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. Translational fill failure (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.

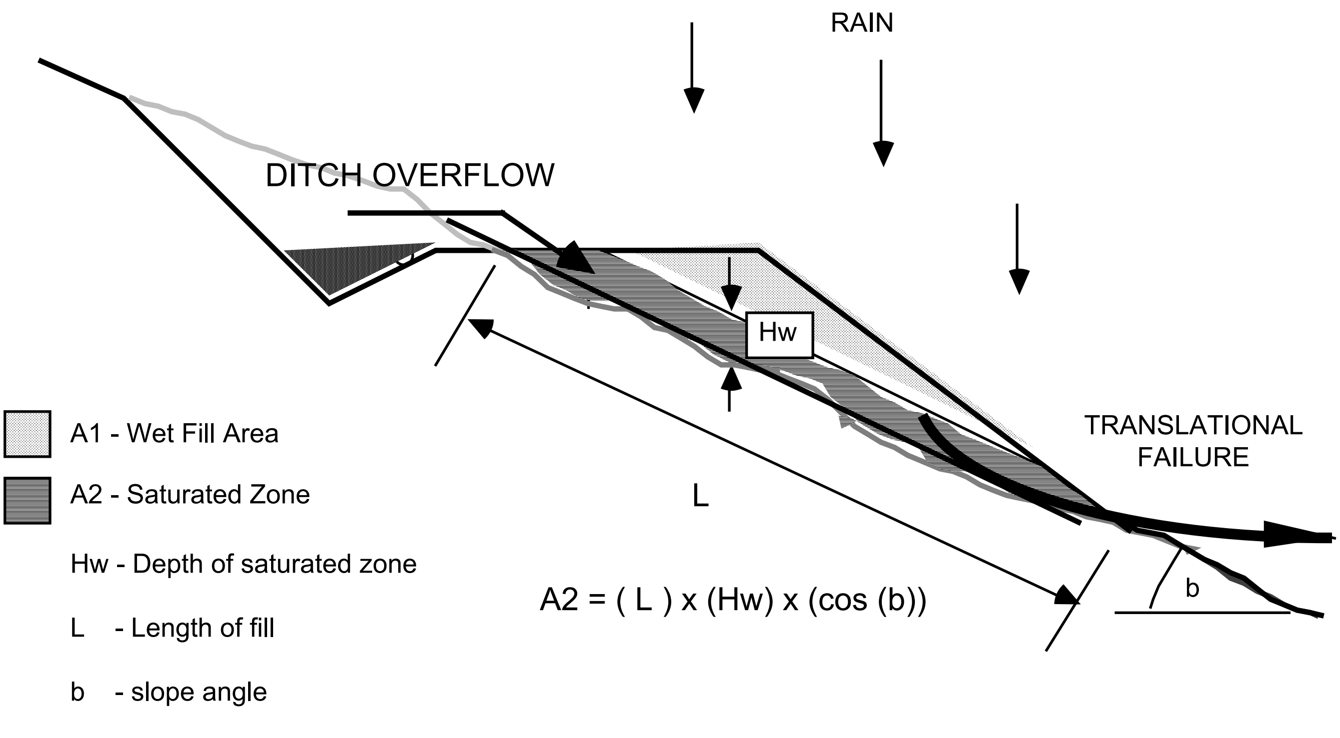


Figure 42. Translational or wedge failure brought about by saturated zone in fill. Ditch overflow or unprotected surfaces are often responsible.

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.

