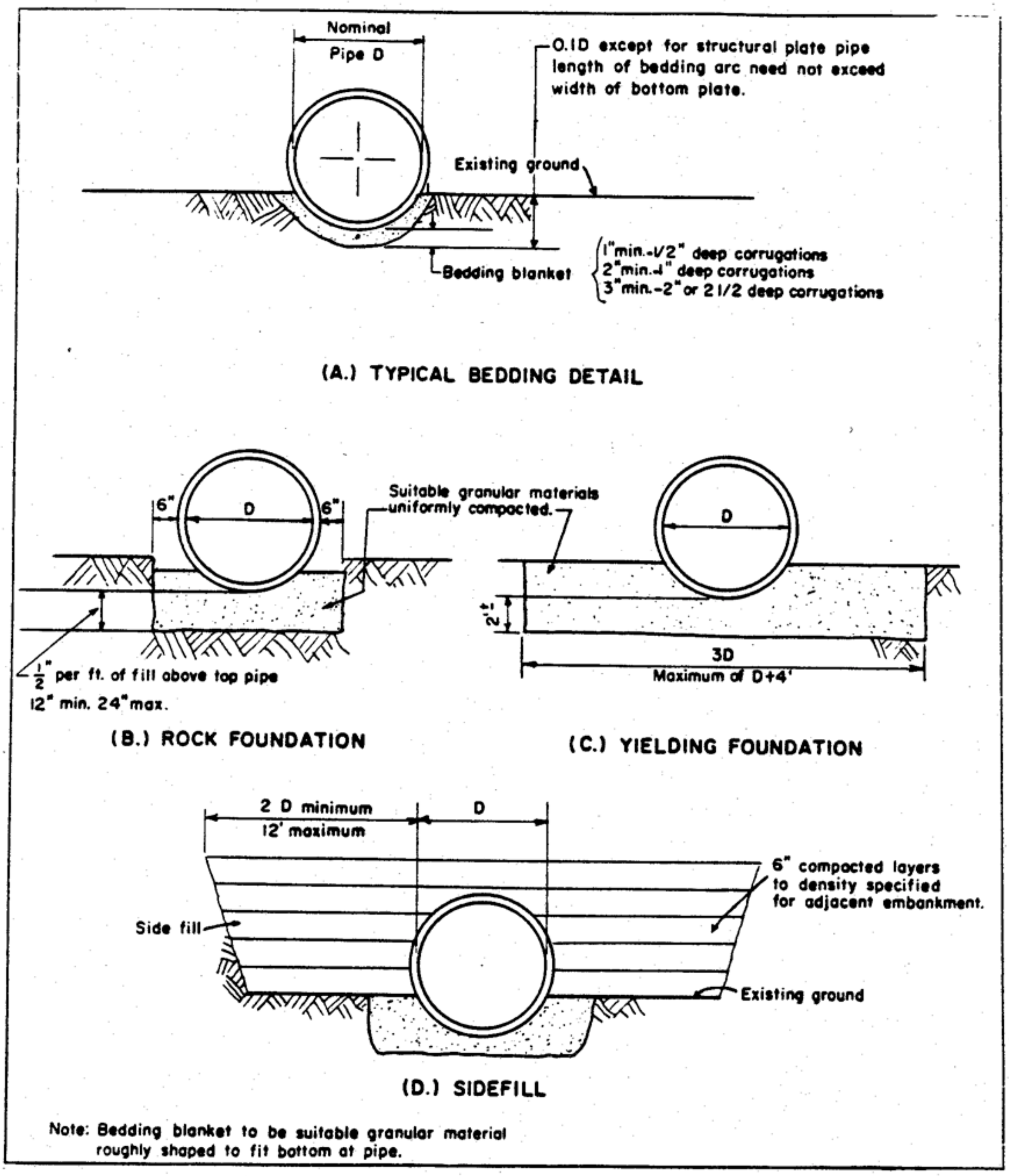


Figure 75. Proper pipe foundation and bedding (1 ft. = 30 cm). (USDA, Forest Service, 1971)

4.3.4 Debris Control Structures

A critical factor in the assessment of channel crossing design and structural capacity is its allowance for handling or passing debris. Past experience has shown that channel crossings have failed not because of inadequate design to handle unanticipated water flows, but because of inadequate allowances for floatable debris which eventually blocked water passage through the culvert. Therefore, each channel crossing has to be analyzed for its debris handing capacity.

When upstream organic debris poses an immediate threat to the integrity of the culvert, several alternatives may be considered.

1. Cleaning the stream of floatable debris is risky and expensive. Since many of the hydraulic characteristics of the channel are influenced by the size and placement of debris, its removal must be carried out only after a trained specialist, preferably a hydrologist, has made a site-specific evaluation of channel stability factors.
2. Various types of mechanical structures (Figures 76, 77 and 78) can be placed above the inlet to catch any debris that may become entrained.
3. A bridge may be substituted in place of a culvert.

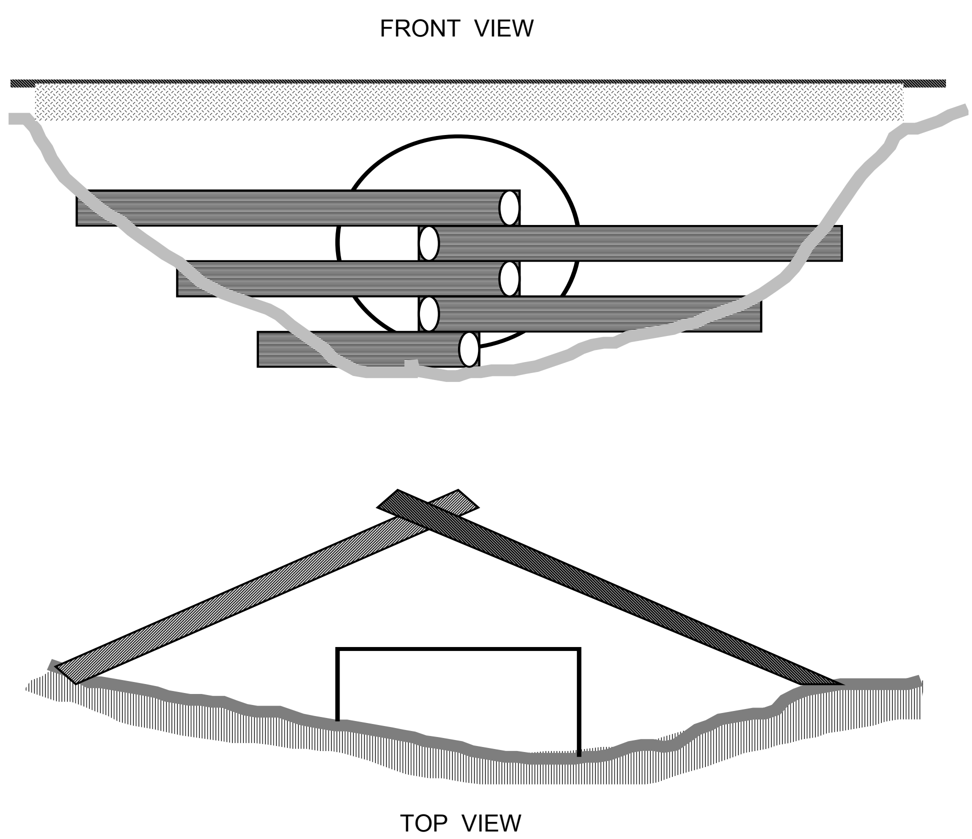


Figure 76. Debris control structure--cribbing made of timber.

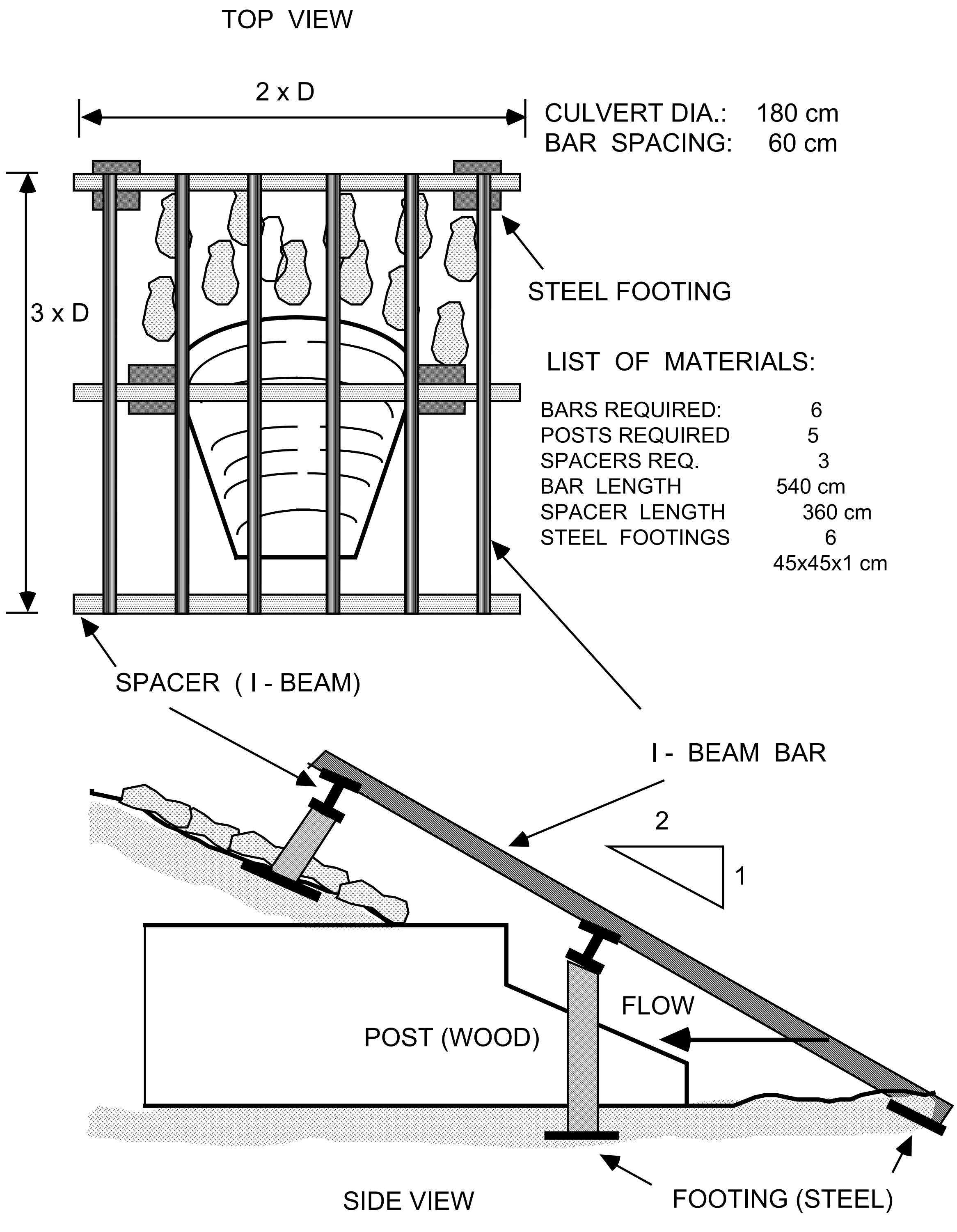


Figure 77. Debris control structure--trash rack made of steel rail (I-beam) placed over inlet.

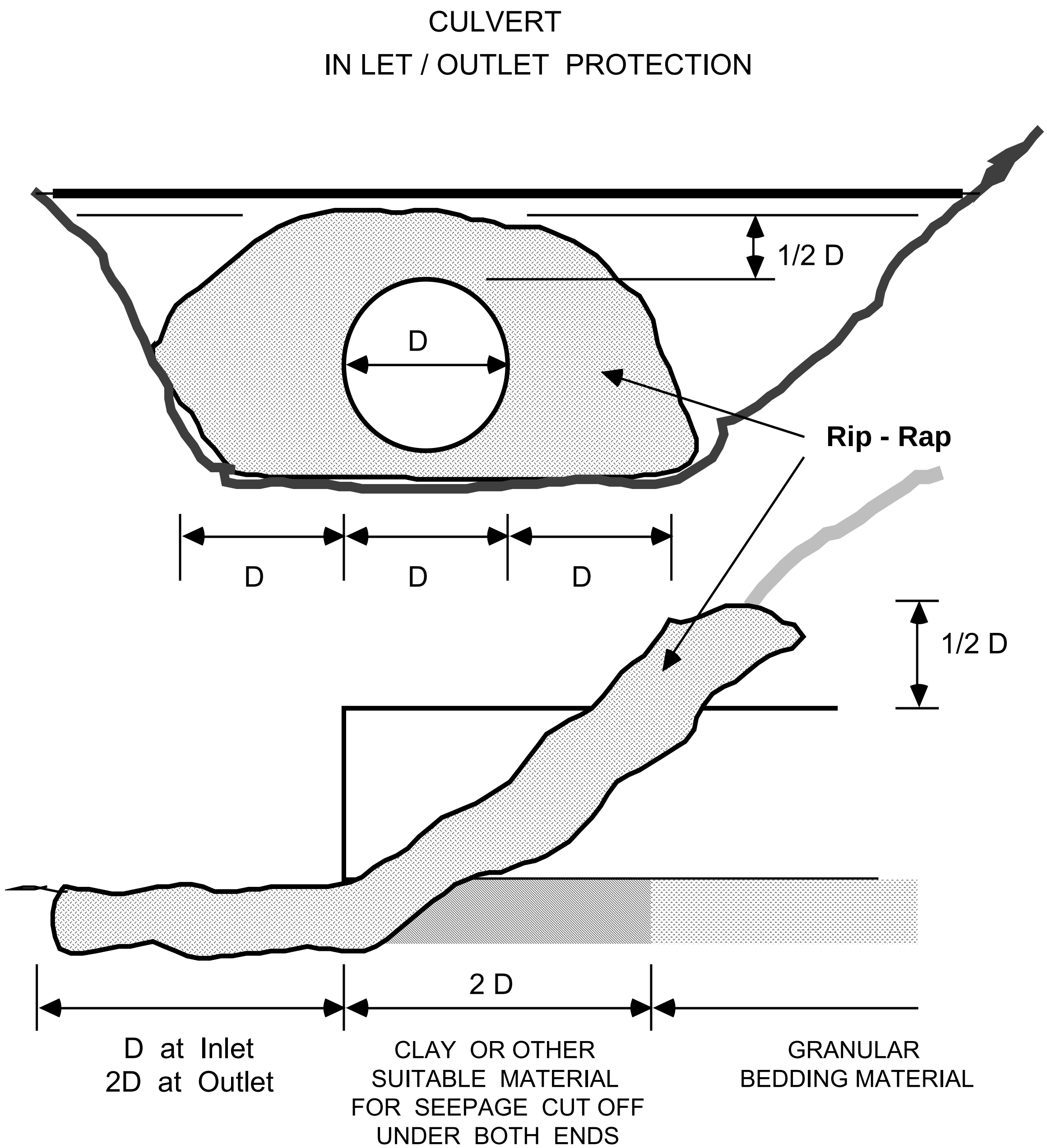


Figure 78. Inlet and outlet protection of culvert with rip-rap. Rocks used should typically weigh 20 kg or more and approximately 50 percent of the rocks should be larger than 0.1 m^3 in volume. Rocks can also be replaced with cemented sand layer (1 part cement, 4 parts sand).

Under high fills, inlets can be provided with upstream protection by rock riprap up to the high water mark (Figure 78). Cambering may also be necessary to ensure the proper grade after fill settlement.

4.3.5 Bridges

Bridges often represent the preferred channel crossing alternative in areas where aquatic resources are extremely sensitive to disturbance. However, poor location of footings, foundations, or abutments can cause channel scour and contribute to debris blockage.

Bridges have been designed using a variety of structural materials for substructure and superstructure. Selection of a bridge type for a specific site should take into consideration the functional requirements of the site, economics of construction at that site, live load requirements, foundation conditions, maintenance evaluations, and expertise of project engineer.

Some arbitrary rules for judging the minimum desirable horizontal and vertical stream clearances in streams not subject to navigation may be established for a specific area based on judgment and experience. In general, vertical clearances should be greater than or equal to 1.5 meters (5 feet) above the 50-year flood level plus 0.02 times the horizontal distance between piers. Horizontal clearance between piers or supports in forested lands or crossings below forested lands should not be less than 85 percent of the anticipated tree height in the forested lands or the lateral width of the 50-year flood. (US Environmental Protection Agency, 1975)

Of course, longer bridge spans will require careful economic evaluations since higher superstructure costs are often involved. Subaqueous foundations are expensive and involve a high degree of skill in the construction of protective cofferdams, seal placement and cofferdam dewatering. In addition to threats to water quality that can occur from a lost cofferdam, time and money losses will be significant. Subaqueous foundations often limit the season of construction relative to water level and relative to fish spawning activity. Thus, construction timing must be rigidly controlled.

It is suggested that the maximum use be made of precast or prefabricated superstructure units since the remoteness of many mountain roads economically precludes bridge construction with unassembled materials that must be transported over great distances. However, the use of such materials may be limited by the capability to transport the units over narrow, high curvature roads to the site, or by the horizontal geometry of the bridge itself.

Another alternative is the use of locally available timber for log stringer bridges. An excellent reference for the design and construction of single lane log bridges is Log Bridge Construction Handbook, by M. M. Nagy, et al., and is published by the Forest Engineering Research Institute of Canada. The reader is referred to this publication for more detailed discussions of these topics.

4.4 Road Surface Drainage

4.4.1 Surface Sloping

Reducing the erosive power of water can achieved by reducing its velocity. If, for practical reasons, water velocity cannot be reduced, surfaces must be hardened or protected as much as possible to minimize erosion from high velocity flows. Road surface drainage attempts to remove the surface water before it accelerates to erosive velocities and/or infiltrates into the road prism destroying soil strength by increasing pore water pressures. This is especially true for unpaved, gravel, or dirt roads.

Water moves across the road surface laterally or longitudinally. Lateral drainage is achieved by crowning or by in- or out-sloping of road surfaces (Figure 79). Longitudinal water movement is intercepted by dips or cross drains. These drainage features become important on steep grades or on unpaved roads where ruts may channel water longitudinally on the road surface.

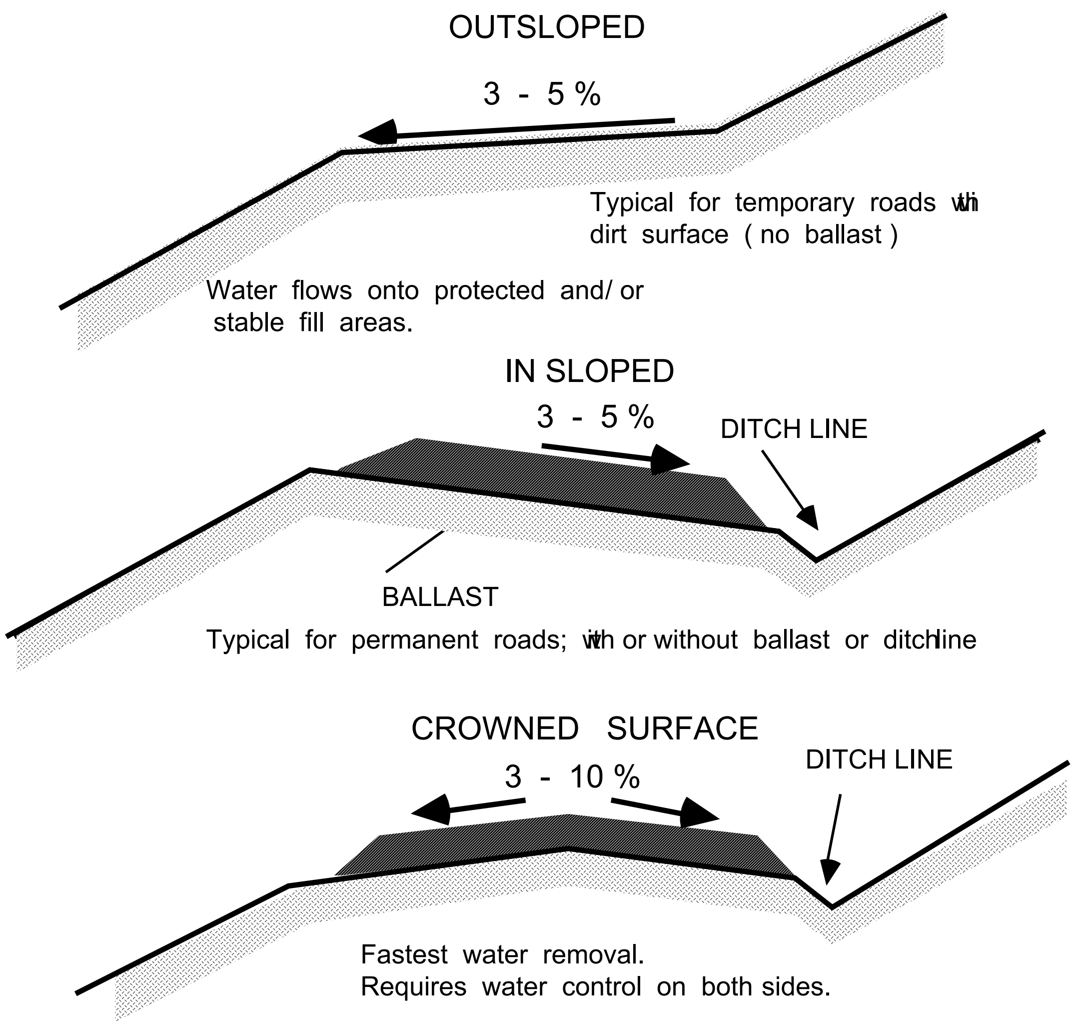


Figure 79. Road cross section grading patterns used to control surface drainage.

table 36 Effect of in-sloping on sediment yield of a graveled, heavily used road segment with a 10 % down grade for different cross slopes*

Transverse grade	Sediment Delivery tonnes/ha/year
conventional 0 - 2 %	970
5 %	400
9 %	300
12 %	260

*4 meter wide road surface
4 - 16 trucks/day
3900 mm annual precipitation

Sloping or crowning significantly reduces sediment delivery from road surfaces. A study by Reid (1981) showed a reduction in sediment delivery by increasing the transverse road surface grade. In this particular case the road surfaces insloped from 5 to 12 percent were compared with conventionally constructed road surfaces at grades of 0 to 2 percent. Sediment yield was reduced by a factor of 3.0 to 4.5 when compared to a conventionally sloped road (Table 30).

Outsloping is achieved by grading the surface at 3 to 5 percent cross slope toward the downhill side of the road. Outsloped roads are simple to build and to maintain. Disadvantages of outsloping include traffic safety concerns and lack of water discharge control. When surfaces become slippery (i.e., snow or ice cover, or when silty or clayey surfaces become wet), vehicles may lose traction and slide toward the downhill edge. Outsloping should only be used under conditions where run-off can be directed onto stable areas. If terrain is less than 20 percent slope and the road gradient is less than 4 percent, outsloping is not an effective way of water removal.

Temporary roads or roads with very light traffic can be outsloped where side slopes do not exceed 40 percent. For safety reasons, when side slopes exceed 40 percent, traffic restrictions should be in force during inclement weather. When outsloping is used for surface drainage, cross drains or dips should be installed on the road surface (Figure 75). Spacing will depend on soil type, road surface and road grade.

Insloping is used where a more reliable drainage system is required such as on permanent roads, roads with high anticipated traffic volumes and/or loads, or in areas with sensitive soils or severe climatic conditions. Insloping is achieved by grading the road surface towards the uphill side of the road at a 3 to 5 percent grade. Water draining from insloped road surfaces is collected and carried along the inside of the road either on the road surface itself or more commonly in a ditch line. The ditch line can be omitted from the road template, thereby reducing the overall road width. This may be desirable in steep terrain in order to reduce excavation (see also Section 3.2). However, this option must be weighed against potential drainage problems along the uphill side of the road. Dips, cross drains, or culverts must be installed and maintained to remove water from the road prism.

Crowned surfaces provide the fastest water removal since the distance water has to travel is cut in half. The crowned surface slopes at 3 to 10 percent from either side of the road centerline. Crowned surfaces and any associated cross drains or dips are difficult to maintain. Water has to be controlled on both sides of the road through a ditch line and stable areas have to be provided for runoff water. Ballast thickness is typically the largest in the center in order to achieve the correct crown shape.