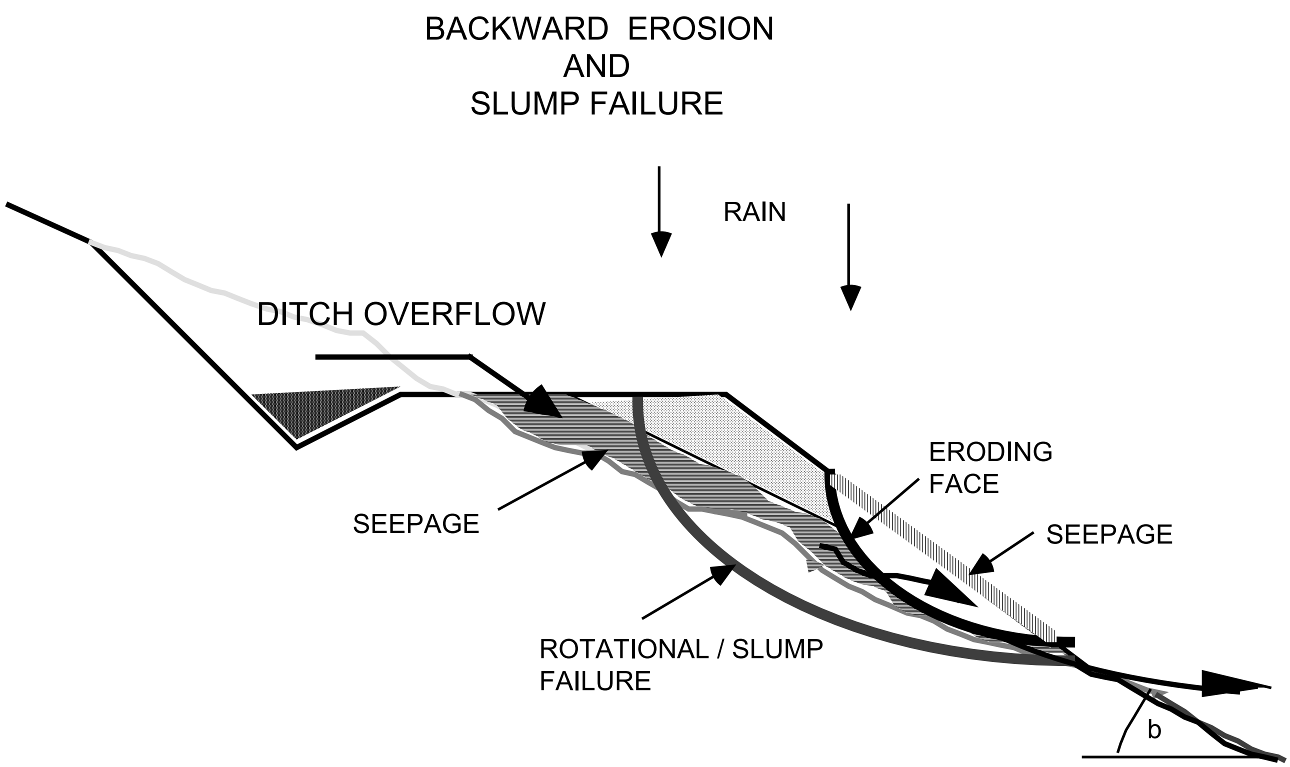


Figure 43. Fill failure caused by backward erosion at the toe of the fill due to excessive seepage and an unprotected toe.

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° which corresponds to a rise:run ratio of 1:1.5. Therefore, a road prism on side slopes steeper than 50 - 55% (26 - 29°) 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° (1:1.5).

Practical considerations suggest that fill slope angle and ground slope angle should differ by at least 7°. 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.

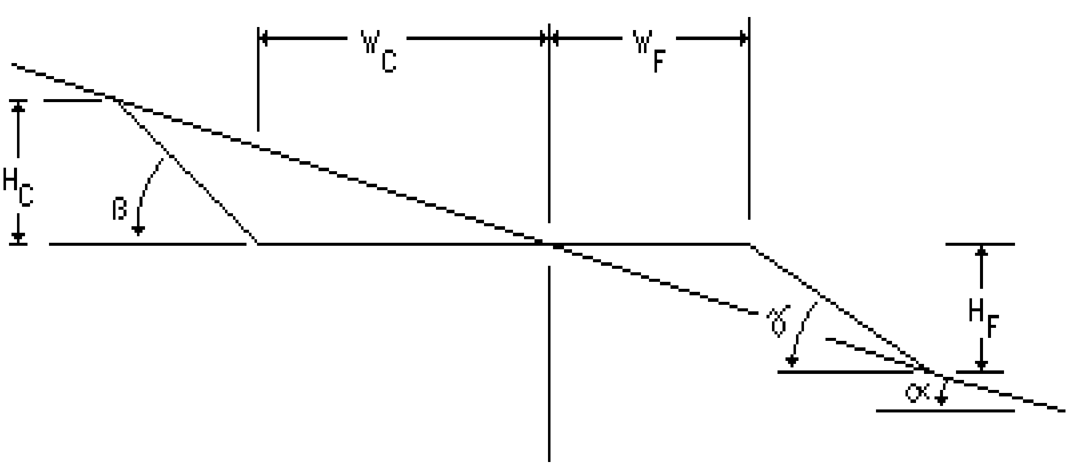


Figure 44. Elements of road prism geometry

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:

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

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

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

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

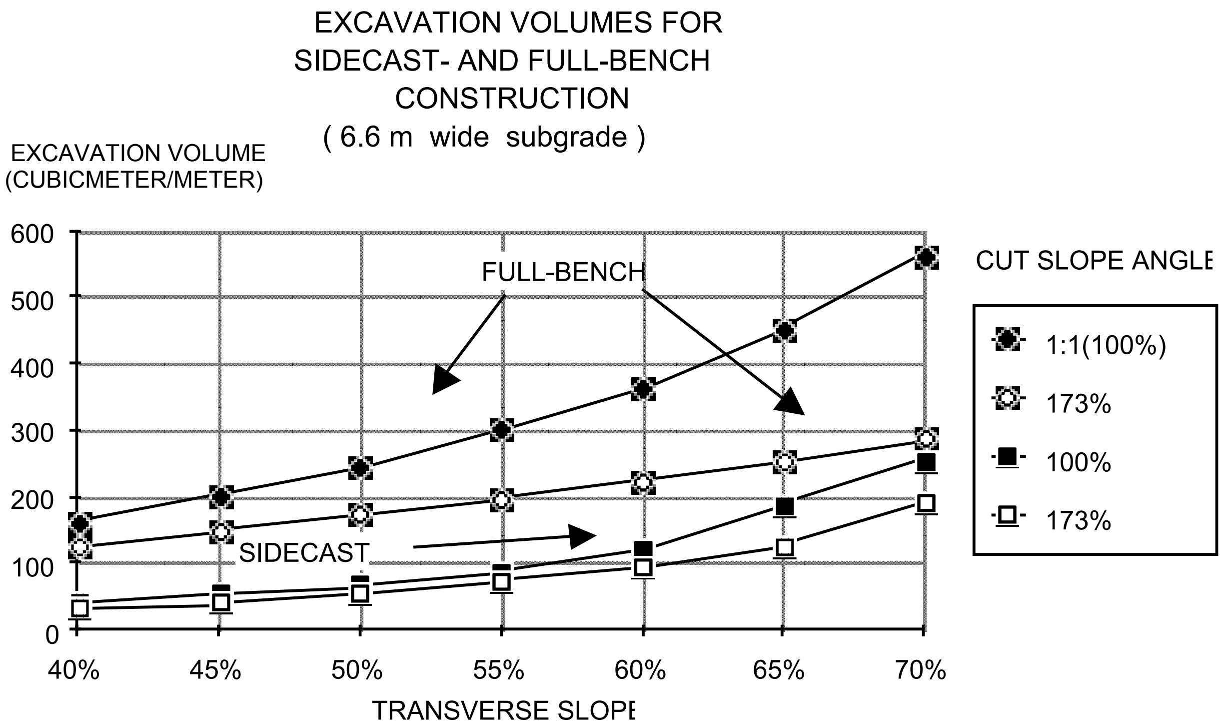


Figure 45. Required excavation volumes for side cast and full bench construction as function of side slope. Assumed subgrade width 6.6 m and bulking factor K = 1.35 (rock).

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%.