4.4.2 Surface Cross Drains

Cross drains are often needed to intercept the longitudinal, or down-road, flow of water in order to reduce and/or minimize surface erosion. In time, traffic will cause ruts to form, channeling surface water longitudinally down the road. Longitudinal or down-road flow of water becomes increasingly important with:

- increasing grades
- rutting frequency
- road surface protection

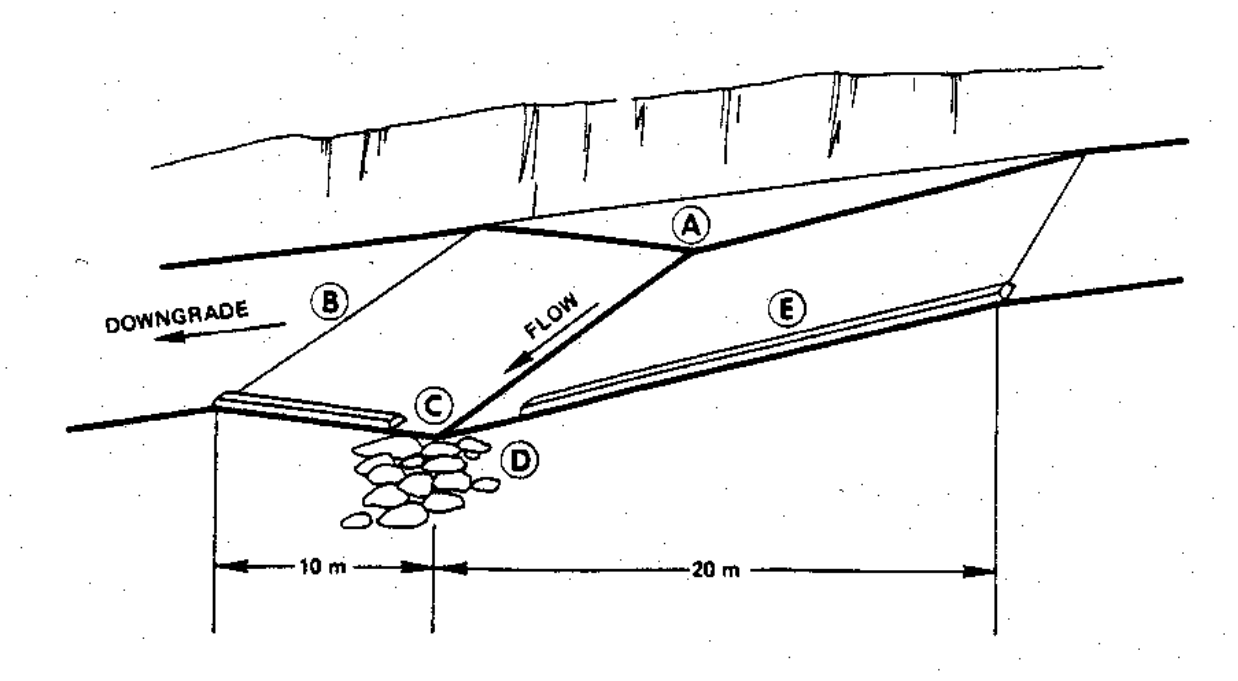


Figure 80. Design of outsloped dips for forest roads. A to C, slope about 10 to 15 cm to assure lateral flow; B, no material accumulated at this point - may require surfacing to prevent cutting; D, provide rock rip-rap to prevent erosion; E, berm to confine outflow to 0.5 m wide spillway. (Megahan, 1977)

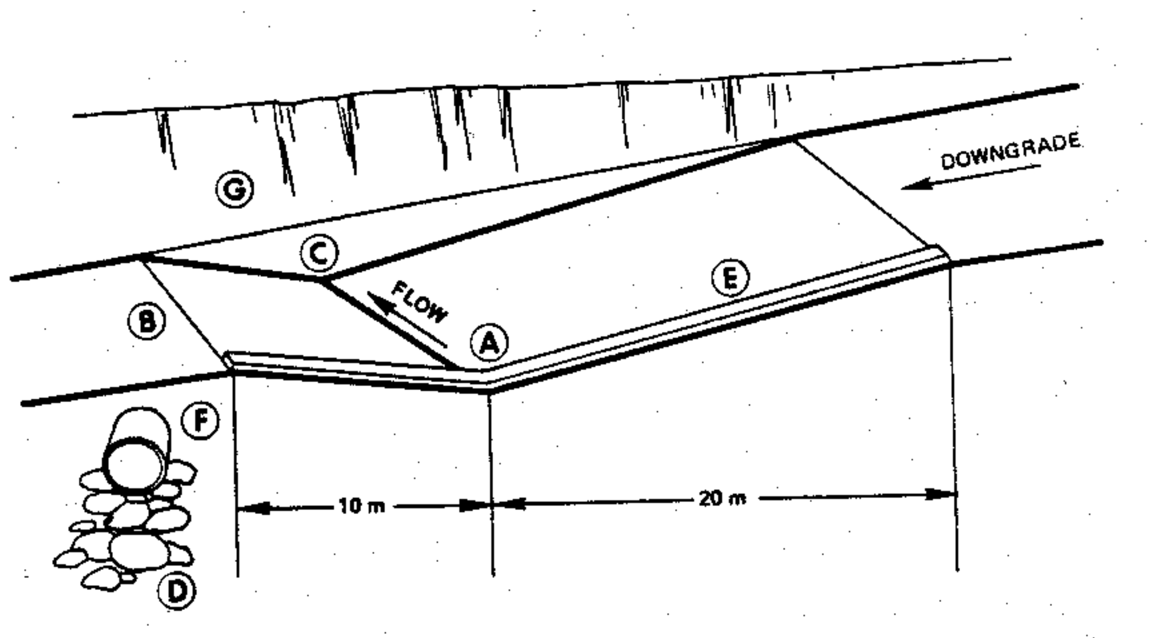


Figure 81. Design of insloped dips for forest roads. A to C, slope about 10 to 15 cm to assure lateral flow; B, no material accumulated at this point - may require surfacing to prevent cutting; D, provide rock rip-rap to prevent erosion; E, berm to prevent overflow; F, culvert to carry water beneath road; G, widen for ditch and pipe inlet (Megahan, 1977).

There are three types of cross drains used for intercepting road surface water: intercept-ing or rolling dips, open top culverts, and cross ditches. Cross drains serve a dual purpose. First they must intercept longitudinal road surface flow, and second they must carry ditch water across the road prism at a frequency interval small enough to prevent concentration of flow. Ditch relief is discussed in more detail in section 4.4.3 and 4.4.4.

Intercepting dips (Figures 80 and 81) when properly constructed, are cheaper to maintain and more permanent than open-top culverts. However, their usefulness is limited to road grades less than 10 percent. At steeper grades, they become difficult to construct and maintain.

Dip locations are determined at the time the grade line is established on the ground or during vertical alignment design. The total length of the two vertical curves comprising the dip should be sufficient to allow the design vehicle to pass safely over them at the design speed. The minimum vertical distance between the crest and sag of the curves should be at least 30 cm (1 ft). It is important that the dip be constructed at a 30 degree or greater angle downgrade and that the dips have an adverse slope on the downroad side. The downroad side of the dip should slope gently downward from the toe of the road cut to the shoulder of the fill. The discharge point of the dip should be armored with rock or equipped with a down-drain to prevent erosion of the fill. Equipment operators performing routine maintenance should be aware of the presence and function of the dips so that they are not inadvertently destroyed.

Open top culverts are most effective on steeper road grades. Open top culverts (Figure 82) can be made of durable treated lumber or poles or they may be prefabricated from corrugated, galvanized steel. The trough should be 7 to 10 cm (3 to 4 in) wide and from 10 to 20 cm (4 to 8 in) deep. The gradient required in order for open top culverts to be self cleaning is 4 percent or greater and, as with dips, they should be angled 30 degrees downslope. In order to maintain their functionality they should be inspected and cleaned on a frequent and regular basis.

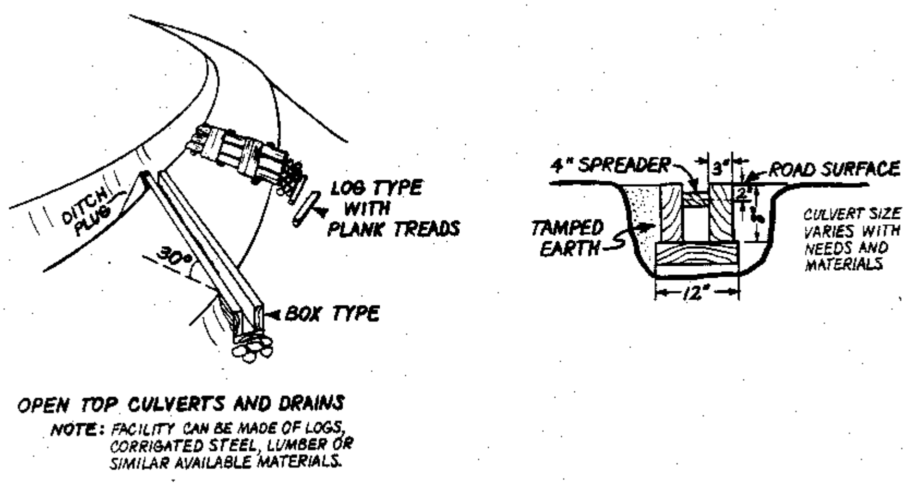


Figure 82. Installation of an open-top culvert. Culverts should be slanted at 30 degrees downslope to help prevent plugging. Structure can be made of corrugated steel, lumber or other, similar material. (Darrach, et al., 1982)

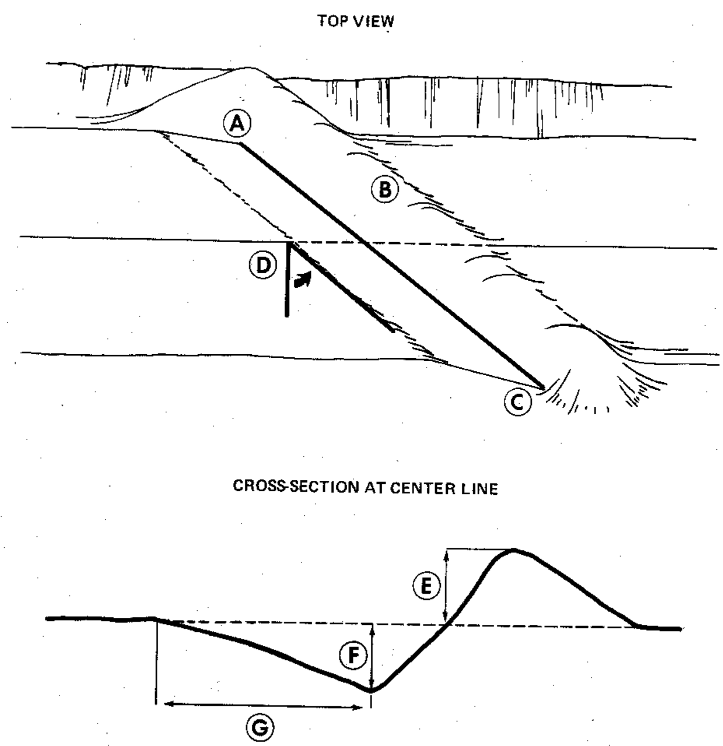


Figure 83. Cross ditch construction for forest roads with limited or no traffic. Specifications are generalized and may be adjusted for gradient and other conditions. A, bank tie-in point cut 15 to 30 cm into roadbed; B, cross drain berm height 30 to 60 cm above road bed; C, drain outlet 20 to 40 cm into road; D, angle drain 30 to 40 degrees downgrade with road centerline; E, height up to 60 cm, F, depth to 45 cm; G, 90 to 120 cm. (Megahan, 1977)

Cross ditches, or water bars, are typically used on temporary roads. They are the easiest and most inexpensive method for cross drain installation (Figure 83). However, they impede traffic, wear out quickly, and are difficult to maintain and are, therefore, not recommended except on very low standard roads. In order to be effective, the cross ditch should be excavated into the mineral soil or subgrade and not just into the dirt or surface layer. Water bars should be installed at a 30 degree angle to the centerline of the road, and ditch and berm should be carefully extended to the cut bank in order to avoid ditch water bypass. A berm should be placed in the cut bank ditch to divert water into the cross ditch. Care should be taken that the berm and ditch is not beaten or trampled down by traffic or livestock.

Spacing requirements for surface cross drains depend on road grade, surfacing material, rain intensities, and slope and aspect. Spacing guides for surface cross drains are given in Table 31.

table 37 Cross drain spacing required to prevent rill or gully erosion deeper than 2.5 cm on unsurfaced logging roads built in the upper topographic position 1/ of north-facing slopes 2/ having gradient of 80 % 3/ (Packer, 1967).

Road Grade (%)	Material: Hard sediment	Basalt	Granite	Glacial silt	Andesite	Loess
	Cross drain spacing, m					
2	51	47	42	41	32	29
4	46	42	38	37	27	24
6	44	40	35	34	25	22
8	42	38	33	32	23	20
10	39	35	29	29	20	17
12	36	32	27	27	17	15
14	33	29	24	23	14	11

1. In middle topographic position, reduce spacing 5.5 m; in lower topographic position, reduce spacings 11m.
2. On south aspects, reduce spacings 4.6m.
3. For each 10% decrease in slope steepness below 80%, reduce spacing 1.5m.

A Japanese open-top culvert spacing guide uses road grade as input (Figure 84) On steep grades, spacing is similar to data given in Table 30. However, on gentler grades (2 - 8%), the Japanese spacing guide provides for considerably wider spacings. This is a good illustration of a case where local conditions take precedent over general guidelines developed for large geo-graphical areas.)

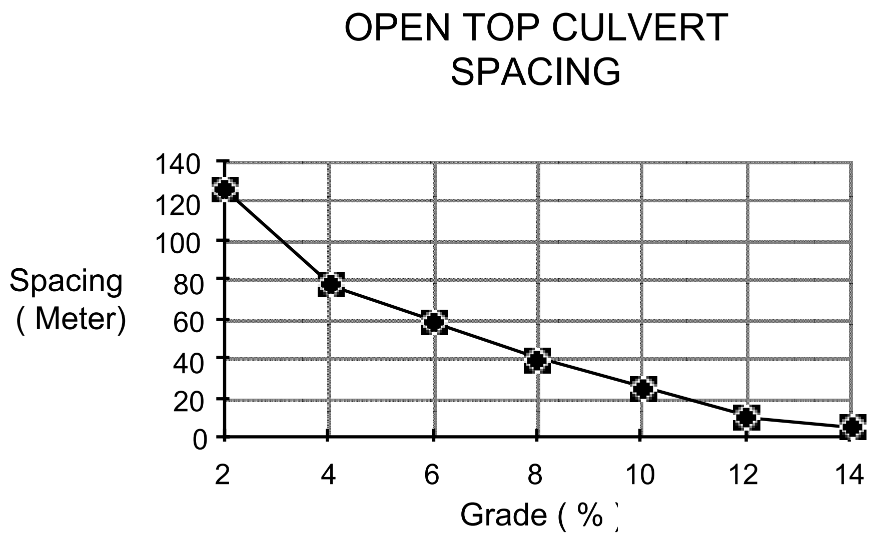


Figure 84. Spacing standard for open-top culverts on forest road surfaces, Japanese Islands. (Minematsu and Minamikata,1983)

Equal attention must be given to location of cross drains in relation to road and topographic features. Natural features such as slope breaks or ideal discharge locations that disperse water should be identified and incorporated into the drainage plan as needed. Possible locations for cross drains are shown in Figure 85.

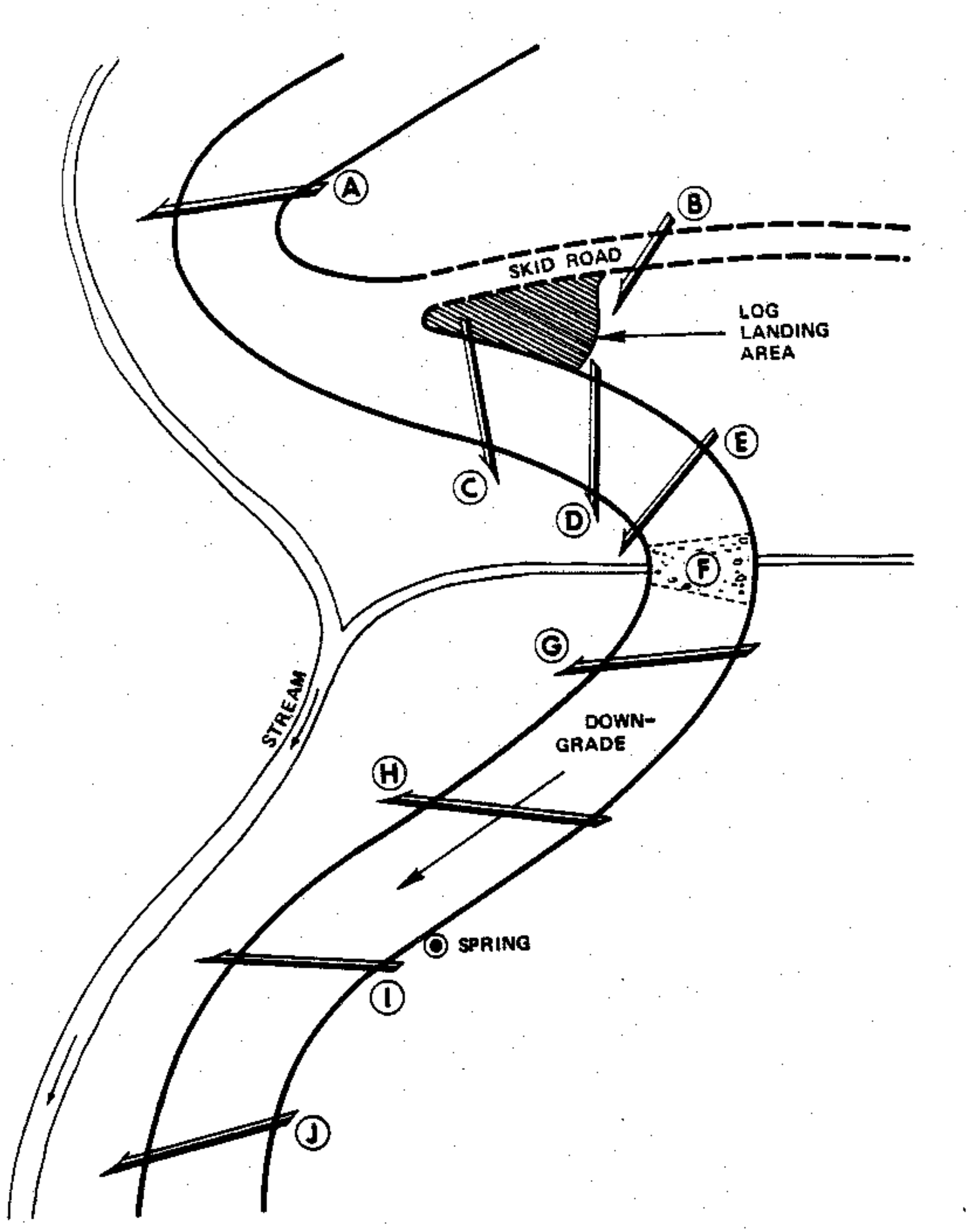


Figure 85. Guide for locating cross drains. Several locations require cross drains independent of spacing guides. A and J, divert water from ridge; A, B, and C, cross drain above and below junction; C and D, locate drains below log landing areas; D and H, drains located with regular spacing; E, drain above incurve to prevent bank cutting and keep road surface water from entering draw; F, ford or culvert in draw; G, drain below inside curve to prevent water from running down road; I, drain below seeps and springs. (Megahan, 1977)

4.4.3 Ditches and Berms

Ditches and berms serve two primary functions on upland roads: (1) they intercept surface run-off before it reaches erodible areas, such as fill slopes, and (2) they carry run-off and sediment to properly designed settling basins during peak flow events (when circumstances warrant the use of settling basins). Ditches and berms are commonly located at the top of cut and fill slopes and adjacent to the roadway, although midslope berms may be useful in controlling sediment on cut and fill slopes before erosion control cover has been established.

The required depth and cross sectional area of a roadside ditch is determined by the slope of the ditch, area to be drained, estimated intensity and volume of run-off, and the amount of sediment that can be expected to be deposited in the ditch during periods of low flow. Triangular or trapezoidal-shaped ditches may be utilized, whichever is appropriate. The ditch cross section is designed so that it will produce the desired water velocity for a given discharge. Minimum full capacity flow velocities should be 0.76 to 0.91 meters/second (2.5 to 3 feet/second) to permit sediment transport. It is best to remember that, in shaping a ditch, given equal grade and capacity, a wide, shallow cross section will generate lower water velocities with correspondingly lower erosion potential than will a narrow, deep cross section. Maximum permissible velocities for unlined ditches of a given soil type are listed in Table 32.

table 38 Maximum permissible velocities in erodible channels, based on uniform flow in straight, continuously wet, aged channels. For sinuous channels, multiply allowable velocity by 0.95 for slightly sinuous, 0.9 for moderately sinuous, and 0.8 for highly sinuous channels. (U. S. Environmental Protection Agency, 1975)

	Maximum permissible velocities (m/s)		
	Clear water	Water carrying fine silt	Water carrying sand and gravel
Fine sand (noncolloidal)	0.46	0.76	0.46
Sandy loam (noncolloidal)	0.52	0.76	0.61
Silt loam (noncolloidal)	0.61	0.91	0.61
Ordinary firm loam	0.76	1.07	0.67
Volcanic ash	0.76	1.07	0.61
Fine gravel	0.76	1.52	1.13
Stiff clay (very colloidal)	1.13	1.52	0.91
Graded, loam to cobbles (noncolloidal)	1.13	1.52	1.52
Graded, silt to cobbles (colloidal)	1.22	1.68	1.52
Alluvial silts (noncolloidal)	0.61	1.07	0.61
Alluvial silts (colloidal)	1.13	1.52	0.91
Coarse gravel (noncolloidal)	1.22	1.83	1.98
Cobbles and shingles	1.52	1.68	1.98
Shales and hardpans	1.83	1.83	1.52

Ditch lining				Manning's n	V_{max}[11] ft/sec	meters/sec
1. Natural earth						
a. Without vegetation						
1) Rock						
a) Smooth and uniform				0.035 - 0.040	20	6.0
b) Jagged & irregular				0.040 - 0.045	15 - 18	4.5 - 5.4
2) Soils						
	Unified	USDA				
Coarse grained — Gravel and gravelly soils	GW	Gravel		0.022 - 0.024	6 - 7	1.8 - 2.1
Coarse grained — Gravel and gravelly soils	GP	Gravel		0.023 - 0.026	7 - 8	2.1 - 2.4
Coarse grained — Gravel and gravelly soils	GM	Loamy Gravel	d	0.023 - 0.025	3 - 5	0.9 - 1.5
Coarse grained — Gravel and gravelly soils	GM	Loamy Gravel	u	0.022 - 0.020	2 - 4	0.6 - 1.2
Coarse grained — Gravel and gravelly soils	GC	Gravelly Loam, Gravelly Clay		0.024 - 0.026	5 - 7	1.5 - 2.1
Coarse grained — Sand and sandy soils	SW	Sand		0.020 - 0.024	1 - 2	0.3 - 0.6
Coarse grained — Sand and sandy soils	SP	Sand		0.022 - 0.024	1 - 2	0.3 - 0.6
Coarse grained — Sand and sandy soils		Loamy Sand	d	0.020 - 0.023	2 - 3	0.6 - 0.9
Coarse grained — Sand and sandy soils		Loamy Sand	u	0.021 - 0.023	2 - 3	0.4 - 0.9
Coarse grained — Sand and sandy soils	SC	Sandy Loam		0.023 - 0.025	3 - 4	0.9 - 1.2
Fine grained — Silts and clays, LL 50	CL	Clay Loam, Sandy Clay Loam, Silty Clay		0.022 - 0.024	2 - 3	0.6 - 0.9
Fine grained — Silts and clays, LL 50	ML	Silt Loam, Very Fine Sand, Silt		0.023 - 0.024	3 - 4	0.9 - 1.2
Fine grained — Silts and clays, LL 50	OL	Mucky Loam		0.022 - 0.024	2 - 3	0.6 - 0.9
Fine grained — Silts and clays, LL 50	CH	Clay		0.022 - 0.023	2 - 3	0.6 - 0.9
Fine grained — Silts and clays, LL 50	MH	Silty Clay		0.023 - 0.024	3 - 5	0.9 - 1.5
Fine grained — Silts and clays, LL 50	OH	Mucky Clay		0.022 - 0.024	2 - 3	0.6 - 0.9
Highly Organic	PT	Peat		0.022 - 0.025	2 - 3	0.6 - 0.9
2. With vegetation						
a. Average turf						
1) Erosion resistant soil				0.050 - 0.070	4 - 5	1.2 - 1.5
2) Easily eroded soil				0.030 - 0.050	3 - 4	0.9 - 1.2
b. Dense turf						
1) Erosion resistant soil				0.070 - 0.090	6 - 8	1.8 - 2.4
2) Easily eroded soil				0.040 - 0.050	5 - 6	1.5 - 1.8
c. Clean bottom with bushes on sides				0.050 - 0.080	4 - 5	1.2 - 1.5
d. Channel with tree stumps						
1) No sprouts				0.040 - 0.050	5 - 7	1.5 - 2.1
2) With sprouts				0.060 - 0.080	6 - 8	1.8 - 2.4
e. Dense woods				0.080 - 0.120	5 - 6	1.5 - 1.8
f. Dense brush				0.100 - 0.140	4 - 5	1.3 - 1.5
g. Dense willows				0.150 - 0.200	8 - 9	2.4 - 2.7
3. Paved				(Construction)		
a. Concrete, w/all surfaces:				Good Poor		
1) Trowel finish				0.012 - 0.014	20	6.0
2) Float finish				0.013 - 0.015	20	6.0
3) Formed, no finish				0.014 - 0.016	20	6.0
b. Concrete bottom, float finished, w/sides of:						
1) Dressed stone in mortar				0.015 - 0.017	18 - 20	5.4 - 6.0
2) Ramdom stone in mortar				0.017 - 0.020	17 - 19	5.1 - 5.7
3) Dressed stone or smooth concrete rubble (riprap)				0.020 - 0.025	15	4.5
4) Rubble or random stone (riprap)				0.025 - 0.030	15	4.5
c. Gravel bottom, sides of:						
1) Formed concrete				0.017 - 0.020	10	3.0
2) Random stone in mortar				0.020 - 0.023	8 - 10	2.4 - 3.0
3) Random stone or rubble (riprap)				0.023 - 0.033	8 - 10	2.4 - 3.0
d. Brick				0.014 - 0.017	10	3.0
3. Asphalt				0.013 - 0.016	18 - 20	5.4 - 6.0

[11] Maximum recommended velocities

table 39 Manning's n for open ditches.

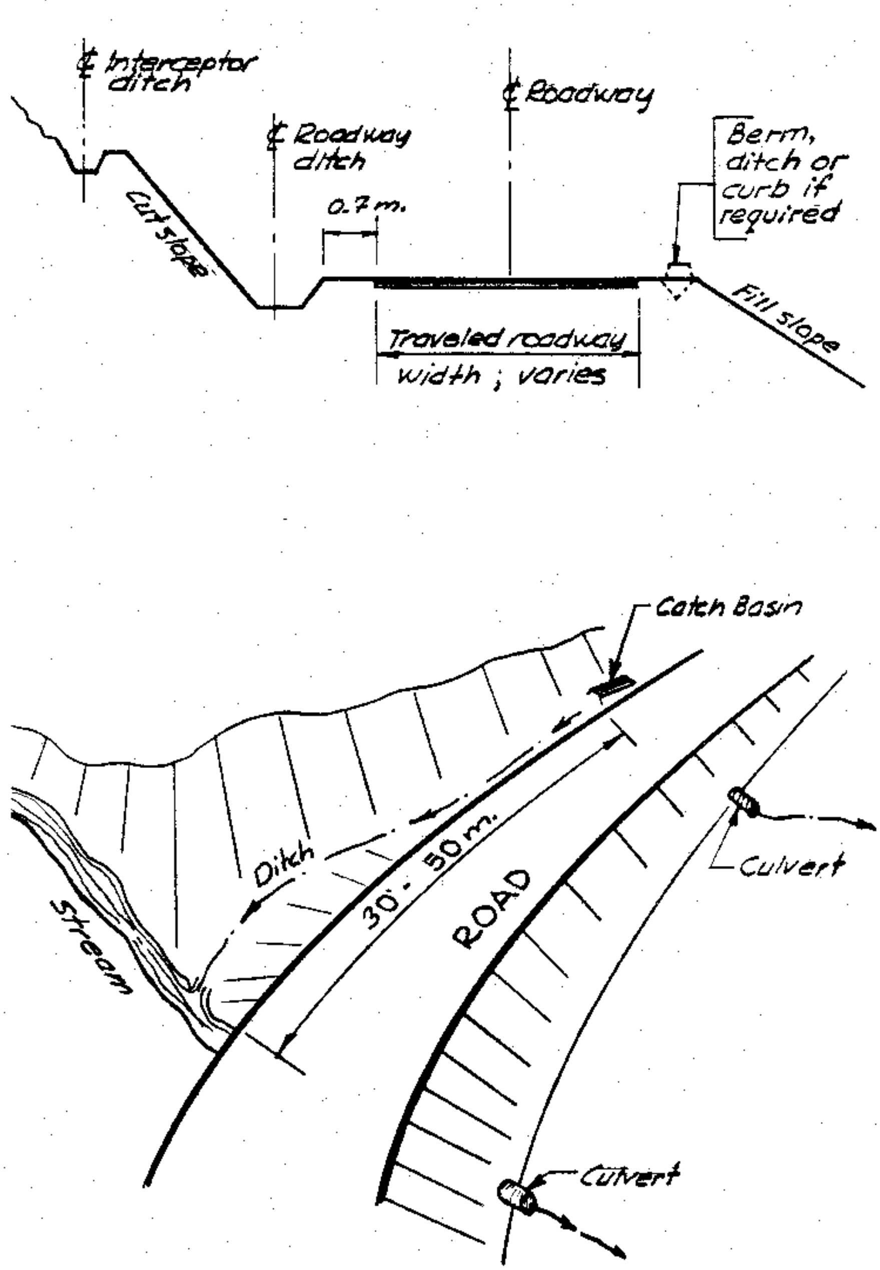


Figure 86. Ditch interception near stream to divert ditch water onto stable areas instead of into the stream. (U. S. Environmental Protection Agency,1975)

The procedure for calculating flow rates is the same as that discussed in Section 4.2. The corresponding roughness factors (Manning's n) for open channels are given in Table 33. Ditches in highly erodible soils may require riprap, rock rubble lining, jute matting, or grass seeding. Riprap or rubble-lined ditches will tend to retard flow enough to allow water movement while retaining the sediment load at low flow periods. Lining ditches can reduce erosion by as much as 50 percent and may provide economical benefits by reducing the required number of lateral cross drains when materials can be obtained at low cost.

Ditch water should not be allowed to concentrate, nor should it be allowed to discharge directly into live streams. A cross drain such as a culvert should carry the ditch water across and onto a protected surface (Figure 81). Spacing of ditch relief culverts is discussed in Section 4.4.4 and 4.5.

The ditch grade will normally follow the roadway grade. However, the minimum grade for an unpaved ditch should be 1 percent. Runoff intensity or discharge values needed to calculate ditch size can be determined by calculations described below for culvert design. However, allowances should be made for sedimentation, plus at least 0.3 m between the bottom of the roadway subgrade and the full flow water surface. The suggested minimum size of roadside ditches is shown in Figure 87.

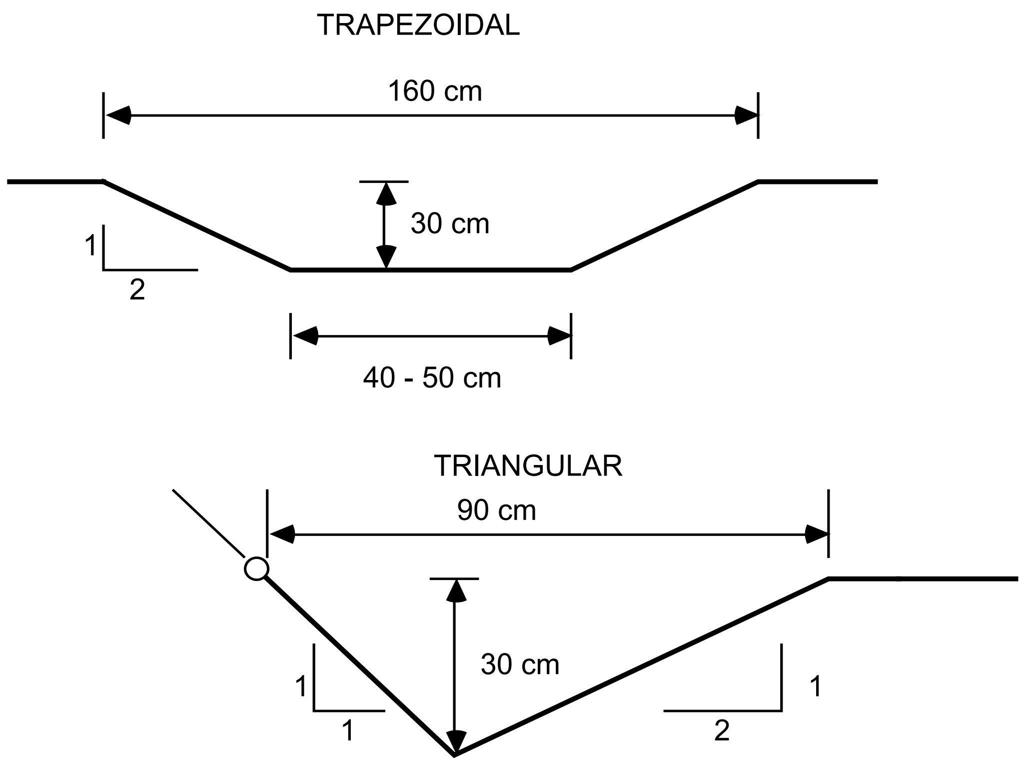


Figure 87. Minimum ditch dimensions.

Velocity of the ditch water is a function of cross section, roughness and grade. For a typical triangular cross section the velocity can be calculated from Manning's equation:

$$V = n^{-1} * R^{2/3} * S^{1/2}$$

where V equals velocity in meters/second and the other values are as defined in Chapter 4.2. For a triangular channel with sideslopes of 1:1 and 2:1, flowing 0.3 meters deep, the hydraulic radius, R, equals 0.12 m. Table 34 lists ditch velocities as a function of roughness coefficients and grade, and Figure 88 provides a nomograph for the solution of Manning's equation.

In most cases ditch lines should be protected to withstand the erosion. For channels with grade steeper that 10 percent, a combination of cross section widening, surface protection and increased surface roughness may be required.

table 40 Ditch velocities for various n and grades. Triangular ditch with side slope ratio of 1:1 and 2:1, flowing 0.30 meters deep; hydraulic radius R=0.12.

Slope (%)	n		
	0.02	0.03	0.04
	meters/sec		
2	1.7	1.2	0.9
4	2.5	1.6	1.2
6	3.0	2.0	1.5
8	3.5	2.3	1.7
10	3.9	2.6	1.9
12	4.3	2.9	2.1
15	4.8	3.2	2.4
18	5.3	3.5	2.6

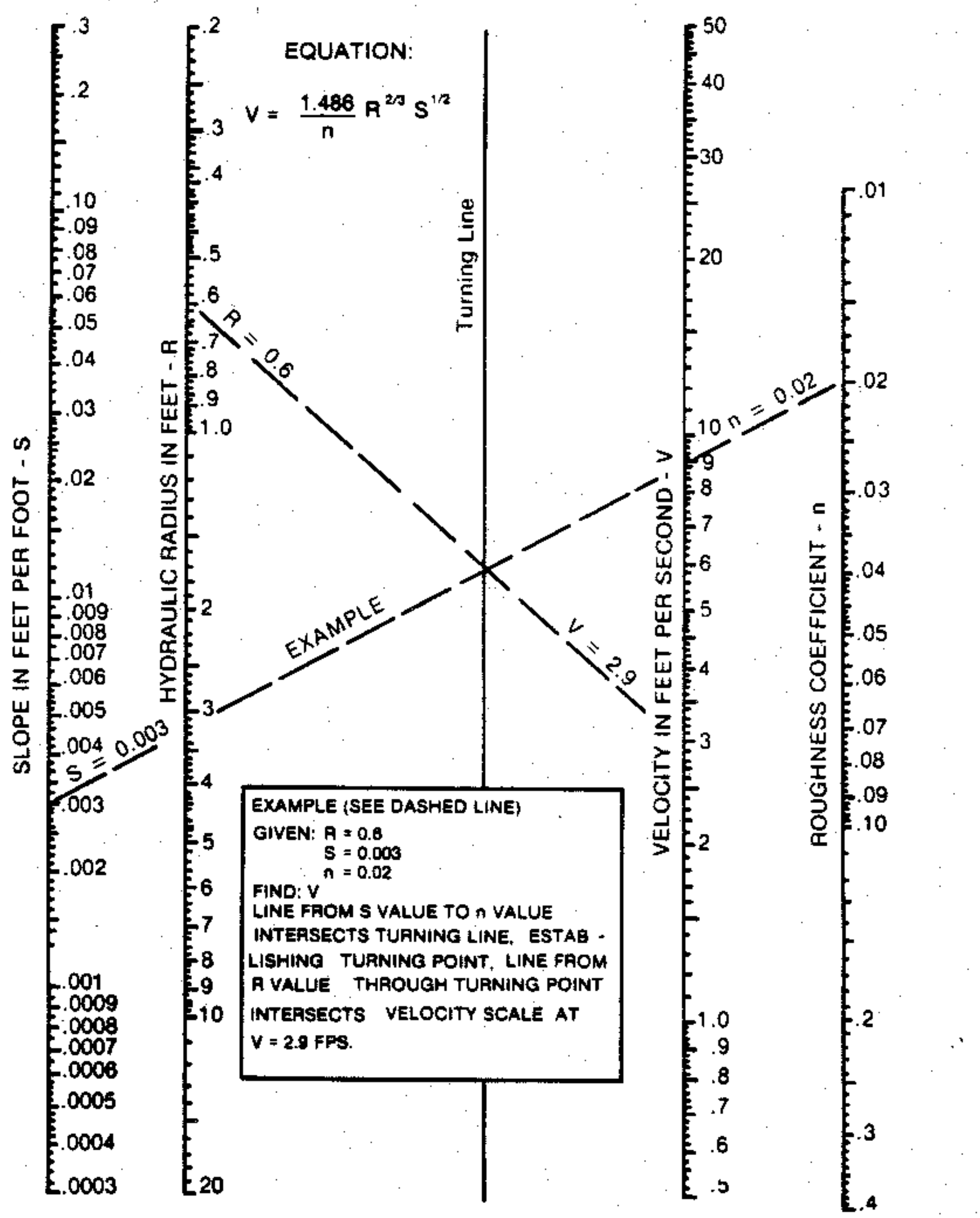


Figure 88. Nomograph for solution of Manning's equation (U.S. Dept. of Commerce, 1965)

EXAMPLE:

Determine whether the water velocity for a road ditch will be below critical levels for erosion. If velocities are too high, make and evaluate changes (see also U.S. Forest Service, 1980). Ditch dimension is a symmetrical, triangular channel, 0.39 m deep with 2.5:1 slopes with sandy banks (SW) and a slope of 0.003 m/m.

Solution:

1. The hydraulic radius, R, is equal to area divided by wetted perimeter.

$$R = 0.38\ m^2/2.1\ m = 0.18\ m$$

Converting to english units, divide meters by 0.3 m/ft.

$$R = 0.60\ ft$$

2. Obtain roughness coefficient from Table 32 (n = 0.020).

3. Obtain maximum allowable velocity 0.46 to 0.76 m/sec (Table 31). Convert to english units by dividing by 0.3 m/ft.

$$V_{max} = 1.5 \text{ to } 2.5\ ft/sec$$

4. From Figure 88, find the velocity for the specified ditch (2.9 ft/sec). Convert to metric by multiplying by 0.3m/ft.

$$V_{ditch} = 0.87\ m/sec.$$

5. Compare the calculated ditch velocity with the maximum recommended velocity for sandy channels:

specified ditch	maximum velocity
0.87 m/sec	0.46 - 0.76 m/sec

The ditch has too great a velocity given the conditions stated above. Therefore, measures must be taken that will reduce the water velocity. Water velocity in ditches can be reduced by protecting the channel with vegetation, rock, or by changing the channel shape. (With vegetative protection, the friction factor (n) becomes 0.030 - 0.050 and the maximum recommended velocity becomes 0.9 - 1.2 m/sec.)

6. Obtain velocity for specified ditch with vegetative protection by referring to Figure 88 (1.9 feet per second).

7. Compare the calculated ditch velocity with the maximum recommended velocity for vegetation protected channels (average turf) with easily eroded soils:

specified ditch	maximum velocity
0.57 m/sec	0.9 - 1.2 m/sec

8. If the specified ditch has a lower velocity than the recommended maximum velocities, it should be stable as long as the vegetation remains intact.

Berms can be constructed of native material containing sufficient fines to make the berm impervious and to allow it to be shaped and compacted to about 90 percent maximum density. Berm dimensions are illustrated in Figure 89.

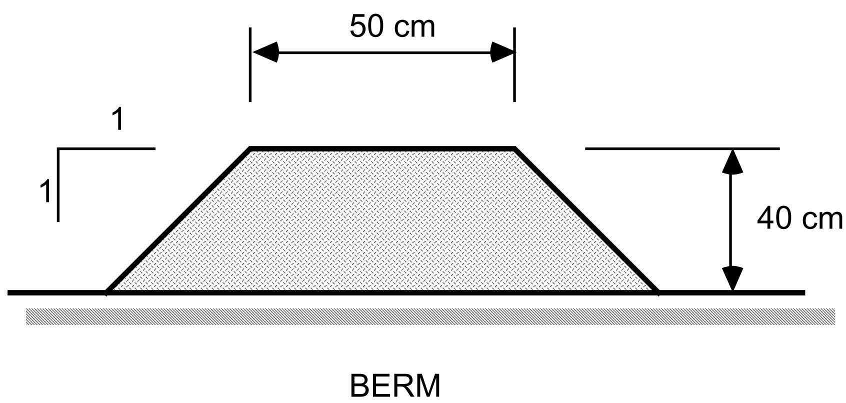


Figure 89. Minimum berm dimensions.

4.4.4 Ditch Relief Culverts

Water collected in the cutslope ditch line has to be drained across the road prism for discharge at regular intervals. Cross drains should be installed at a frequency that does not allow the ditch flow to approach maximum design water velocities. Intercepting dips or open top culverts (Chapter 4.4.2) perform adequately up to a certain point. However, these techniques are not adequate or appropriate when the following conditions are present either in combination or alone:

- high traffic volumes or loads and characteristic rutting
- steep side slopes
- large volumes of ditch water from rainfall,snowfall, springs, or seepage.

Ditch relief culverts do not impact or impede traffic as dips and open-top culverts do. Intercepting dips may become a safety hazard on steep slopes as well as being difficult to construct. It is also undesirable to have large amounts of water running across the road surface because of sediment generation and seepage into the subgrade.

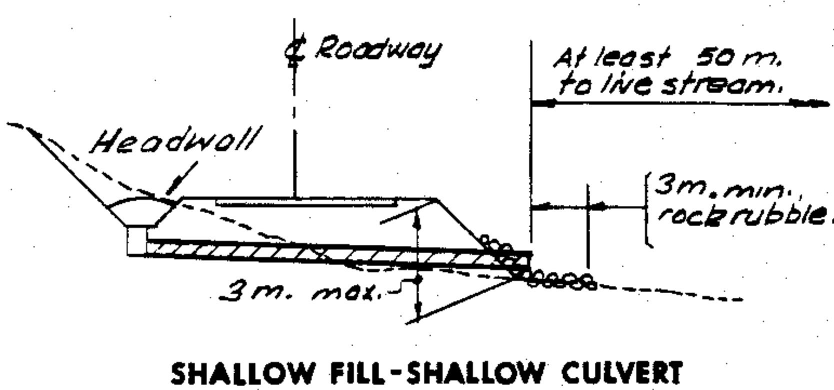
The frequency, location and installation method of ditch relief culverts is much more important than determining their capacity or size. Ditch relief culverts should be designed so that the half-full velocities are 0.7 to 1.0 m/sec in order to transport sediment through the culvert and should be at least 45 cm (18 inches) in diameter depending on debris problems. Larger culverts are more easily cleaned out than narrow ones. Every subsequent relief culvert should be one size larger than the one immediately upstream from it. This way, an added safety factor is built in should one culvert become blocked.

As with dips, open top culverts, and water bars, ditch relief and lateral drain culverts should cross the roadway at an angle greater than or equal to 30° downgrade. This helps insure that water is diverted from the roadside ditch and that sediment will not accumulate at the inlet. Accelerated ditch erosion may (1) erode the road prism making it unstable and unusable, and (2) cause culverts to plug or fail, thereby degrading water quality.

Selection of proper location is as important as spacing. Spacing recommendations should be used as a guide in determining the frequency of cross drain spacing. Final location is dictated by topographic and hydrologic considerations. Considerations discussed for for cross drain locations are also valid for culverts (see Figure 85). Considerations given for stream culvert installation, inlet and outlet protection should also be used for ditch relief culverts.

Culvert outlets with no outlet protection are very often the cause of later road failures. Normally, culvert outlets should extend approximately 30 - 50 cm beyond the toe of the fill. Minimal protection is required below the outlet for shallow fills. However, on larger fill slopes where the outlet may be a considerable distance above the toe of the fill, a downspout anchored to the fill slope should be used (Figure 90). Culvert outlets should be placed such that at least 50 meters is maintained between it and any live stream. If this is not possible, the rock lining of the outlet should be extended to 6 meters to increase its sediment trapping capacity (Figure 91). Coarse slash should be placed near the outlet to act as a sediment barrier.

Where fills consist entirely of heavy rock fragments, it is safe to allow culverts to discharge on to the slope. The size and weight of fragments must be sufficient to withstand the expected velocity of the design discharge. Rock aprons (Figure 92) are the least costly and easiest to install. A guide for selecting rock for use as riprap is illustrated in Figure 93.



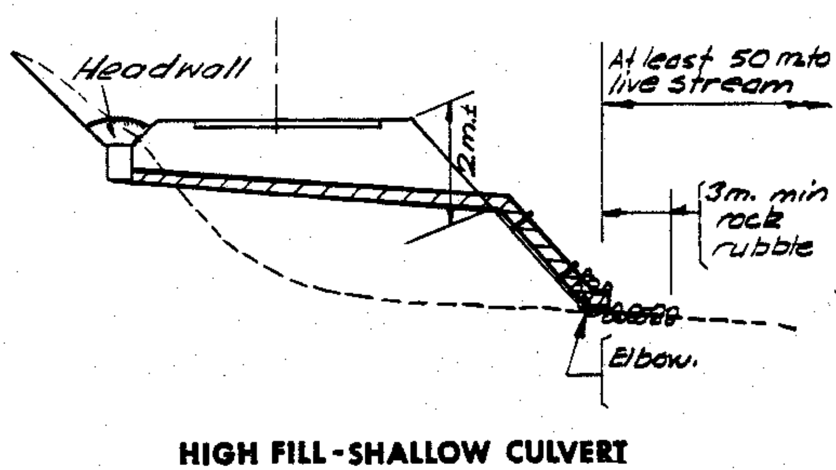


Figure 90. Ditch relief culvert installation showing the use of headwall, downspout and a splash barrier/energy dissipator at the outlet. Minimum culvert grade is 3 to 5 percent. Exit velocities should be checked. (U. S. Environmental Protection Agency, 1975)

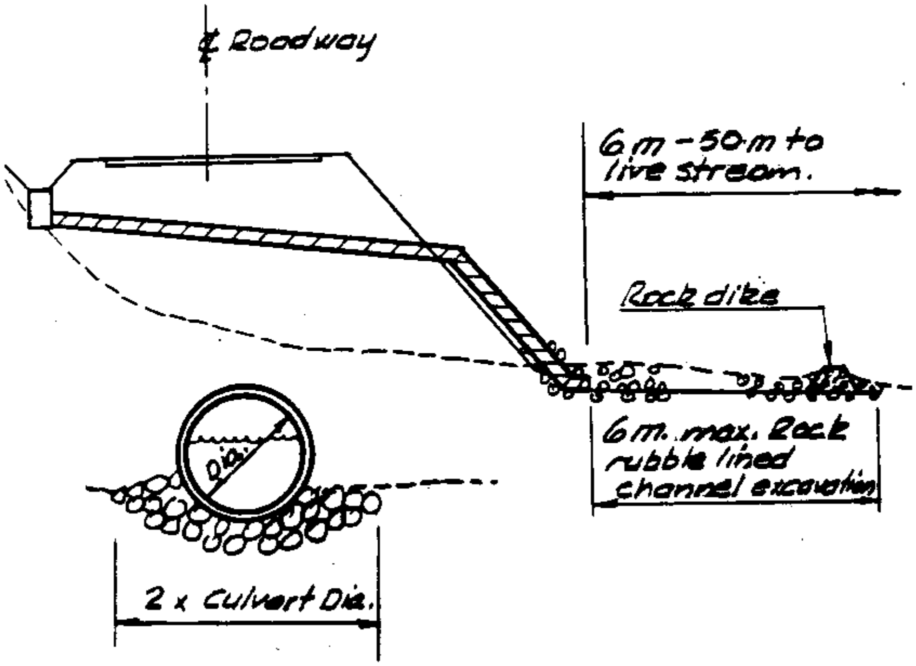


Figure 91. Ditch relief culvert in close proximity to live stream showing rock dike to diffuse ditch water and sediment before it reaches the stream. (U. S. Environmental Protection Agency, 1975)

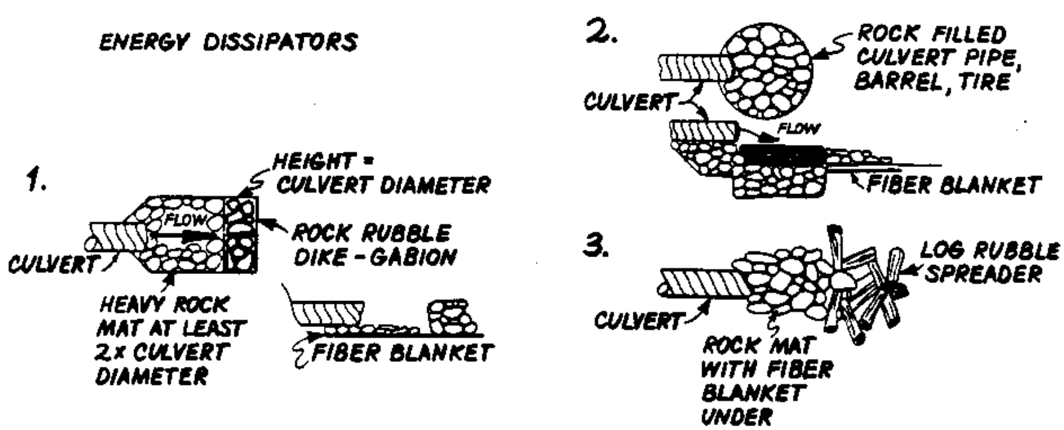


Figure 92. Energy dissipators. (Darrach, et al., 1981)

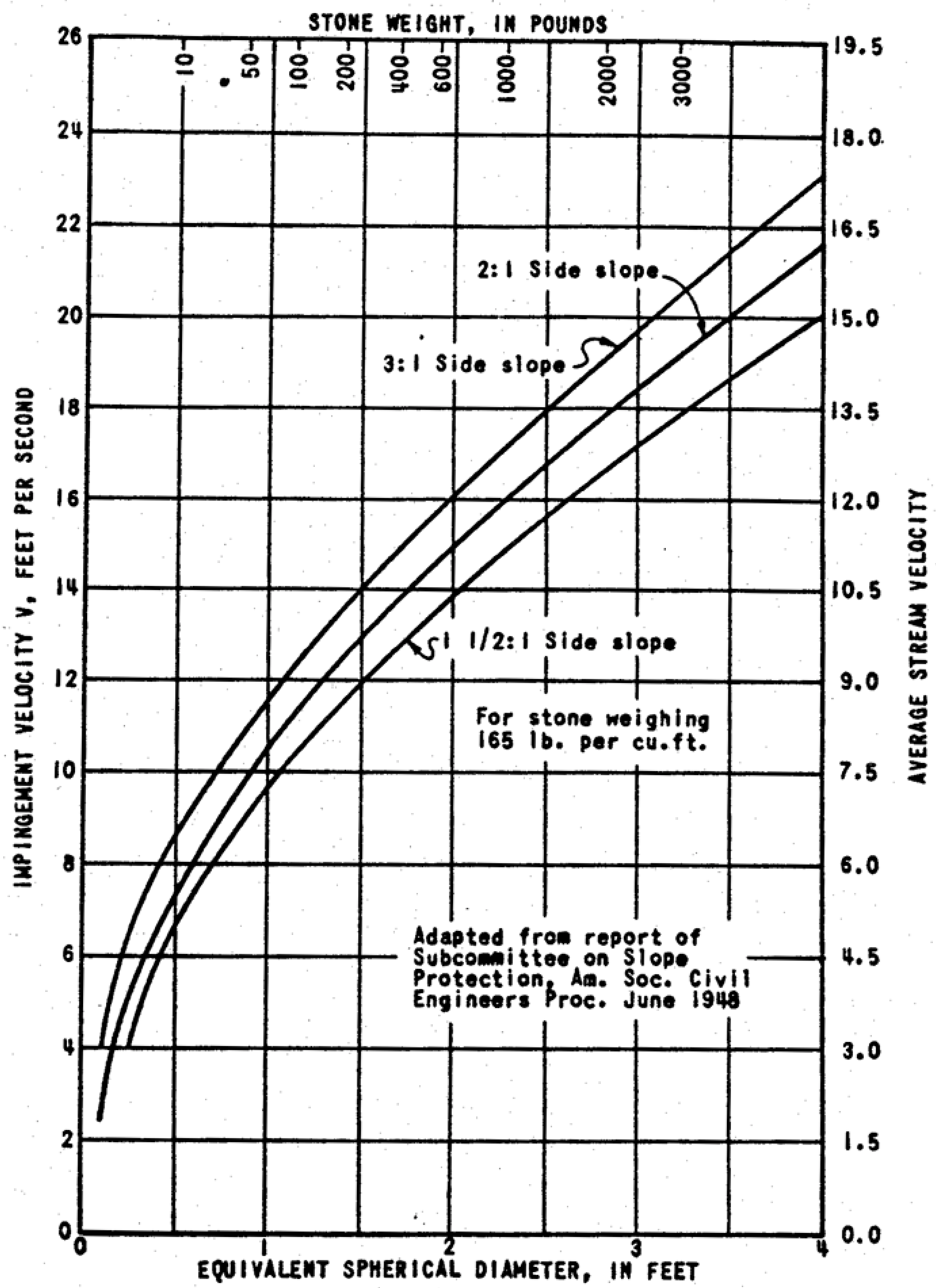


Figure 93. Size of stone that will resist displacement by water for various velocities and ditch side slopes.