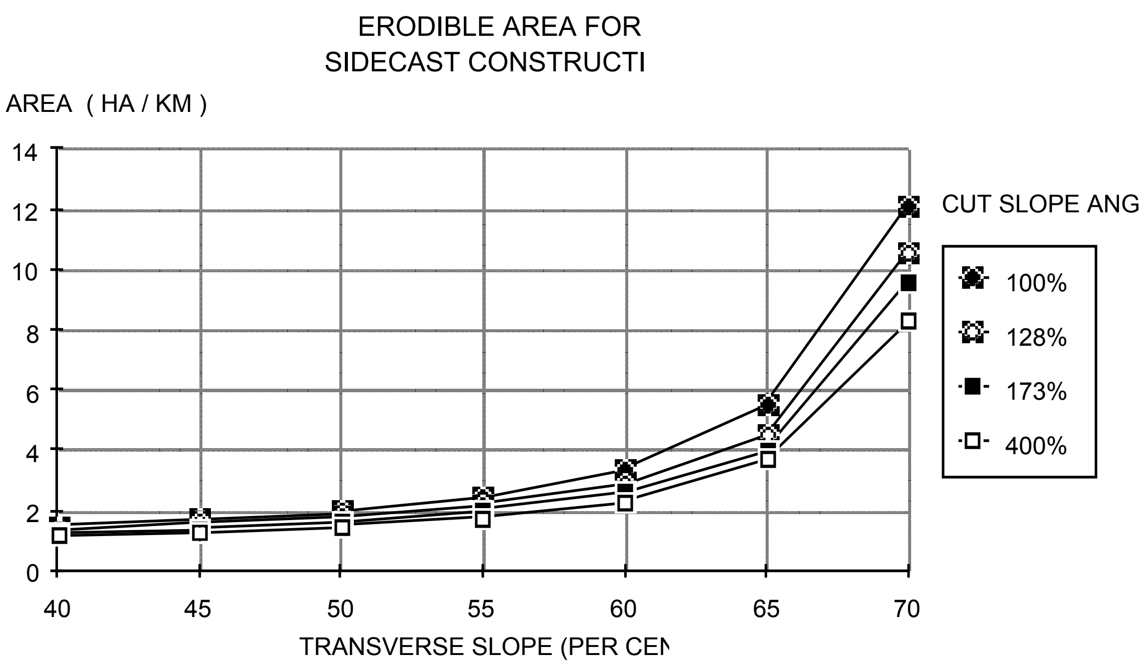


Figure 46. Erodible area per kilometer of road for side cast construction as a function of side slope angle and cut slope angle. The values shown are calculated for a 6.6 m wide subgrade. The fill angle equals 37°.

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.

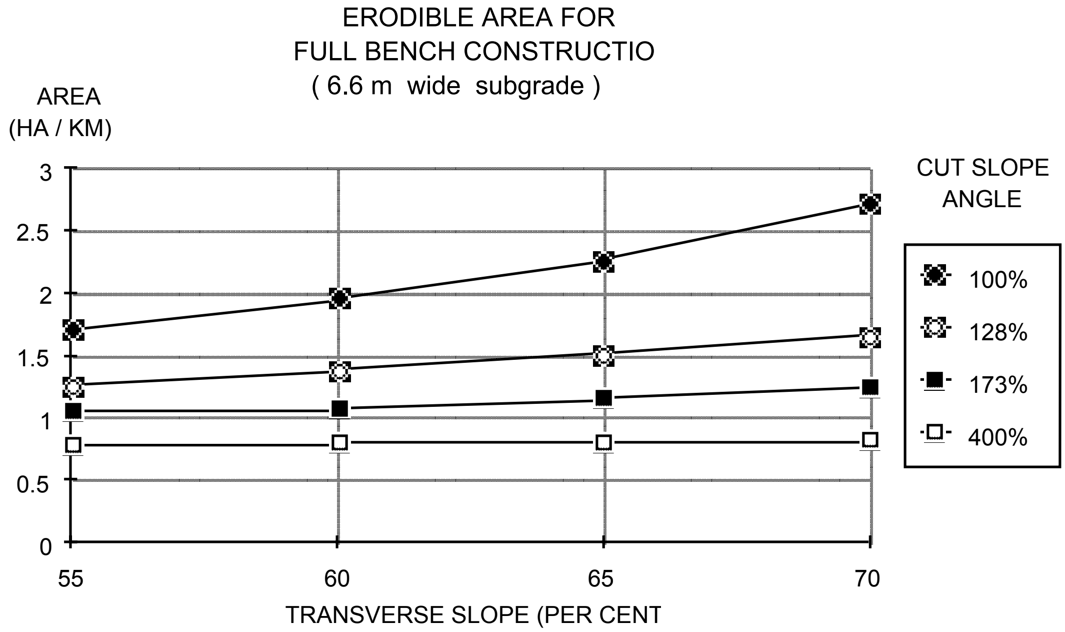


Figure 47. Erodible area per kilometer of road for full bench/end haul construction as a function of side slope angle and cut slope angle. The values shown are calculated for a 6.6 m wide subgrade..

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.