1 ft. = 30 cm (U.S. Dept. of Commerce,1965)

The determination of culvert spacing for lateral drainage across the roadway is based on soil type, road grade, and rainfall characteristics. These variables have been incorporated into a maximum spacing guide for lateral drainage culverts developed by the Forest Soils Committee of the Douglas-fir Region in 1957. The spacing estimates are designed for sections of road 20 feet wide and include average cut bank and ditch one foot deep. Table 2 (Chapter 1.4.1) groups soils by standard soil textural classes into ten erosion classes having erodibility indices from 10 to 100, respectively. (Class I contains the most erodible soils and Class X the least erodible soils.) In order to arrive at an erosion class for a particular soil mixture, multiply the estimated content of the various components by their respective erosion index and add the results.

Example:

Name of component	% Content	Erosion index	Total Erosion Index
rock	20	100	20
Fine Gravel	50	90	45
Silt Loam	30	70	21
			86
		86 = Erosion Class VIII	

The spacing of lateral-drainage culverts can then be obtained from Table 34. The summary equation used to calculate values in Table 34, expressed in metric units, is:

$$Y = (1{,}376\, e^{0.0156X})(G\, R)^{-1}$$

where
- Y = lateral drain spacing (meters)
- e = base of natural logarithms (2.7183)
- X = erosion index
- G = road grade (%)
- R = 25-year, 15-minute rainfall intensity (centimeters/hour)

Values in Table 34 are based on rainfall intensities of 2.5 to 5 cm per hour (1 to 2 in/hr) falling in a fifteen minute period with an expected recurrence interval of 25 years. For areas having greater rainfall intensities for the 25 year storm, divide the values in the table by the following factors:

Rainfall intensity	Factor
less than 2.5 cm/hr (1 in/hr)	Whatever the intensity (0.75, 0.85, etc.)
5 to 7.5 cm/hr (2 to 3 in/hr)	1.50
7.5 to 10 cm/hr (3 to 4 in/hr)	1.75
10 to 12.5 cm/hr (4 to 5 in/hr)	2.00

Roads having grades less than 2 percent have a need for water removal to prevent water from soaking the subgrade or from overrunning the road surface. Thus, spacing for roads with 0.5 percent grades is closer than for roads with 2 percent grades. Usually, local experience will determine the spacing needed for road grades at these levels.

oad rade rcent)	Erosion classes[1] and Indices[2]									
	1/ I	II	III	IV	V	VI	VII	VIII	IX	X
	2/ 10	20	30	40	50	60	70	80	90	100
	meters									
2	270	368								
3	180	245	321	361						
4	135	183	240	271	305					
5	108	147	204	218	243	260	300			
6	90	123	161	182	203	216	251	303		
7	77	105	137	155	174	186	215	260	309	363
8	68	92	120	135	152	162	188	227	270	317
9	60	81	107	120	135	144	167	201	240	282
10	54	74	96	108	122	131	150	182	216	254
11	50	66	87	99	111	118	137	165	197	231
12	45	62	80	92	102	108	125	152	180	212
13	42	57	74	84	93	103	116	140	167	195
14	39	53	69	78	87	93	107	129	155	182
15	36	50	65	72	81	90	101	122	144	170
16	35	47	60	68	77	84	93	114	135	159
17	32	44	57	65	72	80	89	107	127	150
18	30	41	54	60	68	75	84	107	120	148

table 41 Guide for maximum spacing (in feet) of lateral drainage culverts by soil erosion classes and road grade (2% to 18%). (Forest Soils Comm., Douglas Fir Reg., PNW, 1957)

4.5. Subsurface drainage

When groundwater cannot be effectively removed or intercepted by surface drainage, subsurface drainage techniques are required. As discussed in previous sections of this workbook, if water is not removed from subgrade or pavement structures it may create instability, reduce load bearing capacity, increase the danger of frost action and create a safety hazard by freezing of the traveled surface.

Field investigations carried out during the route reconnaissance and design stages may not always reveal subdrainage problems. These less obvious problems can be effectively dealt with during construction. Field investigations should be carried out during the wet season and may include soil and/or geologic studies, borings or trenches to locate groundwater, inspections of natural and cut slopes in the local area, and measurement of discharge when possible. Sites with potential slope stability problems should be more thoroughly evaluated. When groundwater tables approach the ground surface, such as in low, swampy areas, the gradeline should be placed high enough to keep water from being drawn up into the fill by capillary action. Whenever possible, well graded granular materials, such as coarse sand, should be used for fill construction. For a detailed discussion of grading requirements for filter materials the reader is referred to the Earth Manual published by the U.S. Department of the Interior (1974).

Three types of subdrainage systems are commonly used:

(1) Pipe underdrains. This system consists of perforated pipe placed at the bottom of a narrow trench and backfilled with a filter material such as coarse sand. It is generally used along the toes of cut or fill slopes. The trench should be below the groundwater surface and dug into a lower, more inpervious soil layer to intercept groundwater. The drains may be made of metal, concrete, clay, asbestos-cement, or bituminous fiber and should be 15 centimeters (6 inches) in diameter or larger.

(2) Drilled drains. This system consists of perforated metal pipes placed in holes drilled into cut or fill slopes after construction.

(3) French drains. This system consists of trenches backfilled with porous material, such as very coarse sand or gravel. This type of drain is apt to become clogged with fines and is not recommended.

A major difficulty in selecting a drainage system is the lack of adequate performance data for various drainage methods. A good knowledge of seasonal groundwater fluctuations, variation in lateral and vertical permeability, and the ratio of vertical to lateral permeability are critical. Long term monitoring of drainage performance is important in determining appropriate prescriptions for future installations. For example, perforated drains are commonly prescribed but often will not function properly as a result of clogging of pores with fines or from geochemical reactions leading to the formation of precipitates. Several methods may be used to prevent plugging depending on soil characteristics and material availability. The first is to enclose the perforated pipe with geotextile fabric. Second, surround the pipe with an open graded aggregate material, which in turn is surrounded by a fabric. The use of fabric eliminates the need for an inverted filter consisting of various sized gravel and sand layers. Third, if fabric is not available, surround the pipe with a graded aggregate filter. Although the cost of installing such a drainage system is high, it may effectively reduce final road costs by decreasing the depth of base rock needed, thereby reducing subgrade widths and associated costs for clearing, excavating, and maintenance.

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CHAPTER 5

SURFACE AND SLOPE PROTECTIVE MEASURES

5.1 Introduction

Properly designed slope protection and stabilization has to include two components: a vegetational-biological and a mechanical-structural component. For maximum effect, both components must be integrally planned prior to road construction.

Properly designed and planted vegetative covers play a significant role in preventing surface erosion and shallow mass failures. The function of root systems of live plants on shallow soils on steep slopes is that of a binder for individual soil particles or aggregates. They act in several ways to increase slope stability: (1) they bond unstable soil mantles to stable subsoils or substrata, (2) they provide a cover of a laterally strong fine root systems close to the surface, and (3) they provide localized centers of reinforcement in the vicinity of individual trees where embedded stems act like a buttress pile or arch-abutment on a slope.

The structural-mechanical component can consist of conventional retaining walls, either the gravity or cantilever type, or a reinforced earth structure. Structural-mechanical stabilization techniques are called for in cases where the potential for deep-seated slope movement or high, lateral earth stresses exists.

A simplified flow chart is shown in Figure 94 which indicates the appropriate combination of methods to either maintain or achieve a stable and erosion-free slope. Implicit in any slope stability discussion is the effect of water and the importance of proper drainage. Mechanical drainage structures, such as culverts, ditches, water bars, is discussed in Chapter 4.3, 4.4, and 4.5. In addition to mechanical controls, however, vegetation can provide a form of "biological" drainage through plant transpiration. Root systems can effectively dewater soil mantles during their active growing season, but often the periods of most danger from slope failure and erosion do not coincide with peak transpiration periods.

More detailed information concerning biotechnical slope stabilization, the combination of vegetative and structural components can be found in Gray and Leiser (1982), Volgman (1979) and Schiechtl (1978, 1980).

5.2 Surface Protection Measures

The simplest and most cost effective means of stabilizing bare soil surfaces is through the use of vegetation or mulches. The objective of all surface stabilization techniques is to establish, as rapidly as possible, a dense vegetative cover to minimize available sources for sediment. Native plants generally require less expense and maintenance as well as being visually harmonious with the natural landscape. Many exotic species have been cultivated specifically for erosion protection and may also be suitable.

The body of research that points to road construction as the major cause of stream sedimentation in mountainous environments also indicates that surface erosion on severely disturbed soils such as road fills is highest immediately following disturbance and decreases rapidly over time. This suggests that stabilization measures must be employed during and immediately following construction. The methods chosen must provide rapid benefits hence merely seeding disturbed areas may not provide much relief. Transplanting living plants, fertilizing or mulching exposed soil surfaces may be required to achieve the desired level of protection.

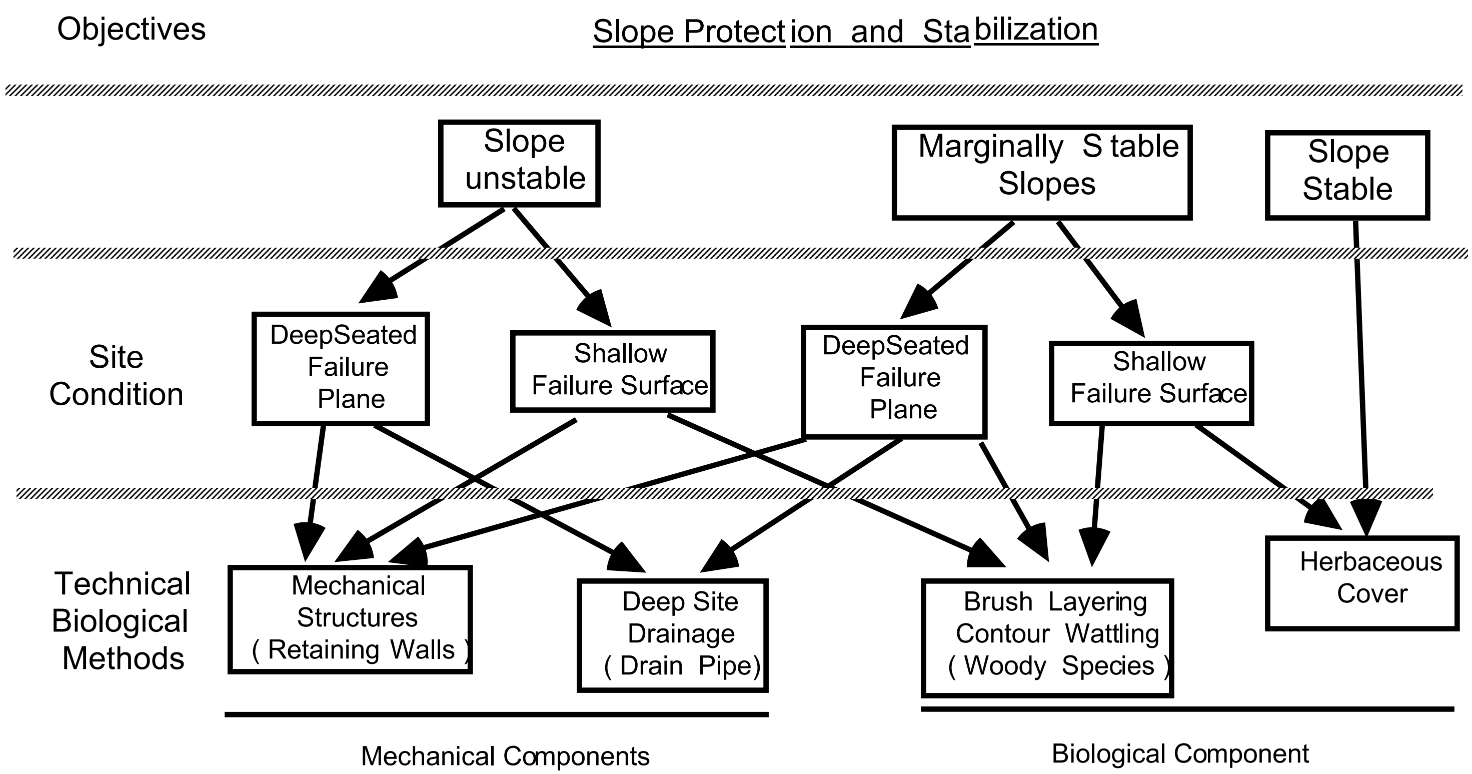

Figure 94. Selection criteria for slope stabilization methods.

5.2.1 Site Analysis

In order to ensure success of any revegetation effort, it is necessary to prepare an overall plan which considers the climate, vegetation, and microsite (soils, microclimate, slope, and aspect).

Climatic information should center on rainfall frequency distribution and amount. Likewise, average temperature, minimum/maximum temperatures, heating degree-days and number of frost free days are important points to consider. The vegetation analysis includes the suitability of native or exotic (introduced) plant species for the specific area in question. Here, the focus should be set on inventorying the entire spectrum of plant species that occupy a given site. The survey should note the particular microsites, soils and aspects in which different species grow. Typical points toconsider include:

- which plants do well in a wide range of conditions
- which plants make good seed crops
- which plants root readily when partially buried or resprout from roots cut during construction
- which plants have the best attributes for erosion control
  (rapid, dense growth; growing season; rooting characteristics).

Microsite evaluation should consider factors such as microclimate, aspect, topography and soils. The microclimate is primarily affected by variations in the radiation balance and the immediate surroundings. Changes in the radiation balance will affect microclimate regime and surface temperatures -- two extremely important factors for plant survival. Examples are changing the surface color or establishing a vapor barrier. Installing a vapor barrier to reduce evaporation losses will result in increases in the surface temperature. By changing the surface to a light color, the radiation absorbed by the surface can be reduced to compensate for the temperature rise. For further information on the interaction of microclimate and vegetation, the reader is referred to Geiger (1961, 1966).

Aspect and topography may reveal the need for specific site treatment either for plant survival and/or local site stabilization and slope preparation to allow for plant establishment. Wet and dry areas should be mapped for determining the need for special dewatering treatments or adoption of a particular seed mix. Likewise slope angles greater than 40 degrees are often difficult to revegetate, except in cases where slopes consist of decomposing bedrock or have a uniform, rocky subsoil. Assessment of the degree of local surface erosion (e.g. shallow versus deep seated) will determine the need for shallow rooted plant cover or a more deeply rooted plant species.

Soil analysis should consider the local soil profile and determine the predominant soil horizon present on the finished surface. Road cuts or fills may expose layers, strata, or horizons which may be significantly different from the surface soil which supports a given plant species community. Factors to consider are pH, salinity, nutrient levels, and texture (water holding capacity).

5.2.2 Site Preparation

In order to ensure success of any revegetation effort, it is necessary to prepare a proper seedbed. This may include reshaping the slope if gravity will cause "ravelling" of loose soil. A 1:1 slope ratio or better is recommended to provide a good seeding surface. Slopes of 6 meters (20 feet) or more should be broken up with small ditches or flat benches on the contour. Roughening the slope along the contours will reduce the chance of rilling and will provide small depressions that retain the seed. Oftentimes, construction work or tree and brush removal generally leave sites sufficiently scarified to permit seed to reach mineral soil.

Site preparation efforts on shallow soils may permanently damage the seeding site. The small volume of soil interlacing rocks may fall in the ditch line and be lost. Likewise, loose soil sidecast on fill slopes is extremely prone to erosion. Compaction of this sidecast material with one pass of a sheepsfoot roller will secure the soil to the slope and provide roughened surface for planting. Fill slopes often are best treated with brush layering or wattling in order to provide added mechanical stability. Typically, fill slopes are more prone to deep seated erosion (rilling or gullying) than are cut slopes.

5.2.3 Seeding and Planting

Type of seed, plant or cutting will determine the most appropriate planting technique. Herbaceous species typically germinate rapidly when compared to woody species. Woody species often must be planted to greater depths than herbaceous plants and may need mulching to keep them from drying out before germination takes place. Woody plants often require protection from herbivores and rodents because of their slow growth.

In addition to providing a dense, fibrous mat of protective material, seeded grasses and legumes[1] improve the organic and nutrient balance of the soil. They also act as "nurse plants" to young native species by providing shade and thereby reducing moisture loss from the soil. Grass seeding is often considered detrimental to tree regeneration, although this need not be the case. For instance, in southeast Alaska, grass seeding of exposed mineral soil helps establish spruce and hemlock seedlings by reducing the disruptive influence of frost heave and by retarding alder invasion. Grass species can also be selected such that competition with tree species for vital soil moisture during critical growth stages is minimized.

Mixtures of at least three plant species are recommended to assure continuous, even protection across a slope. In addition to factors mentioned at the beginning of this section, other factors to consider in selecting an appropriate mixture include:

- -- slope stability, angle, aspect, and exposure
- -- general climatic conditions, including conditions at the time of planting
- -- competitive ability of species to be planted in relation to native weed species or desired ultimate vegetation establishment
- -- susceptibility to foraging by livestock, rodents, and game
- -- visual and esthetic considerations
- -- physical and chemical characteristics of the soil.

It is impossible to recommend specific grass seed mixtures in this document. Likewise, seeding rates depend upon the number of live germinant seeds per unit weight and not simply on seed weight. For example, one kilogram of subterranean clover contains 34,000 seeds whereas one kilogram of timothy grass has 590,000 seeds--a 17.3 fold difference. In general, 1,100 to 1,600 live pure seed per square meter (100 to 150 per square foot) are sufficient seed densities for roadside erosion control in temperate climates (Berglund, 1978). It may be desirable to increase this rate in critical areas--culvert and bridge installations and road fill slopes--and decrease it in less critical or arid areas. Because of wide variations between sites and adaptability of individual grass and forb species around the globe, appropriate specialists should be consulted in each case in order to tailor the seed mixture to site conditions. These specialists include soil scientists, agronomists, ecologists, range conservationists, wildlife biologists, and landscape architects.

Generally, a vigorous, fast-spreading legume is included in the seed mixture because of its beneficial effects in replenishing soil nitrogen. Care must be taken, however, in ensuring that the chosen legume has been treated with an innoculant of the associated root bacteria. A problem associated with most legumes is their high palatability to livestock, deer, elk, and other game. Grazing animals will trample the soil and mechanical structures and create a more erosive condition than existed prior to the treatment. It is therefore

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[1] Any one of a large group of plants of the pea family (Leguminosae). Because of their ability to store and fix nitrogen, legumes, such as alfalfa, are often used in rotation with other cash crops to restore soil productivity.

recommended that legumes not be included in seed mixtures for sites readily accessible to game animals, cattle, sheep, or goats unless the legume is known to be unpalatable to animals (Adams, et al., 1983).

Road construction oftentimes results in the loss of the very thin mantle of fertile topsoil leaving a relatively infertile residual subsoil. Fertilizers are often required to provide young plants with sufficient nutrients. Again, variability from site to site requires the expertise of a specialist in order to determine proper fertilizer selection and application rates. In general, fertilizer prescriptions are developed on the basis of the amount of total nitrogen in the soil. If a soil test shows total nitrogen to be greater than 0.2 percent, no fertilizer is needed.

Fertilization normally occurs together with seeding either prior to or near the end of the rainy season. Two applications--one prior to and one after the rainy season--are extremely effective. Refertilization may also be needed in following years due to reductions in vigor of the crop. If fertilizer costs are prohibitive or supplies limited, it may be desirable to concentrate efforts on such key areas as large fills and culvert and bridge emplacements.

5.2.4 Application Methods

Techniques used in establishing grasses include hand-operated cyclone seeders, truck-mounted broadcast seeders, seed drills, and hydroseeders. Drilling is best as it places the seed directly in the soil at a controlled depth and seeding rate, but may be impossible on steep cut banks and fills. Hydroseeding is the application of seed, fertilizer, and mulch in a slurry of some sort of viscous water soluble binder, such as wood fiber, from a truck-mounted tank. This method is most suitable for large areas and steep slopes where plastering of materials is necessary to achieve uniform coverage. It is also expensive and sometimes impractical due to climatic, terrain, or road access conditions. Hand planting is generally effective for small areas and is often the least expensive. Covering the seed with at least 0.5 to 1 centimeter of soil is critical. Rainfall may help cover it, but raking or dragging seeded areas with tire chains, sections of cyclone fence, or similar objects is the most effective.

Soils which are heavily disturbed or which have little surface organic material to retard water runoff need protection afforded by any readily available mulching materials. Such materials include excelsior, straw, shredded logging residue or slash, and slurried wood or ground paper fibers. Excelsior provides the best protection but is very expensive. Straw mulch is very effective when applied at a rate of 5.5 metric tons per hectare (2 tons per acre) and secured to the surface either mechanically by punching it into the surface with the end of a shovel or chemically with a liquid "tackifier" such as emulsified asphalt. Table 36 shows the effectiveness of different mulches subjected to a rainfall rate of 64 mm (2.5 in) per hour on a 20 percent slope with 15 cm (6 in) of silt loam over compacted calcareous till. Figure 91 outlines a decision matrix to use in order to choose the most effective erosion control combination for a given set of site and climatic conditions.

table 43 Erosion control and vegetation establishment effectiveness of various mulches on highways in eastern and western Washington. Soils: silty, sandy and gravelly loams, glacial till consisting of sand, gravel and compacted silts and clays (all are subsoil materials without topsoil addition). Slope lengths: approximate maximum of 50 m (165 ft). Application rates: Cereal straw - 5,500 kg/ha (2 t/ac); Straw plus asphalt - 5,500 kg/ha (2 t/ac) and 0.757 l/kg (200 gal/t), respectively; Wood cellulose fiber - 1,345 kg/ha (1,200 lbs/ac); Sod - bentgrass strips 46 cm (18 in) by 1.8 m (6 ft) pegged down every third row.

	Surface Cover						
	Jute	Excelsior	Straw	Straw & Asphalt	Asphalt	Wood Fiber	Sod
	Rating						
Erosion Control							
Sheet erosion 1:1 slope	9.0	10.0	8.0	10.0	6.0	3.0	10.0
Sheet erosion 2:1 slope	9.0	10.0	9.0	10.0	7.0	6.0	10.0
Sheet erosion 3:1 + slope	10.0	10.0	10.0	10.0	9.0	10.0	10.0
Rill erosion 1:1 slope	6.0	10.0	8.0	10.0	6.0	3.0	10.0
Rill erosion 2:1 slope	8.0	10.0	9.0	10.0	7.0	5.0	-
Rill erosion 3:1 + slope	10.0	10.0	10.0	10.0	9.0	10.0	10.0
Slump erosion 1:1 slope	10.0	8.0	6.0	7.0	3.0	3.0	8.0
Slump erosion 2:1 slope	10.0	9.0	7.0	8.0	5.0	4.0	9.0
Slump erosion 3:1 slope	Slumps usually do not occur.						
Vegetation Establishment							
1.5:1 glacial till cut slope	7.5	9.0	7.5	8.5	7.5	6.0	-
2:1 glacial till cut slope	8.9	9.5	8.0	9.3	8.7	6.2	-
2:1 sandy loam fill slope	9.0	10.0	9.0	10.0	7.5	8.5	10.0
2.5:1 silt loam cut slope	5.0	10.0	-	7.8	6.0	-	-

Effectiveness rating: 10.0 = most effective, 1.0 = not effective.

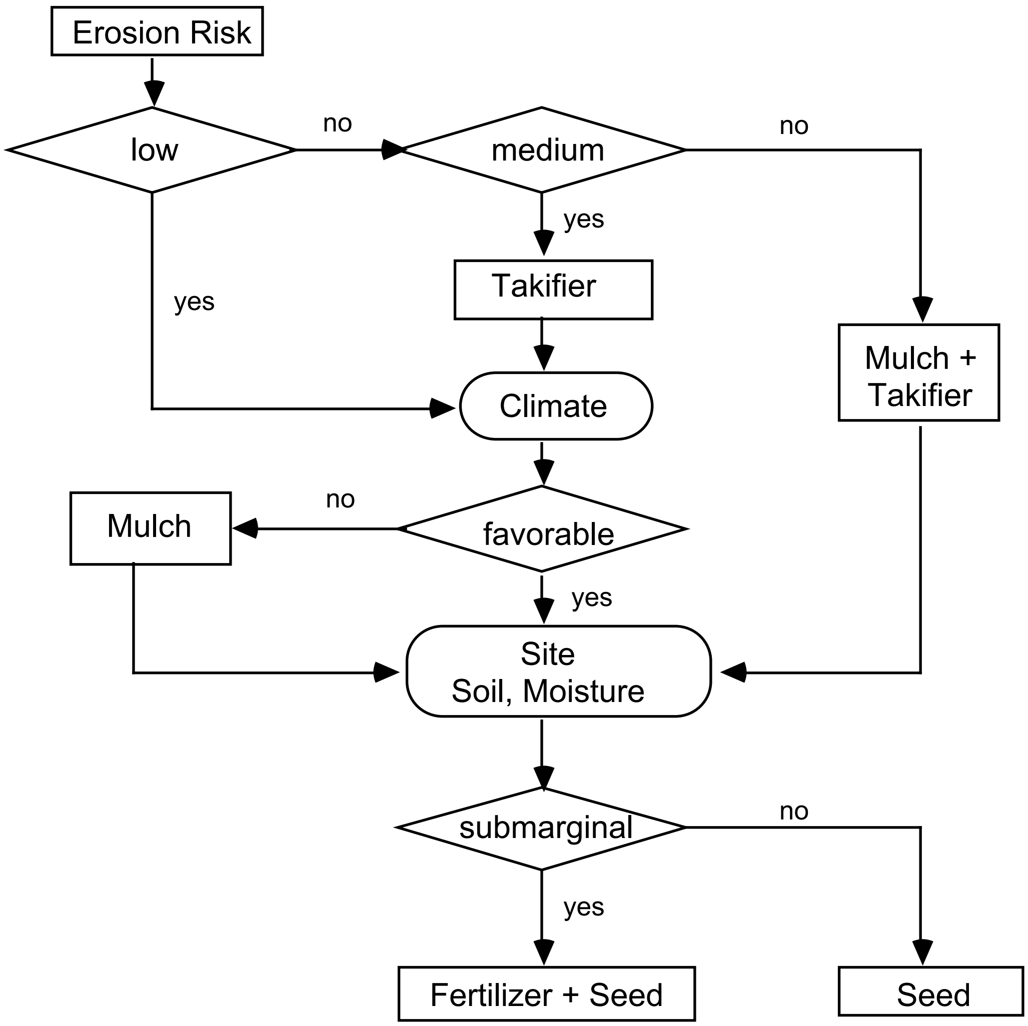


Figure 95. Selection criteria for surface cover establishment methods in relation to erosion risk.