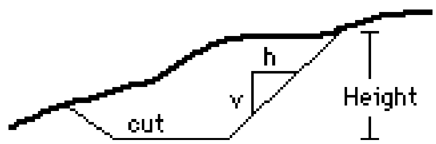
4. Soft consistency; soil can easily be molded by fingers; penetration test blow count approximately 5 blows per decimeter.

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	

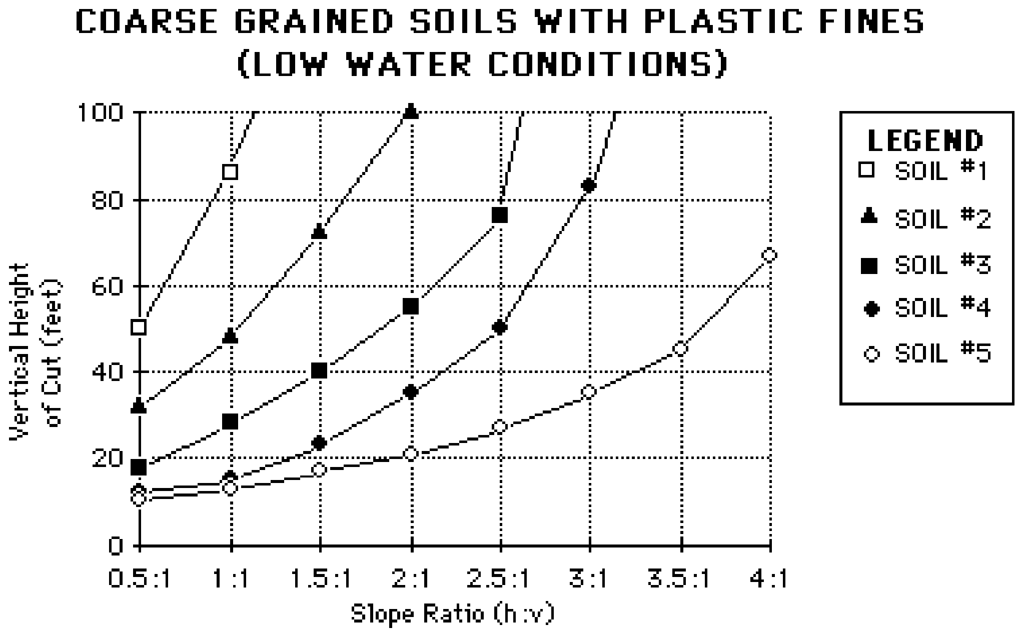


Figure 48. Maximum cut slope ratio for coarse grained soils with plastic fines (low water conditions). Each curve indicates the maximum height or the steepest slope that can be used for the given soil type. (After USFS,1973)

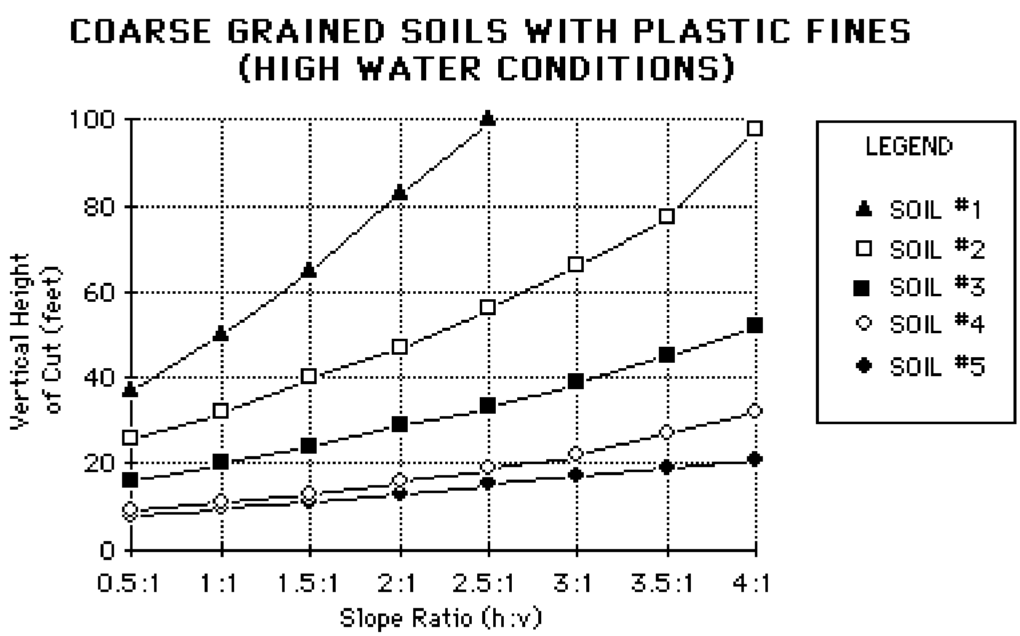


Figure 49. Maximum cut slope angle for coarse grained soils with plastic fines (high water conditions). Each curve indicates the maximum cut height or the steepest slopes that can be used for the given soil type. (After USFS,1973)