When using straw as a mulch, it is recommended that only "clean" straw be used to prevent the introduction of noxious plants. Wood fiber should be applied at a rate of 1.4 to 1.6 metric tons per hectare (0.5 to 0.6 tons per acre). At higher rates, wood fiber improves erosion control but inhibits plant establishment. When mulching follows seed and fertilizer application, rather than in combination with seeding and fertilizing (as is the case sometimes with hydroseeding), there is a much greater chance that seed will be in direct contact with mineral soil and will germinate more readily. Hydroseeding a fiber-seed-water slurry can entrap 60-70 percent of the seed in the mulch layer.

5.2.5 Wattling and filter strips

Oftentimes grass cover alone is insufficient to prevent erosion on long, steep slopes. Wattling or filter strips work to break the slope into short segments so that the kinetic energy of water flowing over the surface is dissipated. Many different methods and materials can be employed to achieve this objective. Heede (1975) successfully used submerged burlap strips 30 cm (1 ft) wide, placed vertically into the ground on the contour 0.5 to 1 meter (1.5 to 3 ft) apart from each other to control rilling in semi-desert regions. Filter windrows can be fabricated from slash accumulated during road construction and can easily be constructed simultaneously along with the road (Cook and King, 1983). A rough estimate of production rates for windrow construction during one demonstration is 52 m/hour (170 ft/hour) using a track-mounted Caterpillar 235 hydraulic pull shovel (a large backhoe). Sediment trapping efficiency was estimated at between 75 and 85 percent. Windrows in this demonstration consisted of logs not less than 0.45 m (18 in) diameter secured against undisturbed stumps, rocks, or trees on fill slopes immediately above and parallel to the toe of the slope. Slash (tops, limbs, and brush not exceeding 15 cm (6 in) in diameter and 3.7 m (12 ft) in length were then placed above the logs in neat piles (also see Chapter 6.3.3, Figure 119).

Table 36 is included to provide guidance in determining appropriate windrow widths based on the length of the slope and type of material used to construct the windrow.

Wattling (Figure 96) consists of combined mechanical and natural stabilization techniques in which stakes are placed on the contour 0.5 m (1.6 ft) apart and at 1.2 m (4 ft) intervals between rows. A trench is then dug 20 cm (8 in) wide and 25 cm (10 in) deep against or immediately above the contour stakes. Bundles of live vegetative materials (such as Salix spp., Bambusa spp., Cassia sepium or other locally available material) 13 cm (5 in) in diameter and 3 m (10 ft) long are placed in the trench overlapping end and tail. The wattling bundles are then covered with soil so that part of the branches and leaves above ground are left visible.It is very important that the soil is worked thoroughly into the interstices of the wattles. During the installation workers should walk on the wattles as much as possible to insure maximum compaction and working the soil into the bundles.

Incorrect installation of wattles may actually aid in soil slumping because of collection of water in the wattle trenches. It is therefore important that soil is thoroughly worked into the wattles and no trench remains to be filled with water. Likewise firm staking is important particularly in areas where frost heaving is a problem. An average 10 person crew can treat 200 m^2 to 250 m^2 (2,000 to 2,500 ft^2) in a day (Sheng, 1977b). More detailed information on wattling procedure and installation can also be found in Kraebel (1936), Grey and Leiser (1982), and Schiechtl(1978,1980).

table 44 Windrow protective strip widths required below the shoulders[1] of 5 year old[2] forest roads built on soils derived from basalt[3], having 9 m cross-drain spacing[4], zero initial obstruction distance[5], and 100 percent fill slope cover density[6].

(U. S. Environmental Protection Agency, 1975)

Obstruction spacing	Protective windrow width by type of obstruction					
	Depressions or mounds	Logs	Rocks	Trees and stumps	Slash and brush	Herbaceous vegetation
	meters					
0.3	10.6	11.2	11.6	12.1	12.5	13.1
0.6	11.3	12.2	13.1	14.0	14.9	15.9
0.9	11.9	13.1	14.3	15.9	17.4	18.6
1.2	12.2	14.0	15.9	17.7	19.5	21.3
1.5	12.5	14.6	17.0	19.2	21.6	23.8
1.8		15.2	18.0	20.7	23.5	26.2
2.1		15.9	18.9	22.2	25.6	28.7
2.4		16.2	19.8	23.5	27.1	30.8
2.7		16.5	20.4	24.7	29.0	32.9
3.0				25.9	30.5	35.1
3.4				27.8	31.7	36.9
3.7						38.7

1 For protective strip widths from centerlines of proposed roads, increase widths by one-half the proposed road width.

2 If storage capacity of obstructions is to be renewed when roads are 3 years old, reduce protective strip width by 7 m.

3 If soil is derived from andesite, increase protective strip width by 30 cm; from glacial till, increase 1 m; from hard sediments, increase 2.4 m; from granite, increase 2.5 m; from loess, increase 7 m.

4 For each increase in cross-drain spacing beyond 9 m, increase protective strip width 30 cm.

5 For each 1.5 m increase in distance to the initial obstruction beyond zero (or the road shoulder), increase protective strip width 1.2 m.

6 For each 10 percent decrease in fill slope cover below a density of 100 percent, increase protective strip width 0.30 m.

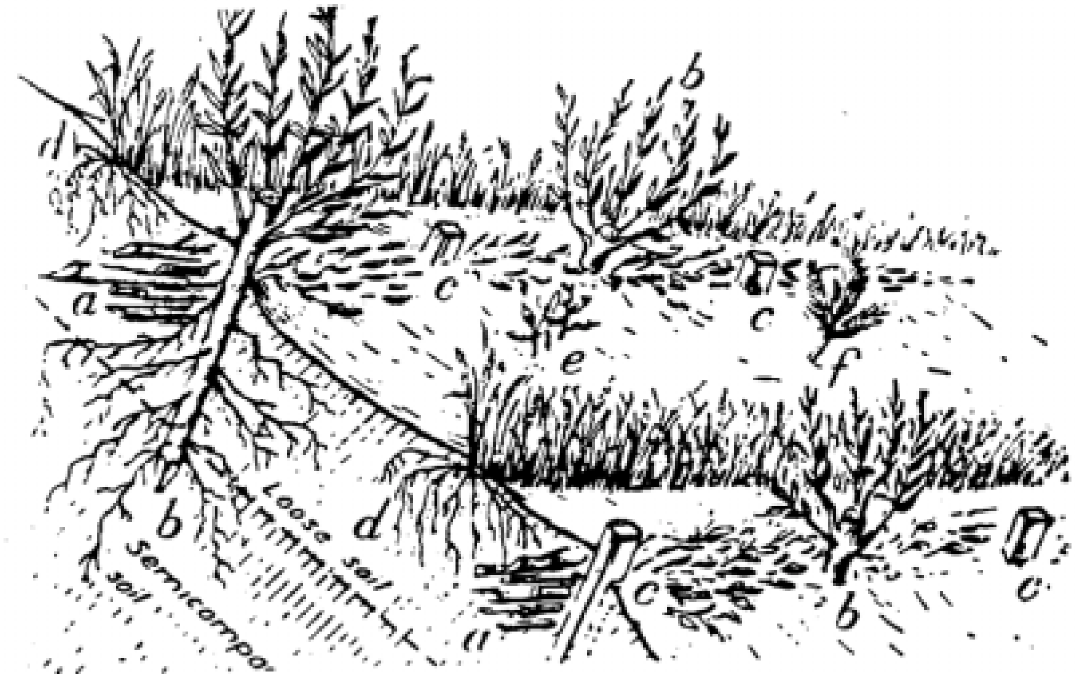


Figure 96. Preparation and installation procedure for contour wattling, using live willow stakes (after Kraebel, 1936).

5.2.6 Brush Layering

Contour brush layering (Figure 97) involves embedding green branches of shrubs or trees on successive horizontal layers into the slope. Brush layering is different from wattling in that (1) branches are placed into the slope perpendicular to the strike instead of parallel creating better resistance to shallow shear failure, (2) staking is not required, (3) brush layers and surfaces can be reinforced with wire mesh or other material, (4) brush layers can be incorporated into the construction process of a fill. That is, brush layers are laid down, the next lift of soil is placed and compacted, and the process is repeated.

Contour brush layering is comparable to the "reinforced earth" concept where the cuttings or branches act in the same fashion as the reinforcing strips.

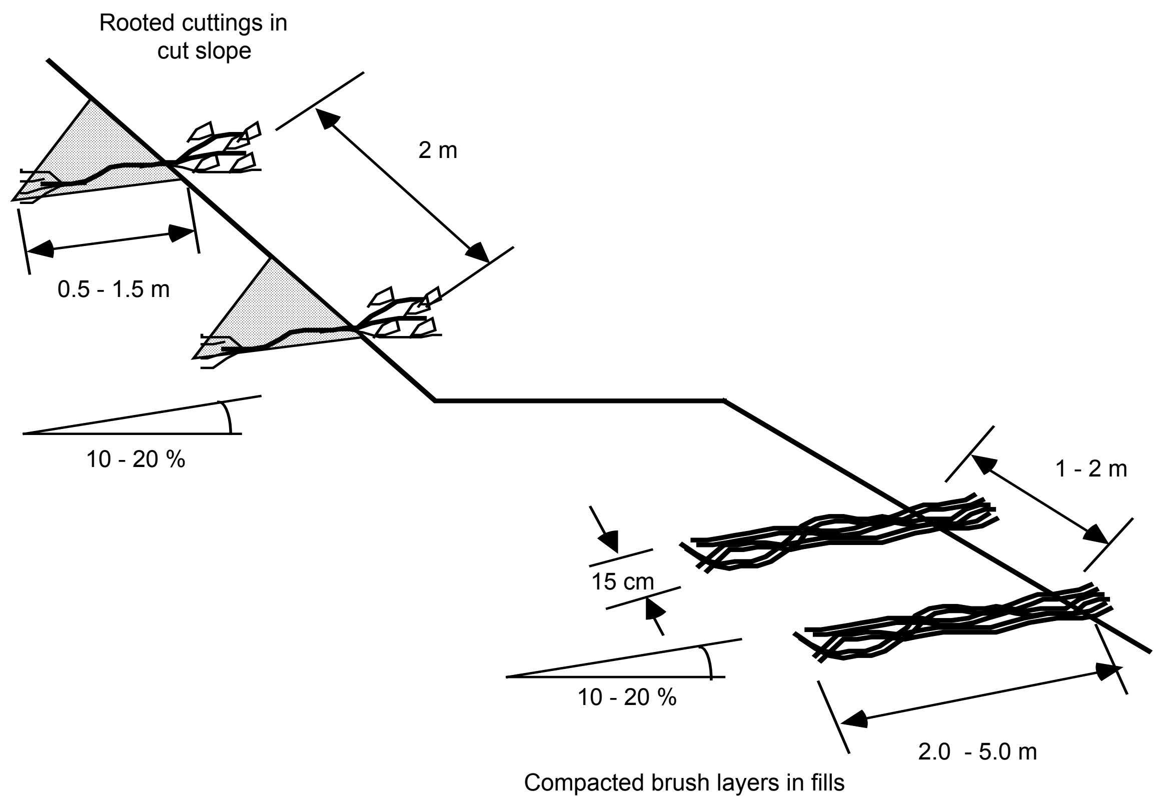


Figure 97. Brush layer installation for slope stabilization using rooted plants for cut slope and green branches for fill slope stabilization.

According to Schiechtl (1978,1980) three brush layering techniques may be used. The first technique uses brush layers consisting of rooted plants or rooted cuttings only. Approximately 5 to 20 rooted seedlings per meter are required (See Figure 97).

The second technique utilizes green cuttings or branches from alder, cottonwood or willow. On cut slopes, cuttings from 0.5 to 2.0 meters in length are used. On fill slopes, cutting length can vary from 2.0 to 5.0 meters. This method is particularly suited for use in critical and sensitive areas

The third technique is a combination of the first two methods where rooted seedlings or cuttings are installed together with branches or cuttings. From 1 to 5 rooted cuttings per meter are required.

In all three methods, the material should be placed with the butt ends slightly dipping into the fill (20 percent) and the tips protruding a few centimeters. Vertical spacing of brush layers can vary from 0.5 to 1.5 meters depending on soil type, erosion hazard, slope angle and length of slope. A good practice is to vary the vertical spacing on long slopes with short spacings at the bottom and increasing the spacing towards the upper end of the slope.

A variation to the contour brush layering approach where the layers are positioned along the contours or horizontally is to arrange the layers at a 10 - 40 percent incline. This variation is called for on wet, heavy soils or slopes with numerous small springs. Water collecting in the berms or brush layers is drained off and does not stagnate and infiltrate into the slope.

Installation procedures typically proceed from bottom to top. Fill slope installation is simple. However, care should be taken that the brush layer dips into the slope at least 20 percent. The next soil layer is placed on top and compacted. Cut slope installation requires the opening of a ditch or berm. As with fills, work progresses from bottom to top. The excavation of the upper berm is used for filling-in and covering the lower brush layer.

5.2.7. Mechanical Treatment

Mechanical surface stabilization measures consist of diversion ditches and terraces, serrations, or scarification and can be used in conjunction with vegetative methods discussed above. These methods generally require detailed engineering design and location. T. C. Sheng (1977a) discusses several different methods for the construction of bench terraces together with tables providing design information and costs.

Serrations consist of steps of 60 to 120 cm (2 to 4 ft) cut vertically and horizontally along the normal, intended slope gradient. After treatment, the slope is seeded, fertilized, and mulched as discussed in Chapter 5.2.3. The steps provide improved seedbeds free of sliding forces normally experienced on steep slopes. Serrations are only effective on cut slopes of soft rock or similar material that will stand vertically or near vertically for a few years in cut heights of approximately one meter. Likewise, this method is not applicable to soil types where the rate of slough is so high that vegetative cover is buried and destroyed. If acceptable slope material is soft, the slope should be allowed to slough before seeding until about one-third of the steps are filled. Otherwise, grass may be destroyed by the excessive rate of initial slough.

Roughened or scarified slopes may not be as esthetically pleasing to the eye as smoothly graded cut and fill slopes, but they are far more effective in increasing infiltration and impeding runoff. Scarified slopes also provide small depressions for the retention of seed and also help mulch to better adhere to the slope. Roughening may be accomplished by several means including deep cleated bulldozers traveling up and down the slope, sheepsfoot rollers, rock rippers, and brush rakes mounted on bulldozers. The path of the roughened slope should trend perpendicular to the direction of flow.

5.3 Mass Movement Protection

Deep seated mass failures can be dealt with in three ways. The methods are categorized by the way they affect soil stability.

1. <u>Avoidance Methods:</u> Relocate road on a more stable area (for large, unstable fills probably by far the most appropriate approach).

2. <u>Reducing Shear Stress:</u> This is achieved through excavation of unacceptable materials. It creates a reduction in soil weight and can be accomplished by: a) removal of soil mass at the top of the potential slide, b) flattening of cut slopes above the road, c) benching of cut slopes.

3. <u>Increasing Shear Strength:</u> This is achieved through retaining structures. They can be grouped into a) rock buttresses at the toe of fill slope, b) cribs or gravity retaining walls at toe of fill or cut, and c) piling walls, likewise at the toe of fills or cuts.

Engineering and structural methods for stabilizing slopes can be grouped into four categories:

1. <u>Excavation and filling techniques.</u> This would include excavating the toe of an earth flow until successive failures result in a stable slope, removing and replacing failed material with lighter, more stable material, or recompacted debris, excavating to unload upper portions of a mass failure, and filling to load the lower portions of a mass failure (most likely in conjunction with other loading or restraining structures).

2. Drainage techniques. This would include efforts to remove or disperse surface water (as discussed in Chapter 4), drainage of tension cracks, using rock fill underlain by filter cloth to prevent upward migration of water into the road prism, insertion of trench drains, perforated, horizontal drains, or drainage galleries, insertion of vertical drains or wells discharged by syphons, or pumps, and electro-osmosis (the use of direct current passing between wellpoints and steel rods placed midway between the rods to increase the drainage rate) for drainage of low permeability soils.

3. Restraining structures. These include retaining walls, piles, buttresses, counterweight fills, cribs, bin walls, reinforced earth, and pre-stressed or post-tensioned soil or rock anchors (Figure 98). Organizations such as highway departments and railroads have developed charts and tables giving earth pressures for the design of retaining walls that require a minimum of computation. Nearly all of these charts and tables are based on the Rankine formula, which describes earth pressures as a function of unit weight and internal angle of friction of the backfill material.

4. Miscellaneous techniques. Grouting can be used to reduce soil permeability, thereby preventing the ingress of groundwater into a failure zone. Chemical stabilization, generally in the form of ion exchange methods, is accomplished by high pressure injection of specific ion exchange solutions into failure zones or into closely spaced pre-drillled holes throughout the movement zone. Heating or baking of clay soils can sometimes improve their strength, and, rarely, freezing of soils will help gain temporary stability. Localized electro-osmosis can be used to form in situ anchors or tie-backs. Suppression of natural electro-osmosis can be used to reduce unfavorable groundwater pressures. Blasting is sometimes used to disrupt failure surfaces and to improve drainage.

For correcting cut or fill failures, a detailed investigation into the reason for the failure, particularly the position and geometry of the failure surface and other potential failure surfaces, is required prior to prescribing ameliorative measures. The neutral line concept, discussed by Hutchinson (1977) and Sidle, et al. (1985), is of particular interest in assessing the impact of cuts and fills on factors of safety. The neutral line describes where the load attributed to fill material will have no effect on the original factor of safety. If the load falls up-slope of the neutral line, the factor of safety will decrease; if it is downslope of the neutral line, the factor of safety will decrease.

The use of any of these stabilization techniques requires extensive site specific investigations into the mechanics of soils, groundwater, and bedrock occurring on the site. It is advisable to utilize the most experienced geotechnical or highway engineer available in order to provide the most effective design possible. As can be inferred from the above discussion, any of these techniques will be quite costly to design and install. Furthermore, the success of such measures in functioning adequately through time is highly dependent on the skill of the design engineer and the degree of maintenance employed after construction. Hence, avoidance of areas where structural stabilization measures are required will result in considerable short term and long term cost savings, and the major opportunity for reducing landslide risk is at the route planning stage.

The purpose of a retaining structure is to provide stability against sliding or failure and protection against scour and erosion of a slope, or the toe or cutface. The typical retaining structure on forest roads is a gravity retaining wall, which resists earth pressure by the force of its own weight. Excavation and/or fill volume can be significantly reduced particularly on steep side slopes (see also discussion in Chapter 3.2).

The volume of cribs or retaining walls should be 1/6 to 1/10 of that of the total moving mass to be retained. As a rule, the foundation or base should at least extend 1.2 to 2.0 meters below the slip plane in order to be effective.

The forces acting on a retaining wall are similar to those acting on a natural slope. These forces are be grouped into resisting forces (forces resisting failure) and driving forces (forces causing failure) as illustrated in Figure 99.

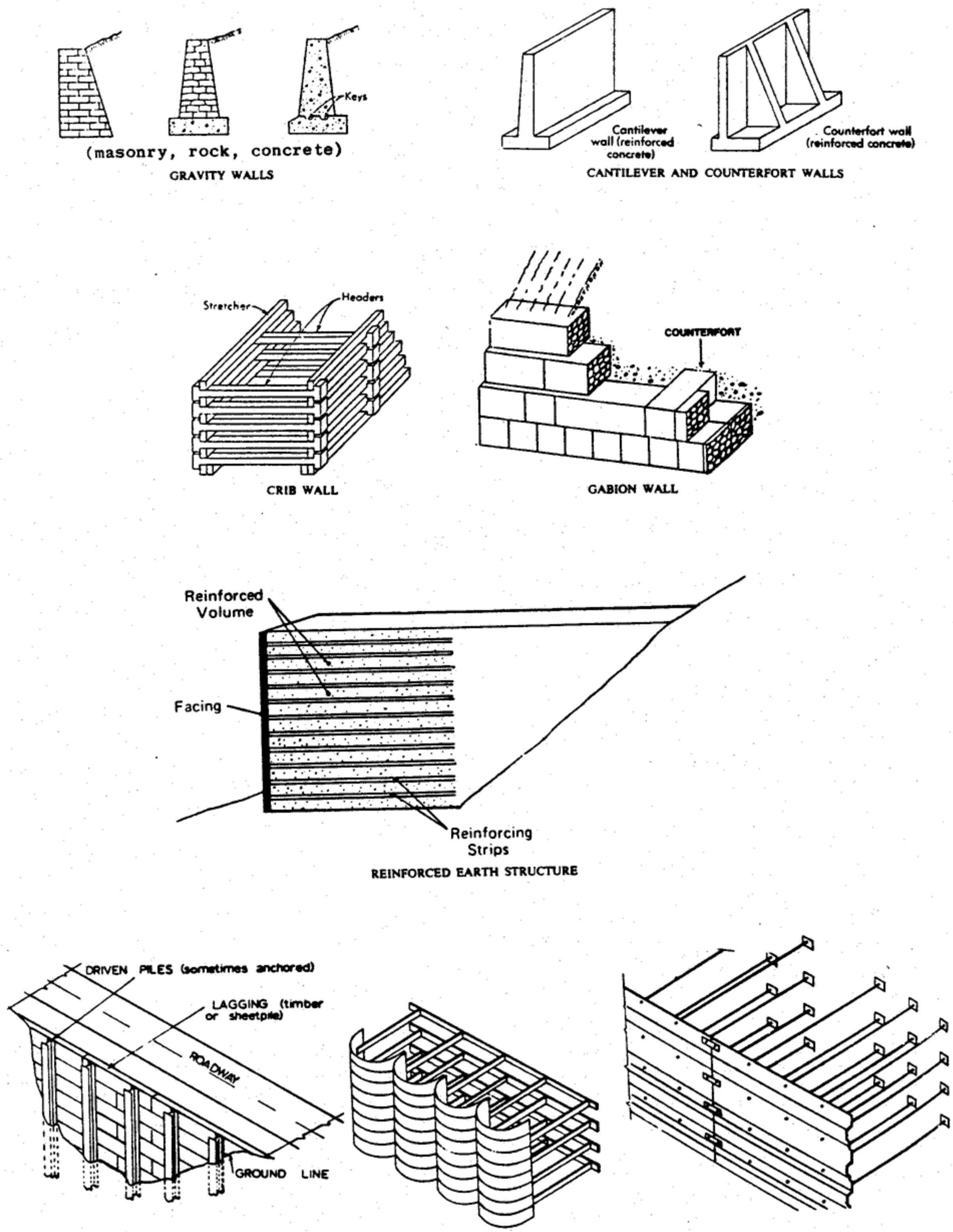
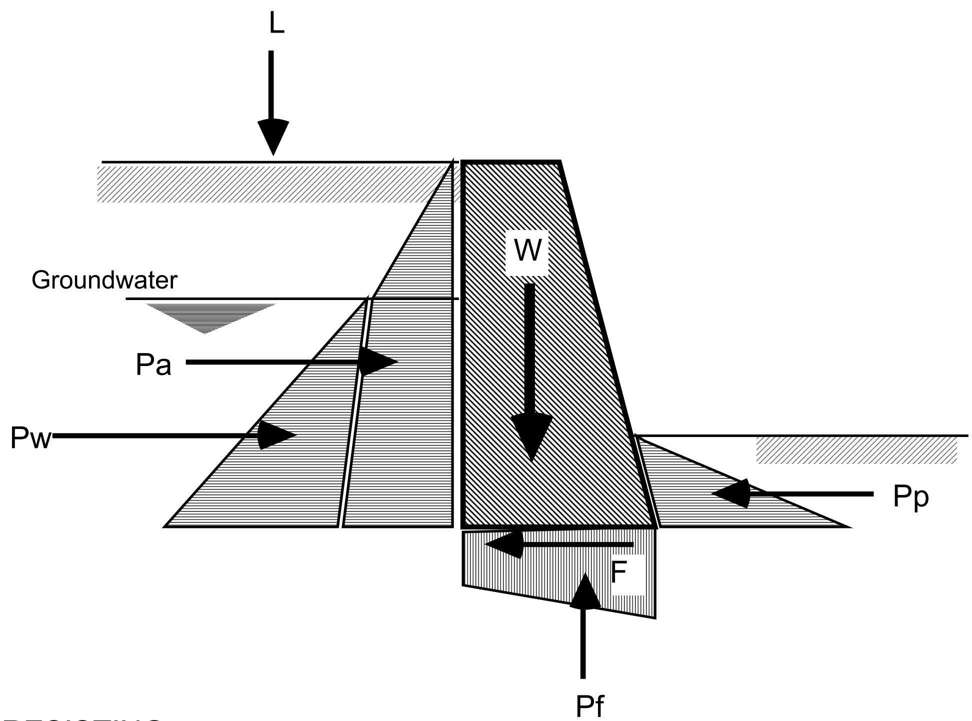


Figure 98. Types of retaining walls.

Failure of a retaining wall can be brought about through

- sliding along its base
- bearing capacity failure
- overturning

FORCES ON RETAINING STRUCTURES



<u>RESISTING</u>

WATER PRESSURE (Pw)
PASSIVE SOIL PRESSURE (Pp)
SLIDING FRICTION (F)
FOUNDATION PRESSURE (Pf)

<u>DRIVING FORCES</u>

WATER PRESSURE (Pw)
ACTIVE SOIL PRESSURE (Pa)
WEIGHT (W)
SURCHARGE LOAD (L)

Figure 99. Forces acting on a retaining wall.

For low toe walls or toe-bench structures it is usually possible to use standard designs. Standard designs have been developed on the basis of soil mechanics and past performance. Standard designs have been developed by a number of sources and are typically available on request. These sources include manufacturers of retaining wall systems (e.g. gabions, crib walls, welded-wire walls, geotextiles), trade associations (e.g. American Wood Preservers Institute), and state and federal agencies (e.g. Forest Service, Highway Administration, local transportation departments). Standard designs can be used safely provided they conform with the local conditions The factors or conditions to consider include maximum wall height, surcharge conditions, strength and finish of structural members, inclination requirements, construction requirements, backfill material, soil conditions at base, and groundwater conditions.

An example of a standard log crib wall is shown in Figure 100. A thorough discussion of timber walls is provided by Schuster et al. (1973). Timber crib walls and gabion structures can withstand some limited, differential base settlement without a significantly affecting the retaining action.Drainage characteristics of the backfill and crib material is important because of potential water pressure build-up. Most standard designs assume free draining sand or gravel fills.

Gabions are rectangular containers made of heavy steel wire and filled with cobble-sized rocks (10 to 30 cm in diameter). A typical gabion retaining wall is shown in Figure 101. Advantages of gabion structures are ease of construction, tolerance of uneven settlement, and good drainage characteristics. Gabion walls are particularly suited in areas where only small, fragmented rocks are available. Typically, they can be built without heavy equipment. Both crib and gabion walls lend themselves to incorporation of vegetative systems to provide additional strength over time as well as providing a more esthetically pleasing appearance.

Gravity retaining structures utilizing standard designs are typically limited to a height of less than 6.0 meters. Structures requiring a larger height have to be designed based on site-specific soil mechanical conditions.

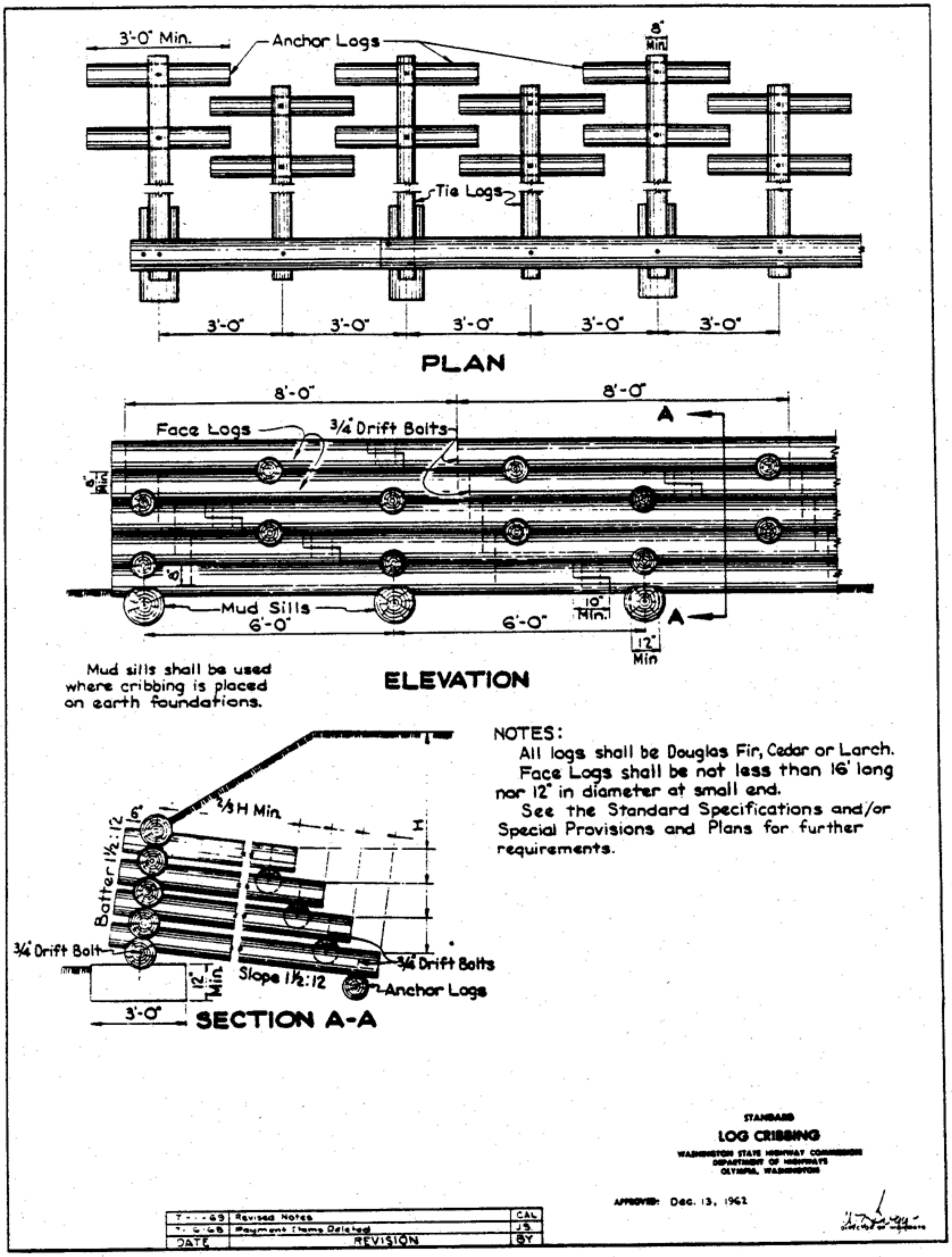


Figure 100. Example of a standard crib wall design. (Wash. State Dept. of Highways)

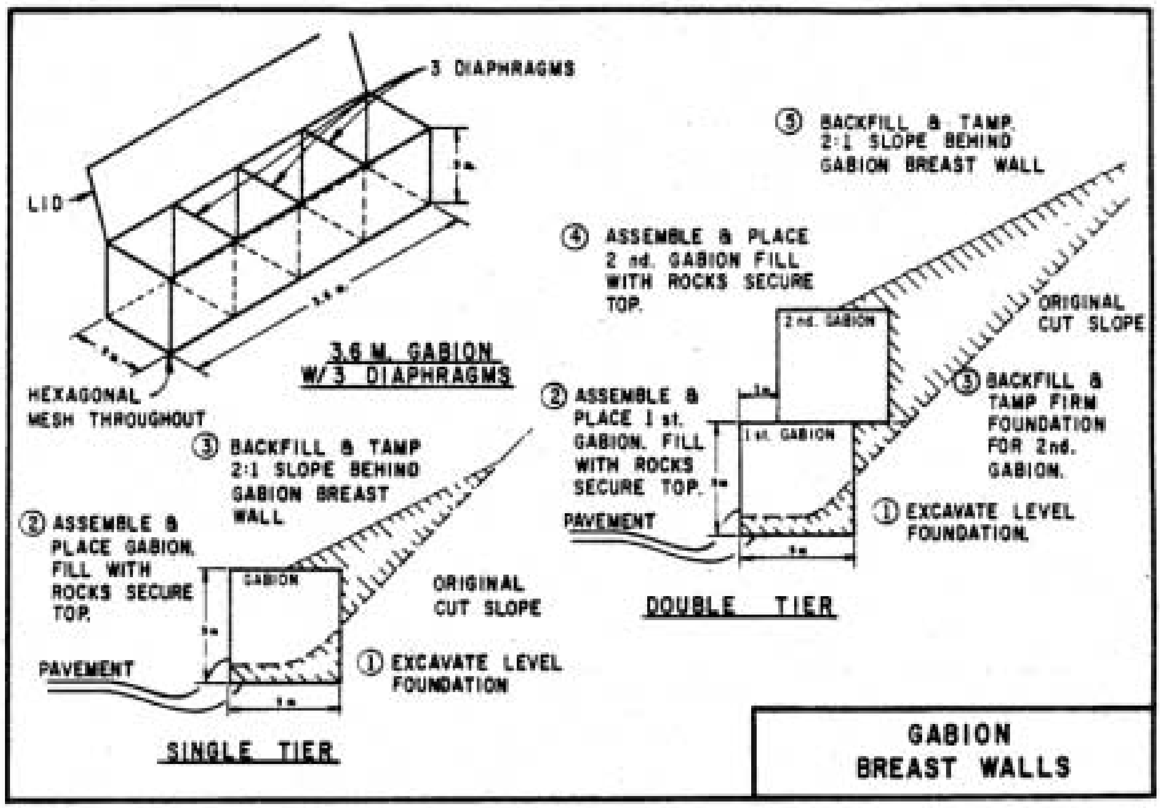


Figure 101. Low gabion breast walls showing sequence of excavation, assembly, and filling. (From White and Franks,1978)

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CHAPTER 6

ROAD CONSTRUCTION TECHNIQUES

6.1 Road Construction Techniques

6.1.1 Construction Staking

Prior to the construction activity the design information has to be moved from the plan to the ground. This is accomplished by staking. Slope stakes are an effective way to insure compliance with the design standards and to keep soil disturbance to an absolute minimum. Various staking methods can be employed. (Dietz et al., 1984; Pearce, 1960) The method discussed here is but one example.

Stakes, marking various road design points, are typically obliterated during the clearing and grubbing phase. In order to relocate the stakes (centerline, slope stakes) it is helpful to establish reference points outside the clearing limits. Reference points should be set at least 3 to 5 meters behind the uphill clearing limits. On the average, reference points (or RP's) should be set at least every 70 to 100 meters. Typically, reference points are placed at points where the centerline alignment can be easily re-established, such as points of curvature. Figure 102 shows the necessary stakes and stake notation needed by the equipment operator to construct a road.

Stakes are used by the equipment operator in locating where to begin cutting. If the selected starting point is too high, considerably more material has to be cut in order to construct the proper subgrade (Figure 103). For example, if the cut results in a 20 percent wider subgrade, approximately 50 percent more volume has to be excavated. (See Section 3.2.2.) If the cut is placed too low, an oversteepened cut slope or extra side casting may result, both of which are undesirable.

Starting the cut at the proper point becomes more important as the side slope increases. As a rule, slope stakes should be set when sideslopes exceed 40 to 45 percent depending on the sensitivity of the area and the operator's experience.

The use of RP's (Reference Points)or slope stakes for proper excavation is shown in Figure 104. Here, the engineer stands on the preliminary centerline of the construction grade and sights for the RP. A slope reading of 30 percent and a slope distance of 5.53 m is recorded. Converting the slope distance of 5.53 m to a horizontal distance of 5.30 m and to a vertical distance of 1.59 m allows the engineer to determine how much the "present" or preliminary centerline has to be shifted to conform with the design centerline. The RP tag requires 6.50 m horizontal distance to centerline with a vertical drop of 4.80 m. From that information, it can be seen that an additional 1.56 m [4.80 - (1.59 + 1.65) = 1.56] has to be cut and the present location has to be shifted by 1.2 m (6.50 - 5.30 = 1.20). Height of instrument or eye-level is assumed to be 1.65 m.

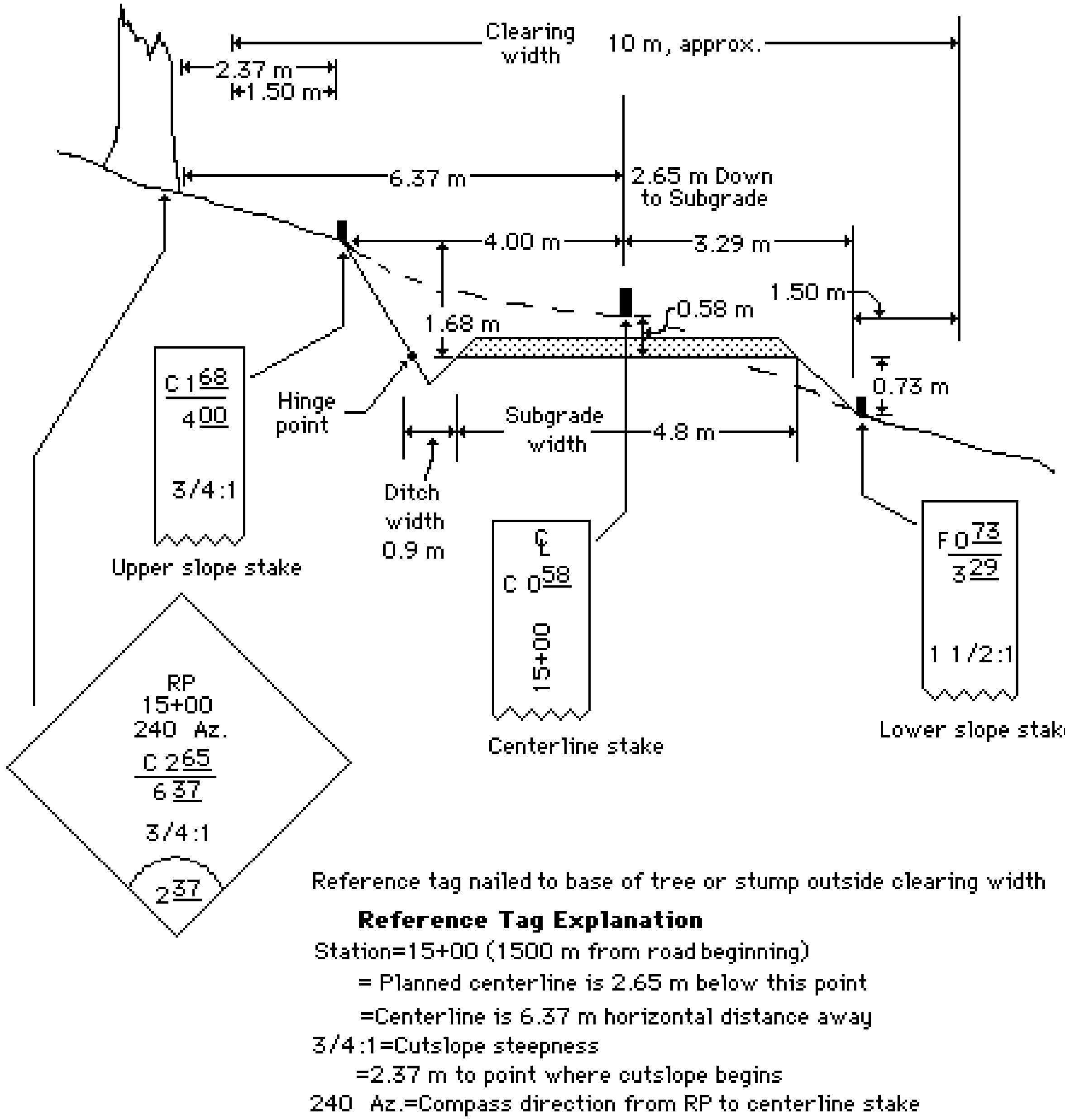

Figure 102. Road cross section showing possible construction information.

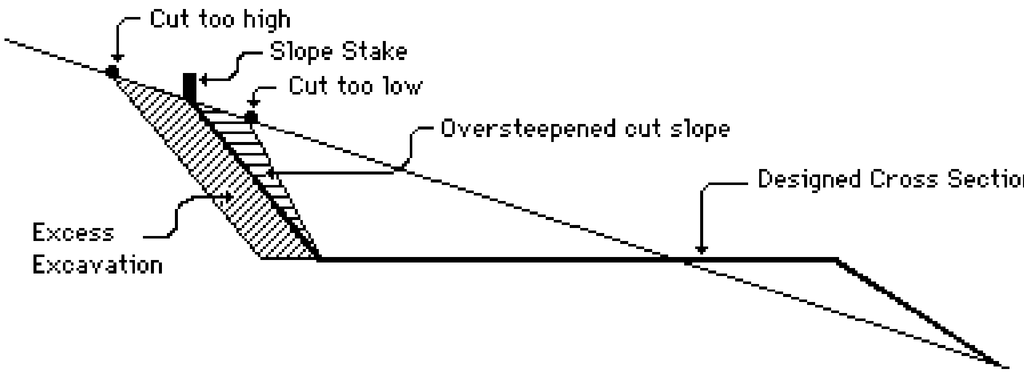

Figure 103. The effect of improperly starting the cut as marked by the slope stake. Starting the cut too high results in excess excavation and side cast. Starting the cut too low leaves an oversteepened cut bank.

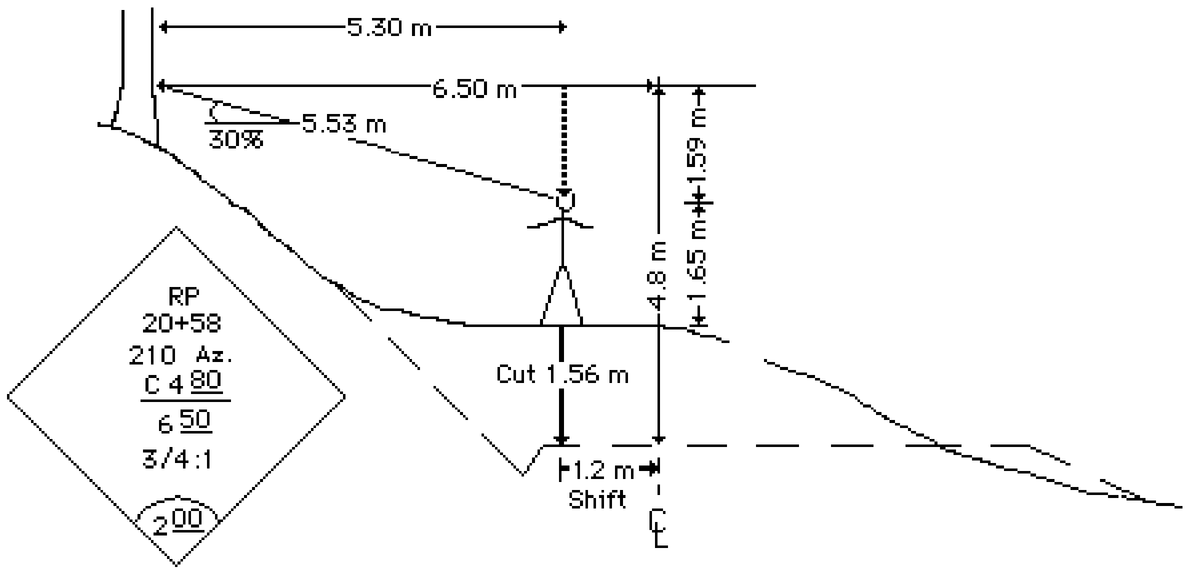

Figure 104. Construction grade check. Engineer stands on center of construction grade and sights to RP tag. Measured distance and slope allow for determination of additional cut.

6.1.2. Clearing and Grubbing of the Road Construction Area

Preparing the road right-of-way or construction area is referred to as clearing and grubbing. During the clearing phase, trees are felled. Grubbing refers to the clearing and removal of stumps and organic debris. Trees should be felled and cleared a minimum of 1 to 3 m from the top of the cut or toe of the fill (Figure 105). The logs can be decked outside the construction area (Figure 105, B to E) or skidded away.

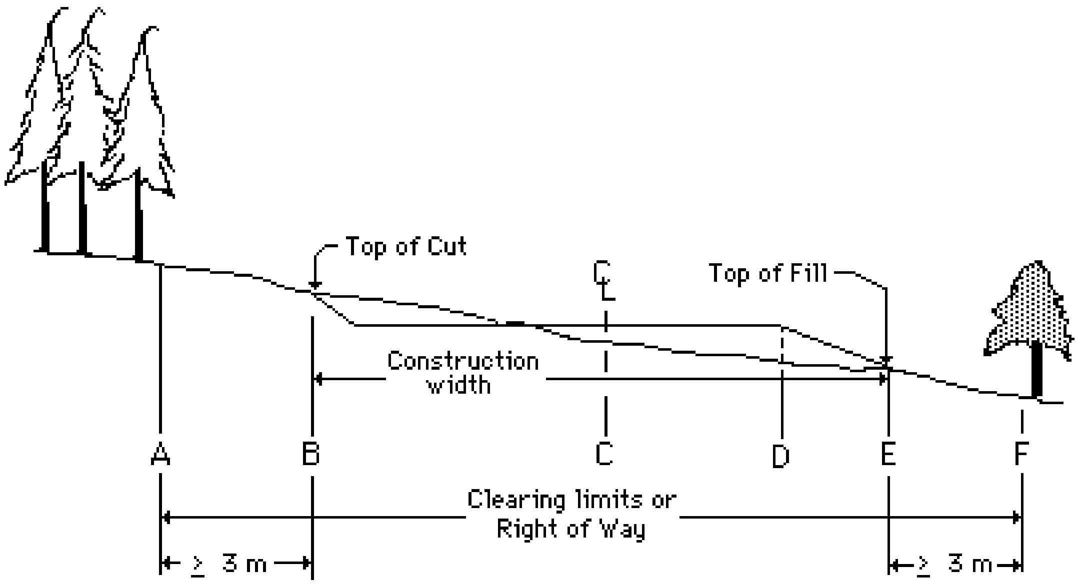

Figure 105. Clearing limits in relation to road bed widths. Significant quantities of organic materials are removed between B and E. Stumps are removed between B and D. Stumps may be left between D and E. Organic debris and removed stumps are placed in windrows at F to serve as filter strips (see Section 6.3.1).

This additional width between construction width and forest edge ensures that space is available to deposit organic debris outside the road construction width and that there is no overlap between forest edge and construction area.

A good construction practice to follow is to remove stumps that are within the construction width (Figure 105, B to E). Trees should be felled to leave a stump 0.8 to 1.2 m high. This helps bulldozers in stump removal by providing added leverage.

Organic overburden or topsoil typically has to be removed over the full construction width (Figure 105, B to D). This is especially true where organic layers are deep or considerable sidecast embankment or fills are planned. Organic material will decompose and result in uneven settlement and potential sidecast failure. Organic material should be deposited at the lower edge of the road (Figure 105, E to F). This material can serve as a sediment filter strip and catch wall (see Section 6.3.1), however care should be taken that this material is not incorporated into the base of the fill. Past road failures show that fill slope failures have been much more frequent than cut slope failures (70 percent and 30 percent , respectively). In most cases, poorly constructed fills over organic side cast debris was the reason for the failures.

During the grubbing phase, or preparation phase, a pioneer road is often constructed to facilitate equipment access, logging equipment movement, and delivery of construction materials, such as culverts. This is often the case when construction activities are under way at several locations. If pioneer roads are constructed, they are often built at the top of the construction width and are usually nothing more than a bulldozer trail. When considerable side hill fill construction is planned, however, the dozer trail should be located at the toe or base of the proposed fill. The trail will serve as a bench and provide a catch for the fill to hold on (Figure 106).

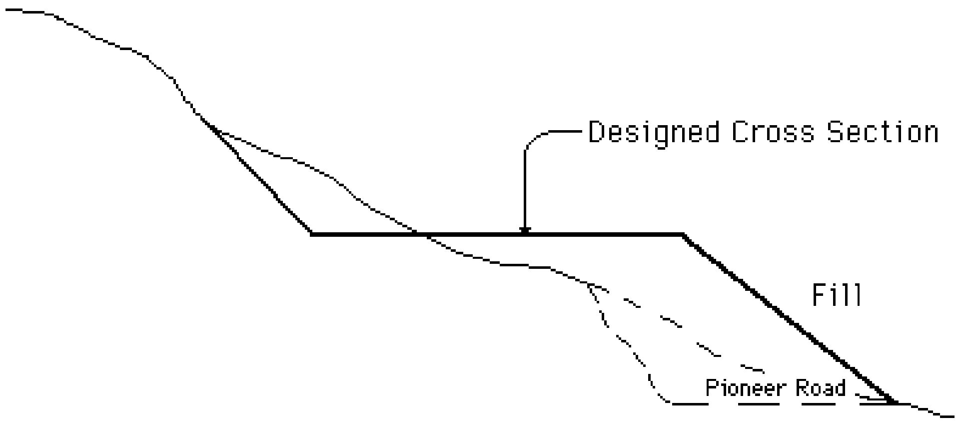

Figure 106. Pioneer road location at bottom of proposed fill provides a bench for holding fill material of completed road.

6.2 General Equipment Considerations

The method and equipment used in road construction is an important economic and design factor in road location and subsequent design. A road to be built by an operator whose only equipment is a bulldozer requires a different design than a road to be built by a contractor equipped with hydraulic excavator, scrapers, and bulldozer. Table 38 lists common road construction equipment and their suitability for the different phases of road construction. A bulldozer can be used in all phases of road construction from excavation and drainage installation to final grading. The front-end loader performs well in soft material. Front-end log loaders can be fitted with a bucket extending their usefulness under the correct conditions.

6.2.1 Bulldozer in Road Construction

Probably the most common piece of equipment in forest road construction is the bulldozer equipped with straight or U-type blades. These are probably the most economical pieces of equipment when material has to be moved a short distance. The economic haul or push distance for a bulldozer with a straight blade is from 17 to 90 meters depending on grade. The road design should attempt to keep the mass balance points within these constraints.

The road design should consider the following points when bulldozers are to be used for road construction.

1. Roads should be full benched. Earth is side cast and then wasted rather than used to build up side cast fills.
2. Earth is moved down-grade with the aid of gravity, not up-grade.
3. Fill material is borrowed rather than pushed or hauled farther than the economic limit of the bulldozer.

4. Rock outcrops should be bypassed. Unless substantial rock blasting is specified requiring drilling and blasting equipment, solid rock faces should be avoided. (This, however, is primarily a road locator's responsibility.)

table 45 Road construction equipment characteristics (from OSU Extension Service, 1983)

Criteria	Bulldozer	Front end Loader	Hydraulic excavator	Dump trucks or scrapers	Farm tractors
Excavation mode (level of control of excavated materials)	Digs and pushes; adequate control (depends on blade type)	Minor digging of soft material; lifts & carries; good control	Digs, swings, & deposits; excellent control; can avoid mixing materials long-distance material movement; excellent control	Scrapers can load themselves; 'top down' subgrade excavation; used for small quantities	Minor digging and carrying; good control because it handles
Operating distance for materials movement	91 m; pushing downhill preferred	91 m on good traction surfaces	23 m (limited to swing distance) be loaded	No limit except by economics; trucks must	31 m (approximately)
Suitability for fill construction	Adequate	Good	Limited to smaller fills	Good for larger fills	Not suitable
Clearing and grubbing (capacity to handle logs and debris	Good	Adequate	Excellent	Not suitable	Handles only small materials
Ability to install drainage features	Adequate	Digging limited to soft materials	Excellent	Not suitable	Adequate for small tasks
Operating cost per hour	Moderate, depending on machine size	Relatively low	Moderate to high, but productivity excellent	Very high	Low
Special limitations or advantages	Widely available ; can match size to job; can do all required with good	Cannot dig hard material; may be traction limited	Good for roads on steep hillsides; can do all required except spread rock for rock surfacing	Limited to moving material long distances; can haul rock, riprap, etc.	Very dependent on site conditions and operator skill

When using bulldozers, the practice of balancing cut and fill sections should be used only when:

- sideslopes do not exceed 45 to 55 percent

- proper compaction equipment is available such as a "grid roller" or vibrating or tamping roller

- fills have a sufficient width to allow passage of either compaction equipment or construction equipment, such as dump trucks.