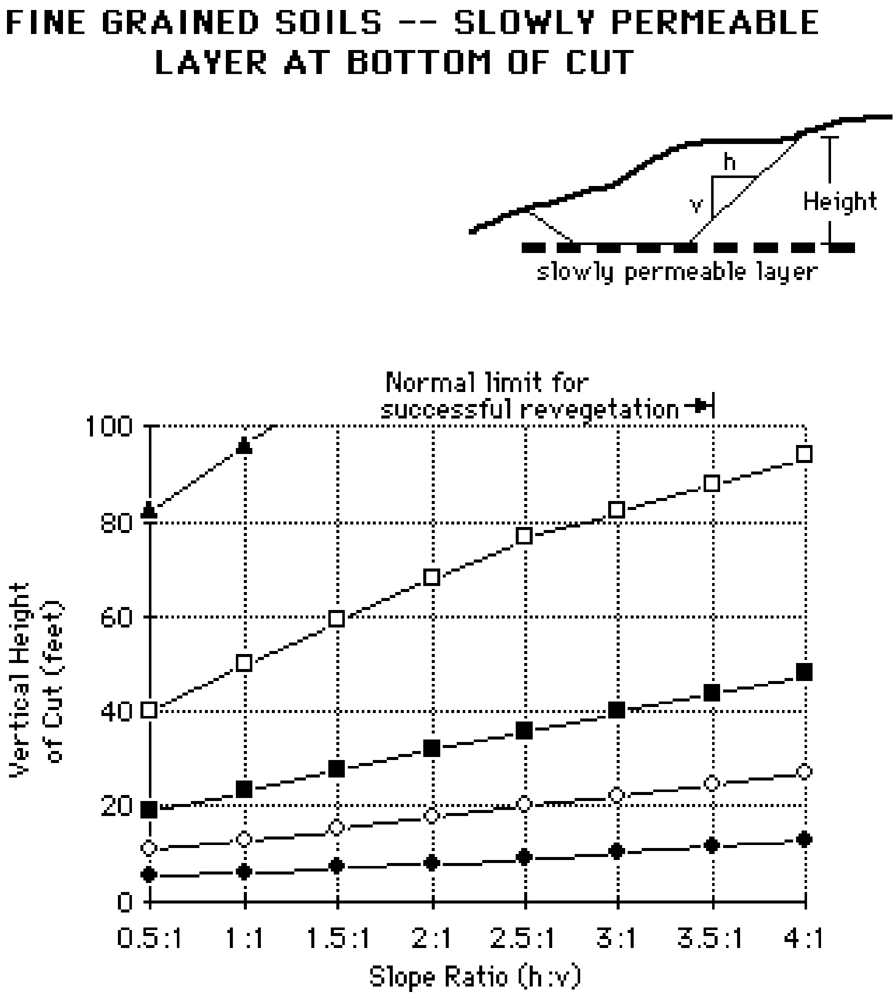
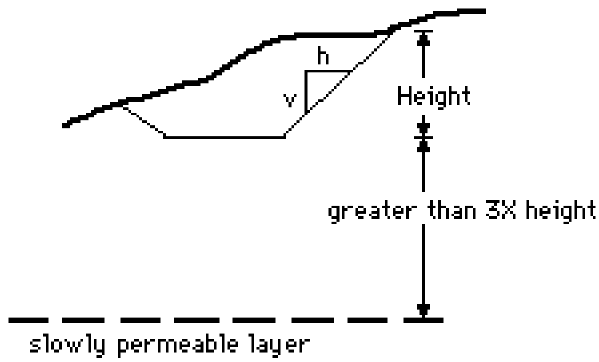


Figure 50. Maximum cut slope angle for fine grained soils with slowly permeable layer at bottom of cut. Each curve indicates the maximum vertical cut height or the steepest slope that can be used for the given soil type. (After USFS 1973)

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 4	Figure 48, Soil 3 Figure 49, Soil 4	Figure 49, Soil 3 no control	90
ML, CL[2] Figure 48, Soil 5	Figure 48, Soil 4 Figure 49, Soil 4	Figure 49, Soil 4 no control	90
MH, OH[2] Figure 50, Soil 4	Figure 50, Soil 3 Figure 50, Soil 5	Figure 50, Soil 4 no control	90

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

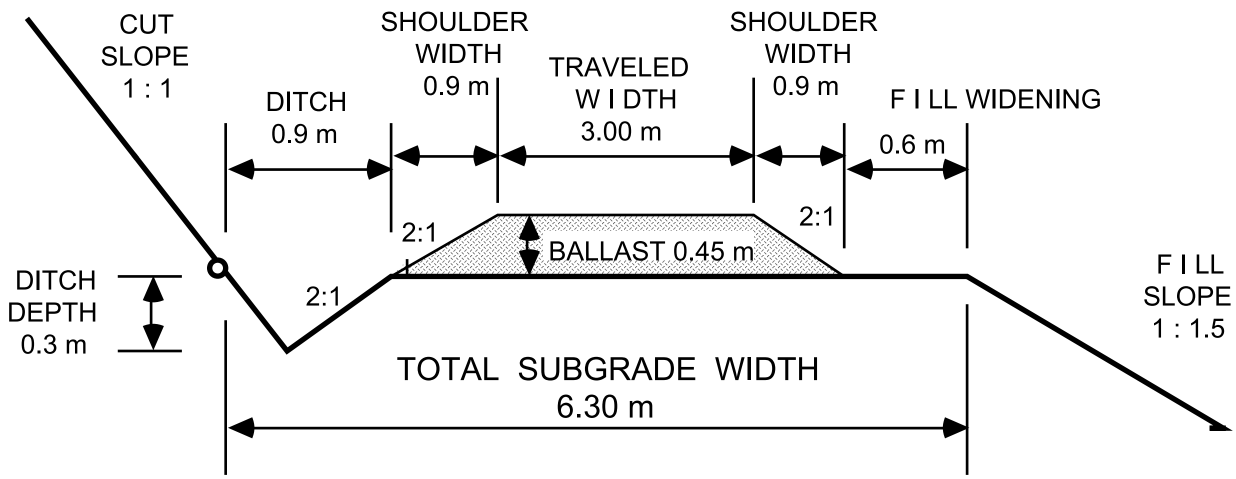


Figure 52. Interaction of subgrade dimension, roadwidth, ballast depth, ditch width and fill widening