Adequate compaction cannot be achieved with bulldozers alone. The degree of compaction exerted by a piece of equipment is directly related to its compactive energy or ground pressure. Effective ground pressure is calculated as the weight of the vehicle divided by the total ground contact area, or the area of tires or tracks in contact with the surface. Bulldozers are a low-ground pressure machine and therefore are unsuitable for this process. Ground pressure of a 149 kW (200 hp), 23 tonne bulldozer (Cat D7G, for example) is 0.7 bar (10.2 lb / in^2). By comparison, a loaded dump truck (3 axles, 10 m^3 box capacity) generates a ground pressure of 5 to 6 bar (72.5 to 87.1 lb / in^2).

Comparative production rates for various size bulldozers are shown in Figure 107. One should note that production curves are based on:

1. 100 % efficiency (60 minutes/hour),
2. power shift machine with 0.05 minute fixed time,
3. machine cuts for 15 m then drifts blade load to dump over a high wall,
4. soil density of 1,370 kg/m^3 (85.6 lb/ft^3) loose or 1790 kg/ m^3 (111.9 lb/ft^3)bank,
5. coefficient of traction $\geq$ 0.5, and
6. hydraulic controlled blades are used.

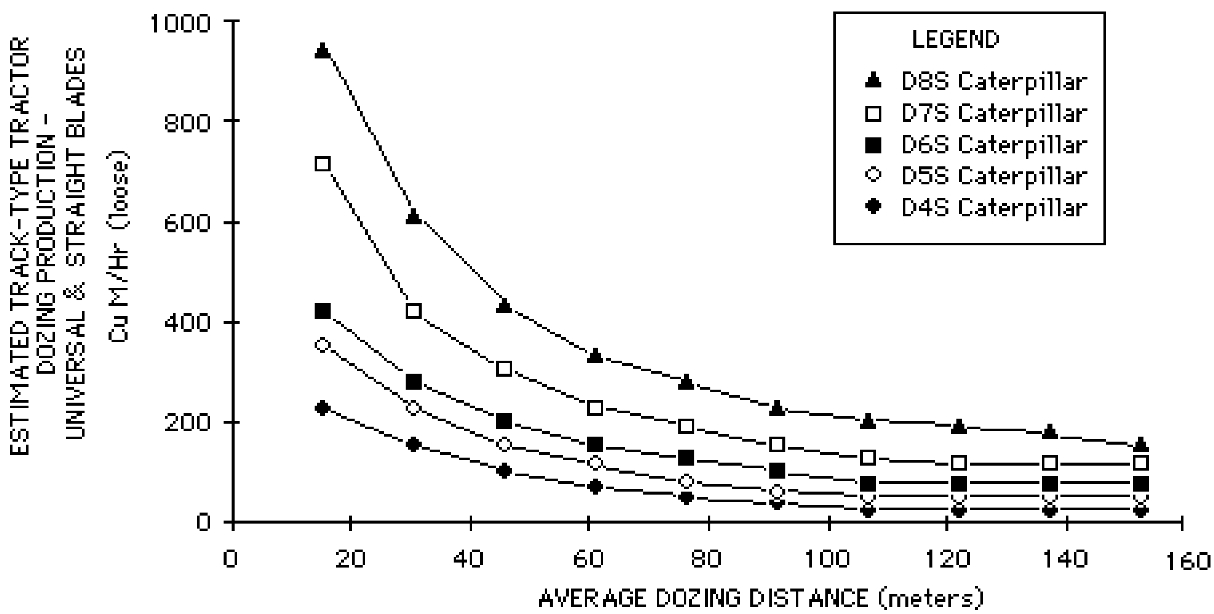

Figure 107. Maximum production rates for different bulldozers equipped with straight blade in relation to haul distance. (from Caterpillar Handbook, 1984)

The graph provides the uncorrected, maximum production. In order to adjust to various conditions which affect production, correction factors are given in Table 39. Adjustment factors for grade (pushing uphill or downhill) are given in Figure 108.

table 46 Job condition correction factors for estimating bulldozer earth moving production rates. Values are for track-type tractor equipped straight (S) blade. (Caterpillar Handbook, 1984)

TRACTOR	TRACK TYPE TRACTOR	WHEEL TYPE
OPERATOR		
Excellent	1.00	1.00
Average	0.75	0.60
Poor	0.60	0.50
MATERIAL		
Loose stockpile	1.20	1.20
Hart to cut; frozen --		
with tilt cylinder	0.80	0.75
without tilt cylinder	0.70	--
cable controlled blade	0.60	--
Hard to drift; "dead" (dry, non-cohesive material) or very sticky material	0.80	0.80
SLOT DOZING	0.60 - 0.80	--
SIDE BY SIDE DOZING	1.15 - 1.25	1.15 - 1.25
VISIBILITY --		
Dust, rain, snow, fog, darkness	0.80	0.70
JOB EFFICIENCY --		
50 min/hr	0.84	0.84
40 min/hr	0.67	0.67
DIRECT DRIVE TRANSMISSION (0.1 min. fixed time)	0.80	--
BULLDOZER*		
Angling (A) blade	0.50 - 0.75	--
Cushioned (C) blade	0.50 - 0.75	0.50 - 0.75
D5 narrow gauge	0.90	--
Light material U-blade (coal)	1.20	1.20

*Note: Angling blades and cushion blades are not considered production dozing tools. Depending on job conditions, the A-blade and C-blade will average 50-75% of straight blade production.

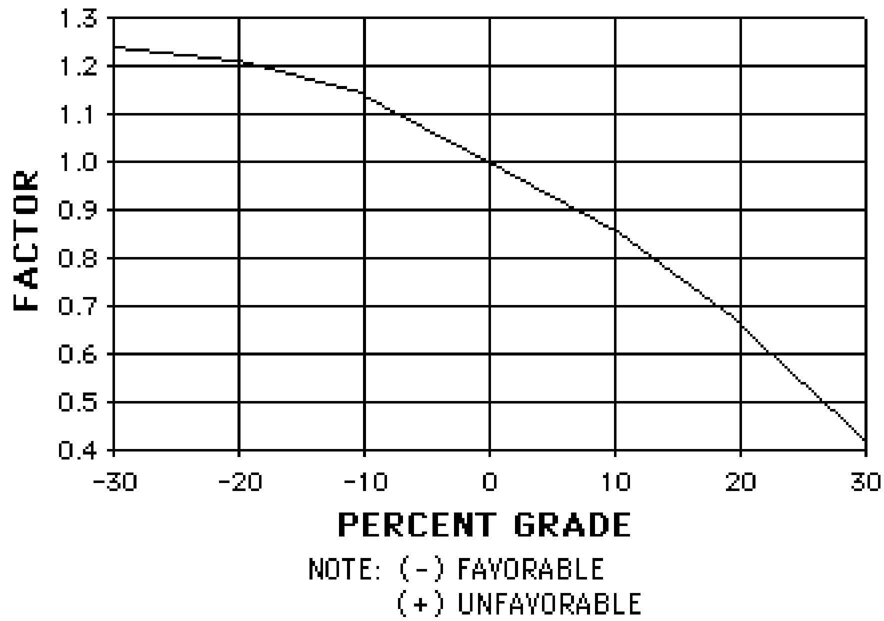

Figure 108. Adjustment factors for bulldozer production rates in relation to grade. (Caterpillar Performance Handbook, 1984)

EXAMPLE

Determine the average hourly production of a 200 hp bulldozer (D7) equipped with a straight blade and tilt cylinder. The soil is a hard packed clay, the grade is 15 percent favorable, and a slot dozing technique is used. The average haul or push distance is 30 m. The soil weight is estimated at 1,200 kg/m^3 loose, with a load factor of 0.769 (30 % swell). An inexperienced operated is used. Job efficiency is 50 min/hour.

The uncorrected maximum production is 430 m^3 loose/hour (from Figure 107) bulldozer curve D7S. Applicable correction factors are:

Job efficiency (50 min/hr)	0.84
Poor operator	0.60
Hard to cut soil	0.80
Slot dozing technique	1.20
Weight correction	0.87

Production = Maximum Production * Correction Factor

= (430 m^3 loose/hr) (0.84) (0.60) (0.80) (1.20) (0.87) = 181 m^3 loose/hour

Production (bank m^3) = (181 m^3 loose/hr) (0.769) = 139 bank m^3/hr

Production rates for bulldozers are also influenced by grade and side slopes. Percent change in haul distance with respect to changes in grade is shown in Table 40. As side slope increases, production rate decreases. Typical production rates for a medium sized bulldozer in the 12 to 16 tonne range (for example, Cat D6) are shown in Table 41.

table 47 Approximate economical haul limit for a 185 hp bulldozer in relation to grade. (Production rates achieved are expressed in percent of production on a 10 percent favorable grade with 30 m haul. (Pearce, 1978)

Haul distance					Grade (%)				
0	+5	+10	+15	+20			(meter)	10	-5
					percent				
15	54	72	90	126	161	198	234		
23	43								
30		44	56	76	100	122	144		
37			47						
45				54	70	86	102		
60				42	54	65	77		
75					43	52	62		
90						43	51		
105							43		

Bulldozers, to summarize, are an efficient and economical piece of equipment for road construction where roads can be full benched and excavated material can be side cast and wasted. It should be noted, however, that side cast material is not compacted. Typically, this type of construction equipment should only be used when: (1) side slopes are not too steep (ideally less than 50 percent), (2) adequate filter strips are provided along the toe of the fill, together with a barrier (natural or artificial) to catch side cast material, and (3) erosion is not considered to be a significant factor either as a result of soil type, precipitation regime, or both. Under these circumstances, bulldozers can be used on slopes steeper than 50 percent. If sideslopes exceed 60 percent, end hauling and/or use of a hydraulic excavator is highly recommended. Side cast wasting from bulldozer construction represents a continuous source for raveling, erosion, and mass failures. On steep slopes, bulldozers should only be used in combination with special construction techniques (trench excavation, see Section 6.3.1).

table 48 Average production rates for a medium sized bulldozer (12-16 tonnes) constructing a 6 to 7 m wide subgrade.

Sideslope (%)	0 - 40	40 - 60	>60
Production rate in meters/hour	12 - 18	8 - 14	6 - 9

6.2.2 Hydraulic Excavator in Road Construction

The hydraulic excavator is a relatively new technology in forest road construction. This machine basically operates by digging, swinging and depositing material. Since the material is placed, as opposed to pushed and/or sidecast, excellent control is achieved in the placement of the excavated soil. This feature becomes more important as the side slope increases. Fill slope lengths can be shortened through the possibility of constructing a catch wall of boulders along the toe of the fill. This feature is particularly important when side slopes increase to over 40 percent.

Mass balance along the centerline is limited to the reach of the excavator, typically about 15 to 20 meters. However, because of excellent placement control, construction of a balanced cross section can be achieved with considerably less excavation. Raveling disturbance and erosion is reduced as well because of lesser excavation and little or no downhill drifting of embankment material (Figure 109).

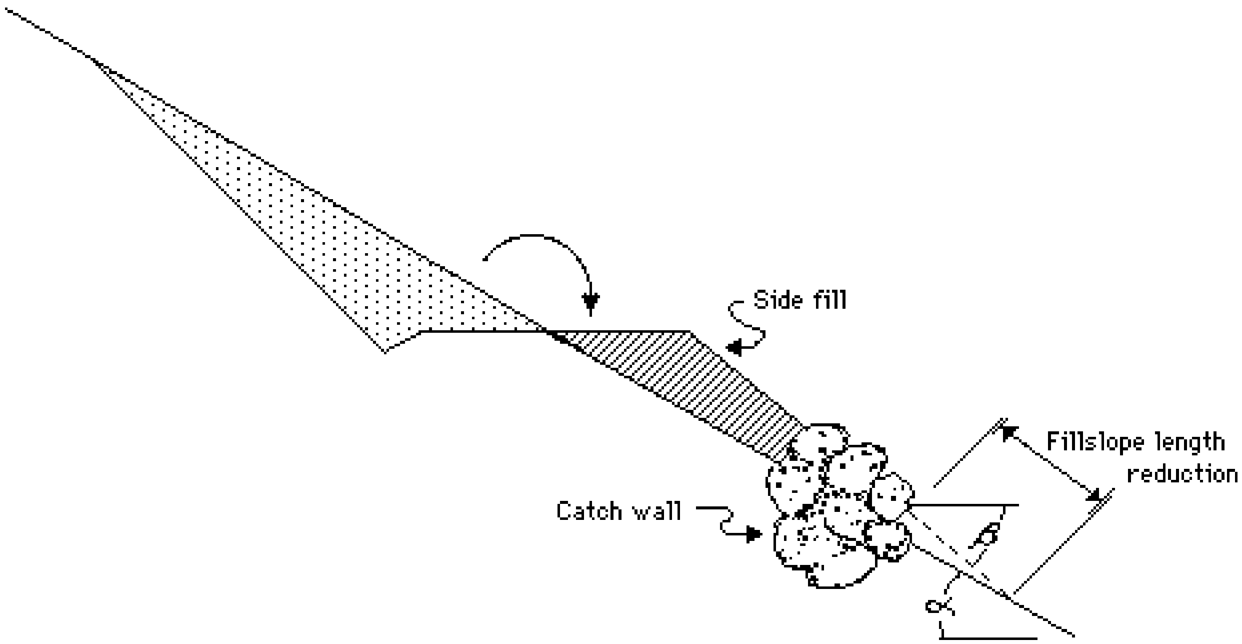

Figure 109. Fill slope length reduction by means of catch-wall at toe of fill. (See also Figure 55)

Production rates for hydraulic excavators are given in Table 41. Production rates are shown for three different side slope classes. The values given are for a medium sized excavator with a 100 kW power rating (e.g., CAT 225, Liebherr 922).

table 49 Production rates for hydraulic excavators in relation to side slopes, constructing a 6 to 7 m wide subgrade.

Side slope %	Production rate meter/hour
0 - 40	12 - 16
40 -60	10 -13
> 60	8 - 10

The excavator production rate approaches the dozer production rate as side slope increases. There are now indications that excavator production rates are higher than dozer production rates on slopes steeper than 50 percent. This difference will increase with increased rock in the excavated material. The bucket of the excavator is much more effective at ripping than the dozer blade. Excavators are also more effective at ditching and installing culverts.

6.3 Subgrade Construction

6.3.1 Subgrade Excavation with Bulldozer

Proper construction equipment and techniques are critically important for minimizing erosion from roads during and after the construction. There are clear indications that approximately 80 percent of the total accumulated erosion over the life of the road occurs within the first year after construction. Of that, most of it is directly linked to the construction phase.

In order to keep erosion during the construction phase to an absolute minimum, four elements must be considered.

1. Keep construction time (exposure of unprotected surfaces) as short as possible.

2. Plan construction activities for the dry season. Construction activities during heavy or extended rainfall should be halted.

3. Install drainage facilities right away. Once started, drainage installation should continue until completed.

4. Construct filter strips or windrows at the toe of fill slopes to catch earth slumps and sheet erosion (see Section 6.3.5).

The formation or construction of the subgrade begins after the clearing and grubbing (stump removal) phase. Three basic construction techniques are commonly used: side cast fills and/or wasting, full bench construction with end haul, and balanced road sections with excavation incorporated into layered fills (Figure 110).

Side cast and wasting traditionally has been the most common construction method. It also has been responsible for the highest erosion rates and making large areas unproductive. In this method, most if not the full road width is placed in undisturbed soil (Figure 110). Excavated material is side cast and wasted, rather than incorporated into the road prism. The advantage is uniform subgrade and soil strength. It is unlikely that the travelled road width will be involved in fill failures. An obvious disadvantage is the potential for erosion of loose, unconsolidated side cast material.

Side cast construction is the preferred construction method for bulldozers. The bulldozer starts the cut at the top of the cutslope and excavates and side casts material until the required road width is achieved (Figure 111). It is important that the cut be started exactly at the "top of cut" construction stake (point B, Figure 105) and the cutting proceed with the required cut slope ratio (see Section 6.1.4). Depending on the type of blade (S - or U- blade) the bulldozer can push or drift excess or excavated material up to 100 meters in front of the blade along the road section to deposit it in a stable place.

As the side slope becomes steeper, less and less of the side cast material is incorporated into the side fill. Bulldozer equipment has very little placement control especially on steeper side slopes where " sliver-fills" often result (Figure 112). These fills perform marginally, at best, and "full benching" with side cast and wasting of excavated material is preferred by many road builders. The result is a stable road surface but with a very unstable waste material fill.

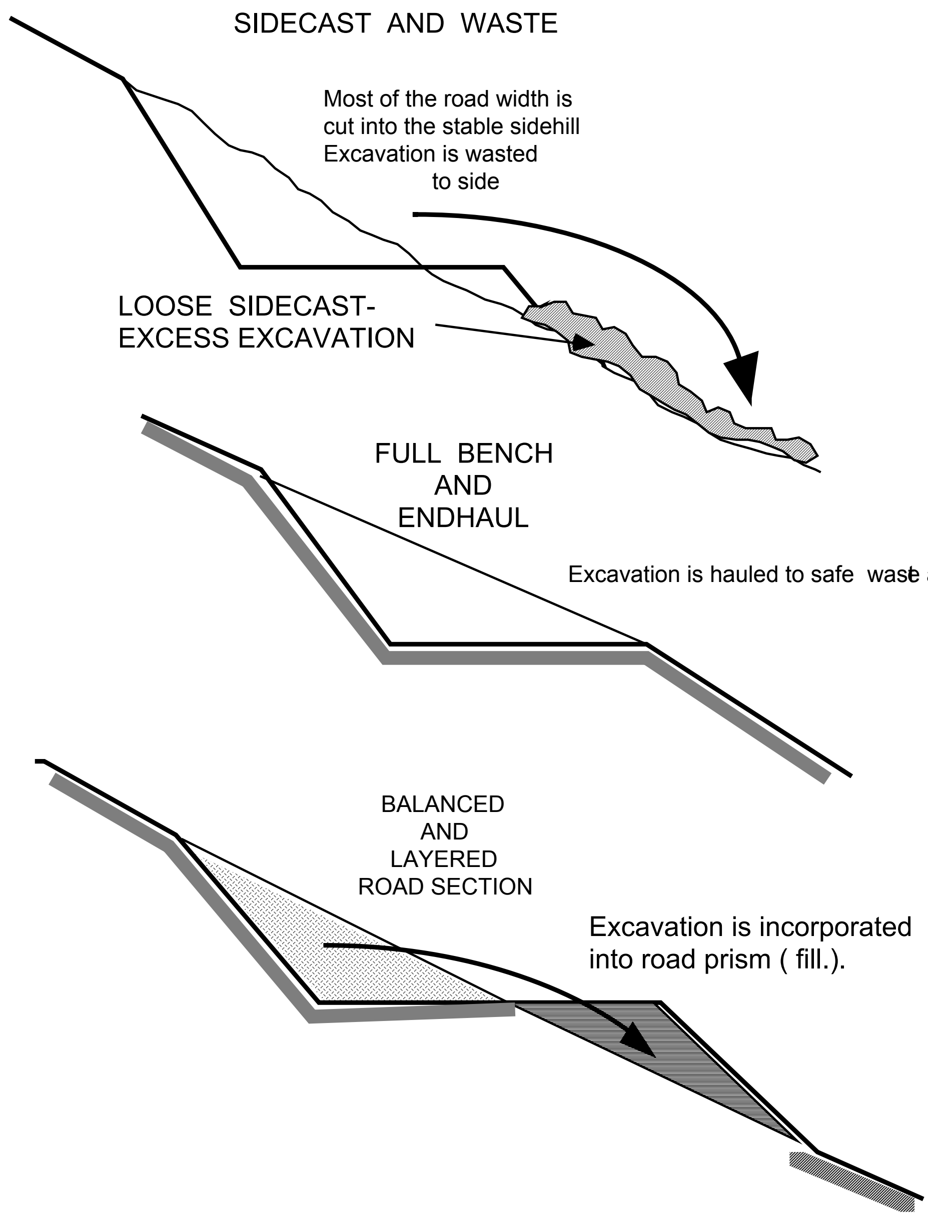

Figure 110. Three basic road prism construction methods

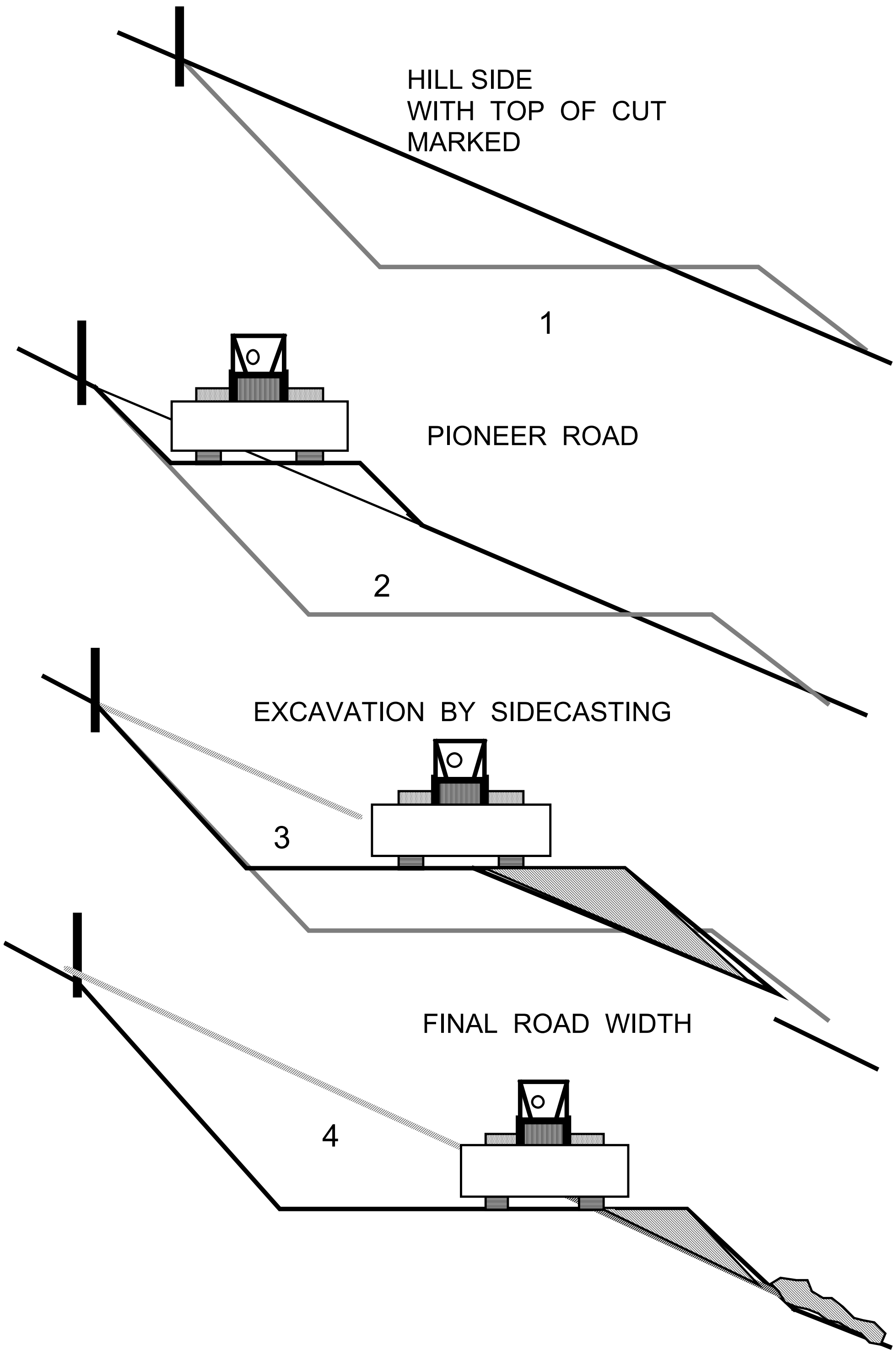

Figure 111. Road construction with a bulldozer; The machine starts at the top and in successive passes excavates down to the required grade. Excavated material is side cast and may form part of the roadway.

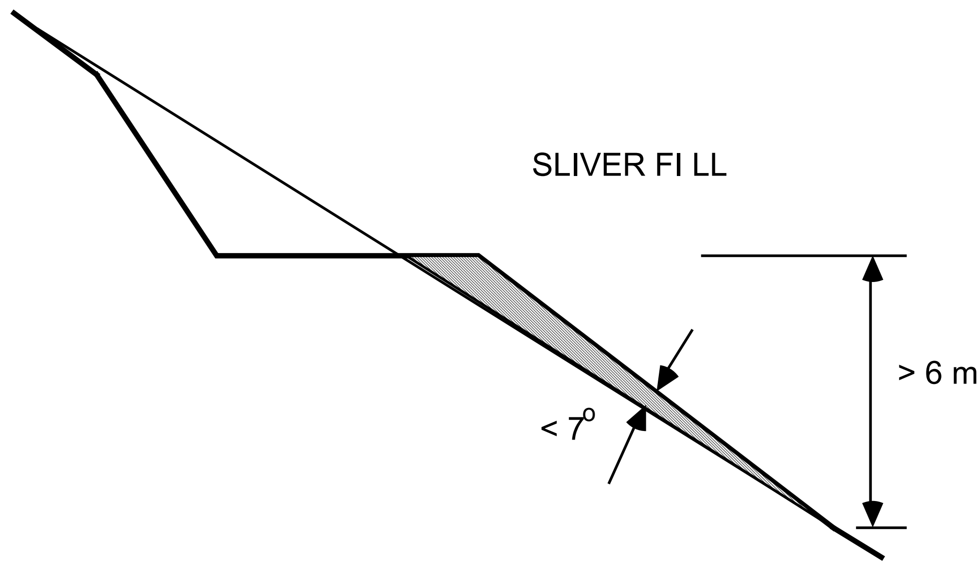

Figure 112. Sliver fills created on steep side slopes where ground slope and fill slope angles differ by less than 7º and fill slope height greater than 6.0 meters are inherently unstable.

Side cast or wasted material cannot remain stable on side slopes exceeding 60 to 70 percent. Under such conditions excavated material has to be end hauled to a safe disposal area. This requires dump trucks and excavators or shovels for loading and hauling.

Unwanted side cast may result from dozer excavation on steep side slopes because of lack of placement control. In order to contain side cast loss within the construction width of a full bench road the so-called "trench-method" has been successfully used in the Pacific Northwest (Nagygyor, 1984). In this method the right-of-way timber is felled parallel to the road centerline. Trees and stumps are not removed. They will act as a temporary retaining wall for loose, excavated material (Figure 113). A pioneer road is built at the top of the cut by drifting material against and on top of the felled trees. Initial excavation and side cast loss can therefore be kept to a minimum. When rock is encountered, dirt drifted against or on top of trees will form a temporary bridge to allow passage of construction equipment.

Actual excavation is started about 10 to 12 meters from the loader by cutting a blade-wide trench and drifting the material towards it. Loose material which escapes during this process is caught by the felled trees and slash. As the cut gets deeper material will fall inside the trench from both sides (Figure 113). Debris, stumps, tops and branches are pushed and loaded together with the excavated material,if it is not placed in designated fills. Otherwise it can be separated out at this point.

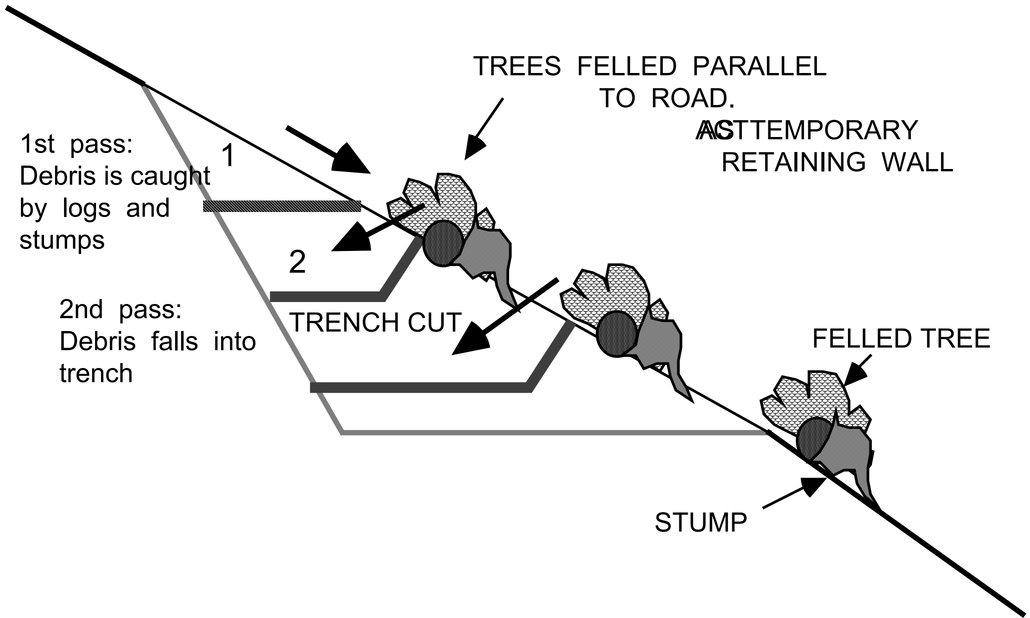

Figure 113. Trench-excavation to minimize sidehill loss of excavation material. Debris and material falls into trench in front of the dozer blade. Felled trees and stumps are left to act as temporary retaining walls until removed during final excavation.

6.3.2 Fill Construction

Fill construction is required to cross draws, creeks, flats or swampy areas and when excess excavation has taken place. Road fills support traffic and therefore must withstand considerable abuse. Only mineral soil, free of organic debris such as stumps, tree tops and humus should be used. Fills should be constructed and built up in layers (Figure 114). Each layer, or lift, should be spread and then compacted. Lift height before compaction depends on the compaction equipment being used. Typically lift height should be about 30 cm and should not exceed 50 cm. A bulldozer is not a good machine for compacting fills because of their low ground pressure characteristics. Fills across draws or creeks are especially critical since they may act as dams if the culvert should plug up. It is considered poor practice to build fills by end dumping instead of layering and compacting (Figure 115).

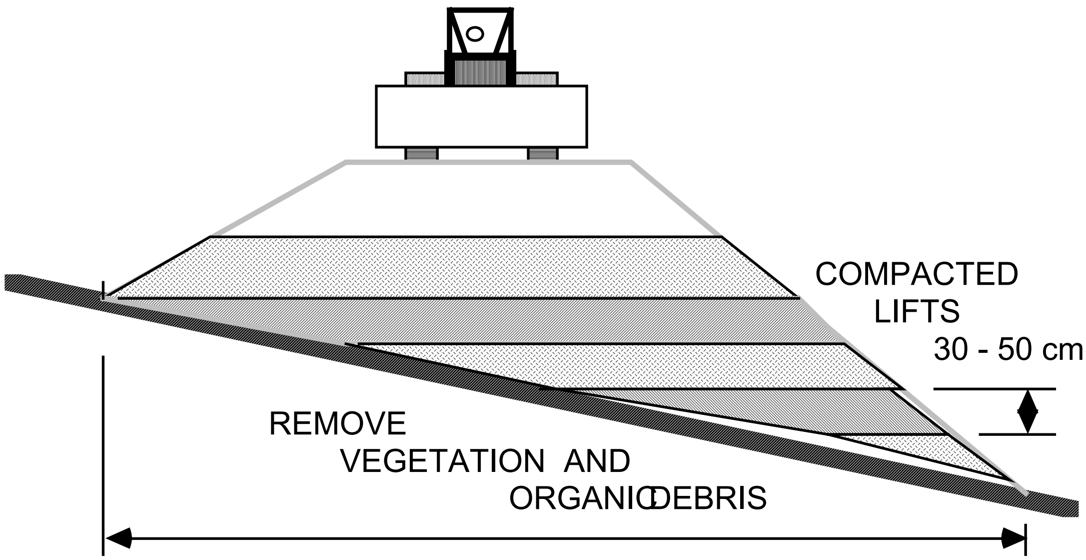

Figure 114. Fills are constructed by layering and compacting each layer. Lift height should not exceed 50 cm. Compaction should be done with proper compaction equipment and not a bulldozer (from OSU Ext. Service 1983).

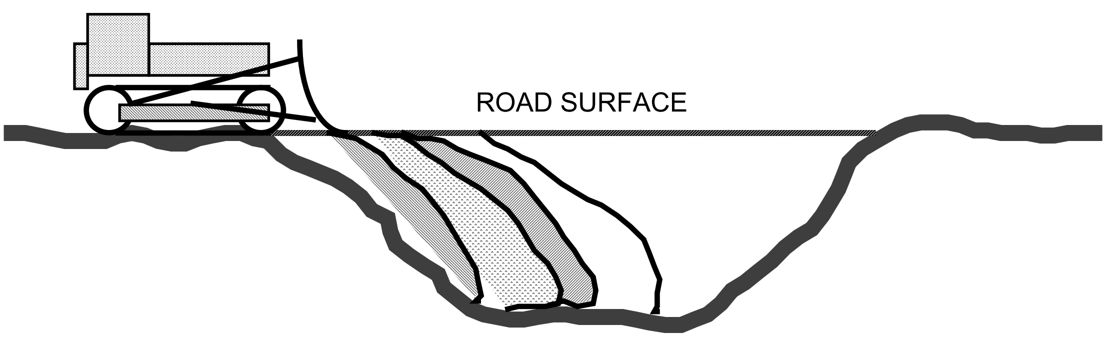

Figure 115. Fills that are part of the roadway should not be constructed by end dumping. (from OSU Ext. Service, 1983).

6.3.3 Compaction

Proper compaction techniques result in significant cost reduction and reductions in erosion. Erosion potential is directly proportional to the excavation volume especially if it is side cast in unconsolidated and loose fills. Conventional side cast techniques where most of the road surface is excavated into a stable hill side results in approximately 25 to 35 percent more excavated material when compared to "balanced" road design and construction where the excavation is incorporated into the road prism. In the former case, most if not all of the excavated material is wasted as loose side cast material readily available for erosion. In the latter case, it has been incorporated into the fill, properly compacted, and presumably unavailable for erosion.

The key to a stable, balanced road design is proper compaction of fill material. Haber and Koch (1982) quantified costs for erosion and compaction for several types of sediment control treatments on roads in southwest Idaho. This study represents an excellent example of applying uniform criteria to examine differences between standard and non-standard construction techniques.

Costs were initially determined for each activity using two methods: (1) local (Boise) labor and equipment rates, taxes, insurance, and servicing (repair and maintenance) including 10 percent profit and risk margin, and (2) Regional Equipment Blue Book Guidebook which include margins for profit and risk, fuel, oil,lubrication, repairs, maintenance, insurance, and incidental expenses. After actual costs for each activity were calculated, average cost per unit and average crew cost was determined based on design quantities. A comparison was then made between actual costs for "non-standard" treatments and actual costs of standard treatments.

Average observed production rates for all activities were calculated for use in predicting time and costs associated with "non-standard" construction techniques. Figure 116 illustrates an example of their results in determining the cost of three different methods of embankment placement. These methods are: (1) side cast embankments with no compactive effort, (2) layer placed (less than 30 cm (12 in) thick) embankments in which each layer is leveled and smoothed before each subsequent layer is placed (some compaction is obtained during the leveling process as the bulldozer passes over the material), and (3) controlled compaction in which embankments are placed in layers (less than 20 cm (8 in) thick) followed by compaction with water and vibratory roller to achieve relative density of 95 percent.

As expected, side cast embankment construction per volume costs the least and controlled compaction the most. (Road 106781 was shorter and only a small quantity of earth was moved resulting in a higher unit cost.) Total cost, however, for a road expressed in cost per unit length may be very similar for side cast embankment and layered placement considering the fact that total excavation volume may be up to 35 percent less for the latter case. As mentioned before, most of this excavated material is now consolidated rather than loose. Combined with proper fill slope surface treatment and filter windrows very little erosion can be expected.

It is worth noting that production rates of manual labor for excavation work are generally 3.8 to 4 m^3 (5 $yd.^3$) of dirt during eight hours of work (Sheng,1977). However, these rates will vary widely depending on terrain, soil, environmental, and psychological conditions of the work crew.

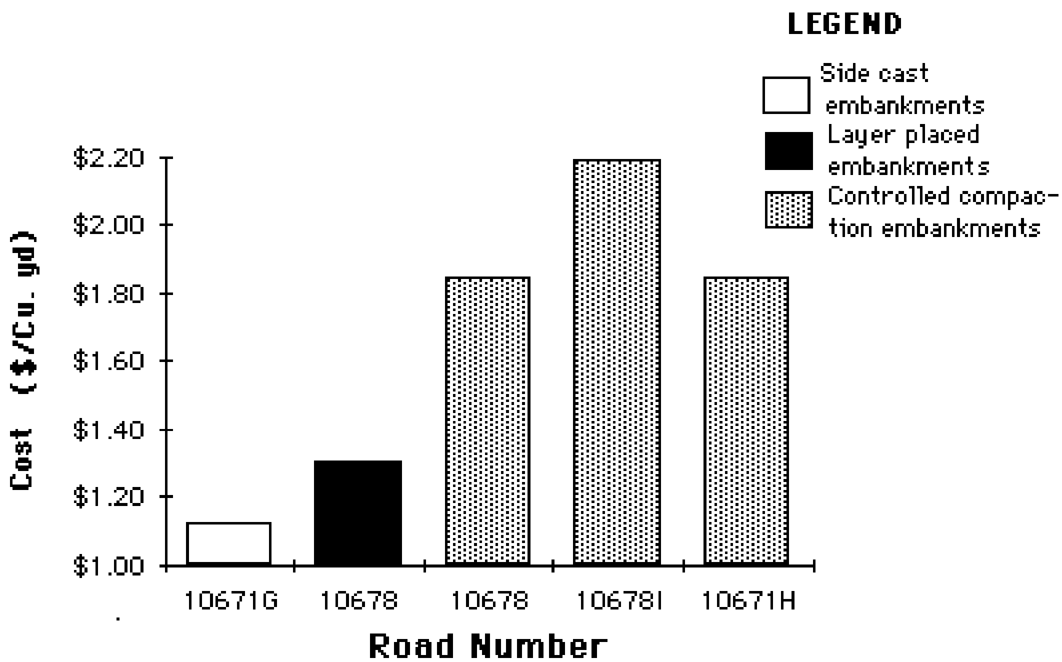

Figure 116. Excavation cost comparison for three different embankment construction techniques (1 cu.yd. = 0.9 m^3). (after Haber and Koch,1983)

6.3.4 Subgrade Construction with Excavator

Excavators are becoming more and more common in road construction. Because of their excellent placement control of excavated material, they are ideal machines for construction under difficult conditions. The backhoe or excavator should be the preferred machine on steep side slopes. The construction sequence differs from the bulldozer approach and is explained below.

The excavator works from a platform or pioneer road at the lower end of the finished road.

Ist pass: Pioneering of log and stump removal accomplished in the fist pass. Just enough overburden is moved to provide a stable working platform (Figure 117). Logs are piled at the lower side of the clearing limit.

2nd pass: After completion of the first pass the operator begins retracing its path. During this pass unsuitable material is stripped and placed below the toe of the fill (Figure 118).

3rd pass: During the third pass, now working forward again, the exposed mineral soil is dug up for the embankment construction. At the same time a ditch is prepared and the cut slope smoothed and rounded. The portion of pioneer road or platform, which consist of organic debris, is outside the load bearing road surface fill. (Figure 119).

On steep side slopes the excavator is able to place large boulders at the toe of the fill (in a ditch line) and place excavated material against it (Figure 55 and 109). Total excavation and exposed surface area can be kept to a minimum.

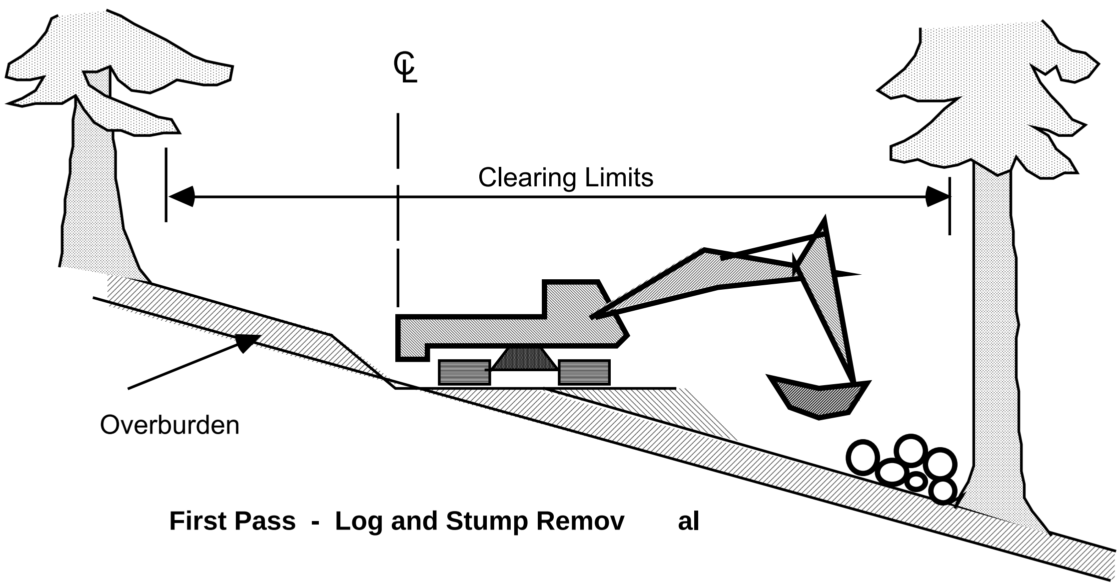

Figure 117. First pass with excavator, clearing logs and stumps from construction site. Working platform or pioneer road just outside of planned road surface width.

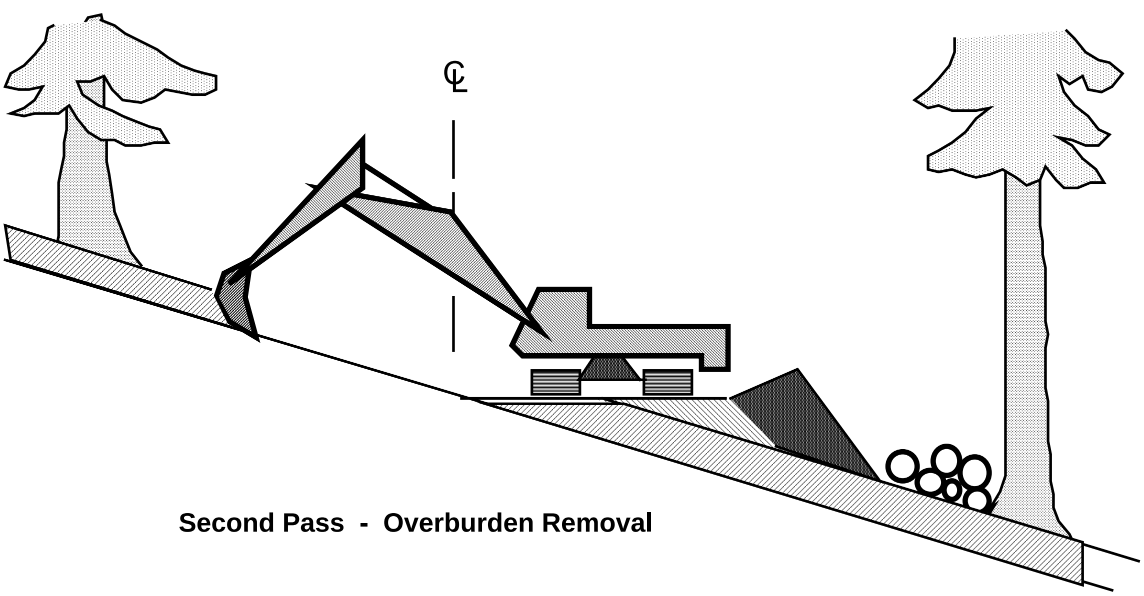

Figure 118. Second pass with excavator, removing or stripping overburden or unsuitable material and placing it below pioneer road.

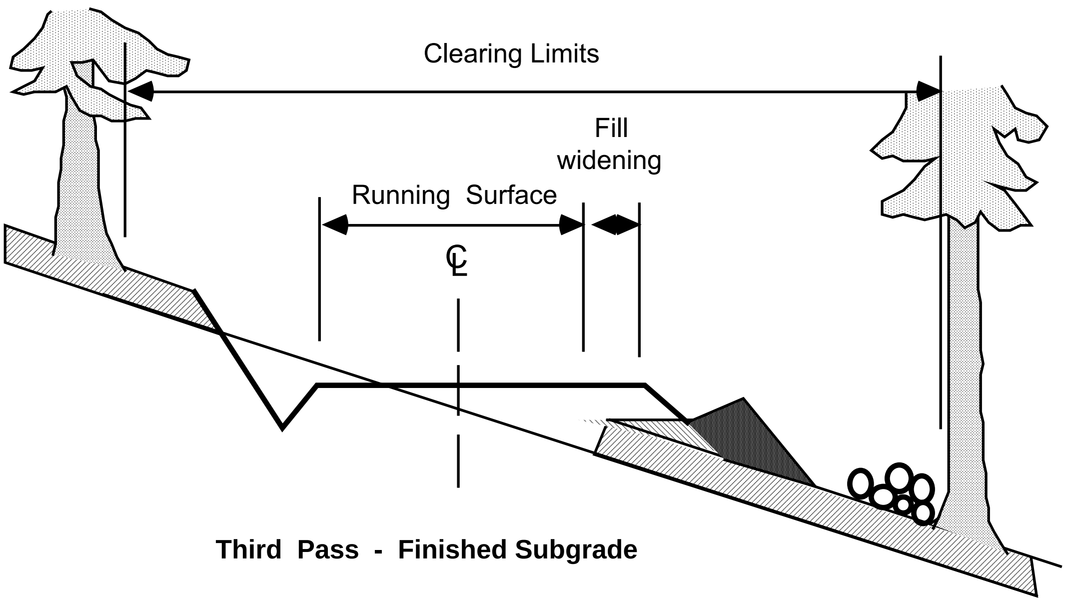

Figure 119. Third pass, finishing subgrade and embankment fill over pioneer road. Road side ditch is finished at the same time.

6.3.5 Filter Windrow Construction

Erosion from newly built fill slopes can effectively be trapped through filter strips or windrows made of slash and placed at the toe of the fills. This measure is particularly important and effective where the road crosses a draw or creek. The effect of such filter strips on soil loss from new fill slopes is shown in Table 42. Fill erosion from newly built slopes can be reduced by more than 95 percent over a 3 year period (Cook and King ,1983). This time period is sufficient in most cases to allow for other measures such as surface seeding, mulching, or wattling to become established.

table 50 Fill slope erosion volume for windrowed and nonwindrowed slopes during a 3 year period following construction (Cook and King,1983).

Slope Class*	Filter Windrow (no windrow)	Unprotected
	----------- m^3 / 1000 m ------------	
1	0.30	33.29
2	0.65	64.30

*class 1: vertical fill height < 3 meter
class 2: " " " 3 to 6 meter

<u>Construction of filter strips:</u>

1. Suitable material from the clearing and pioneering activity should be stockpiled at designated areas either above or below the clearing limits. Slash should consist of tops, limbs and branches, not to exceed 15 cm in diameter and 3,5 m in length. Stumps and root wads are not suitable material and should be excluded.

2. Windrows are constructed by placing a cull log (reasonably sound) on the fill slope immediately above and parallel to the toe of the fill (Figure 120) for the fill material to catch against. The log should be approximately 40 cm in diameter and should be firmly anchored against undisturbed stumps, rocks or trees.

3. Slash should be placed on the fill above the cull log. The resulting windrow should be compacted, for example, by tamping it with the bucket of an excavator. It is important that part of the slash be embedded in the top 15 cm of the fill. Filter strips are built during subgrade construction in order to maximize their effectiveness. Care should be taken so as not to block drainage structures (outlets) or stream channels.

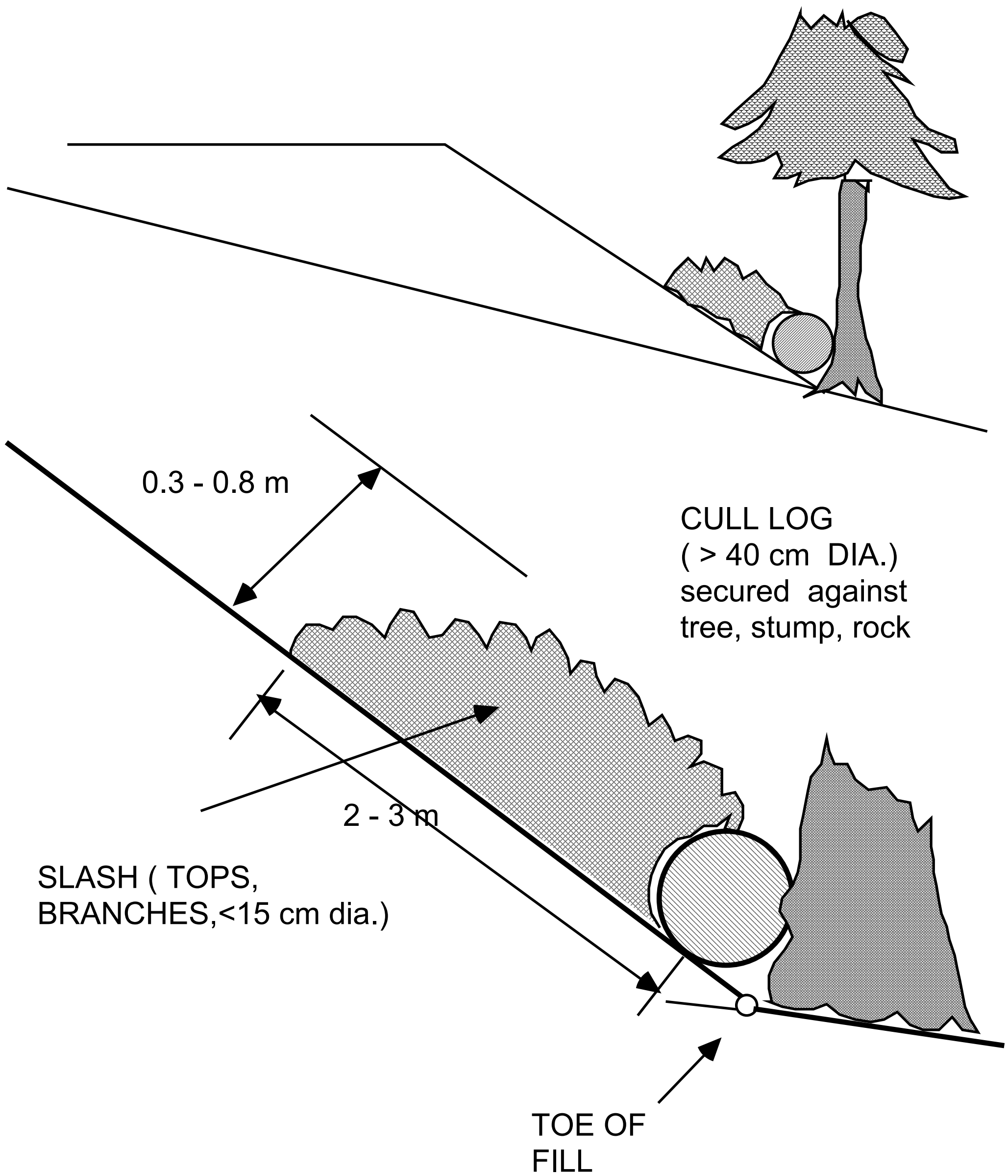

Figure 120. Typical filter windrow dimensions built of slash and placed on the fill slope immediately above the toe. The windrow should be compressed and the bottom part embedded 15 cm in the fill slope. (after Cook and King, 1983)

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