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

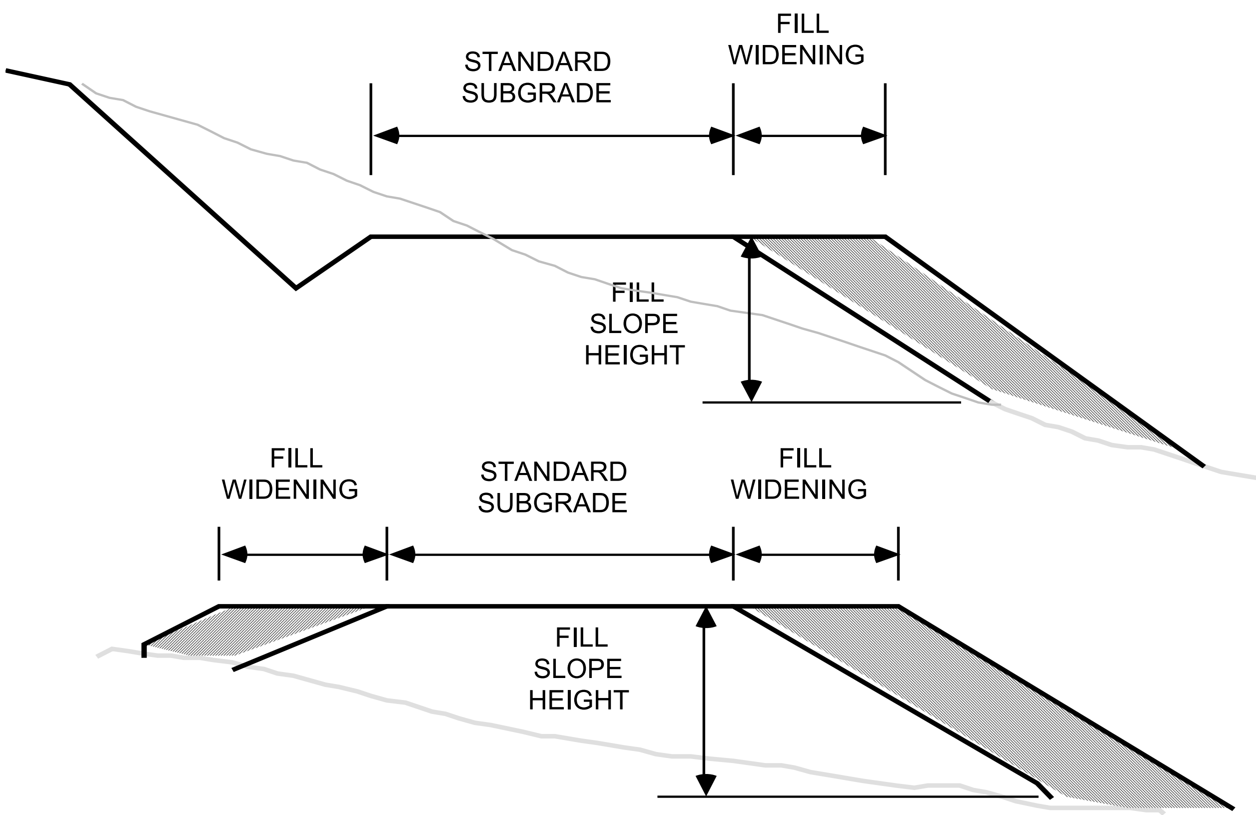


Figure 53. Fill widening added to standard subgrade width where fill height at centerline or shoulder exceeds a critical height. Especially important if sidecast construction instead of layer construction is used.

FILL SLOPE LENGTH REDUCTION

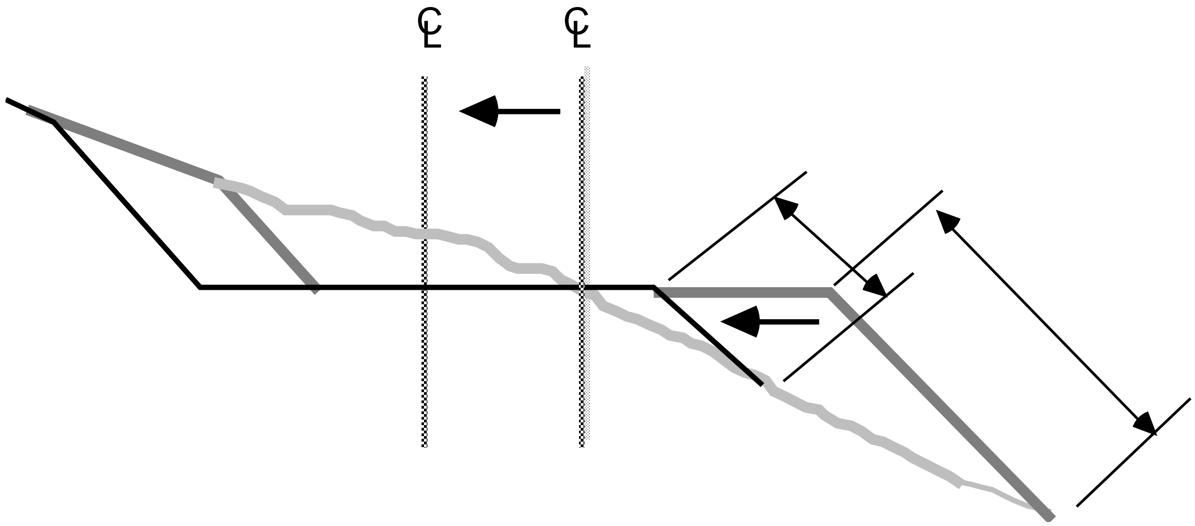


Figure 54. Template and general road alignment projected into the hill favoring light to moderate cuts at centerline in order to minimize fill slope length. Fill slopes are more succeptible to erosion and sloughing than cut slopes.

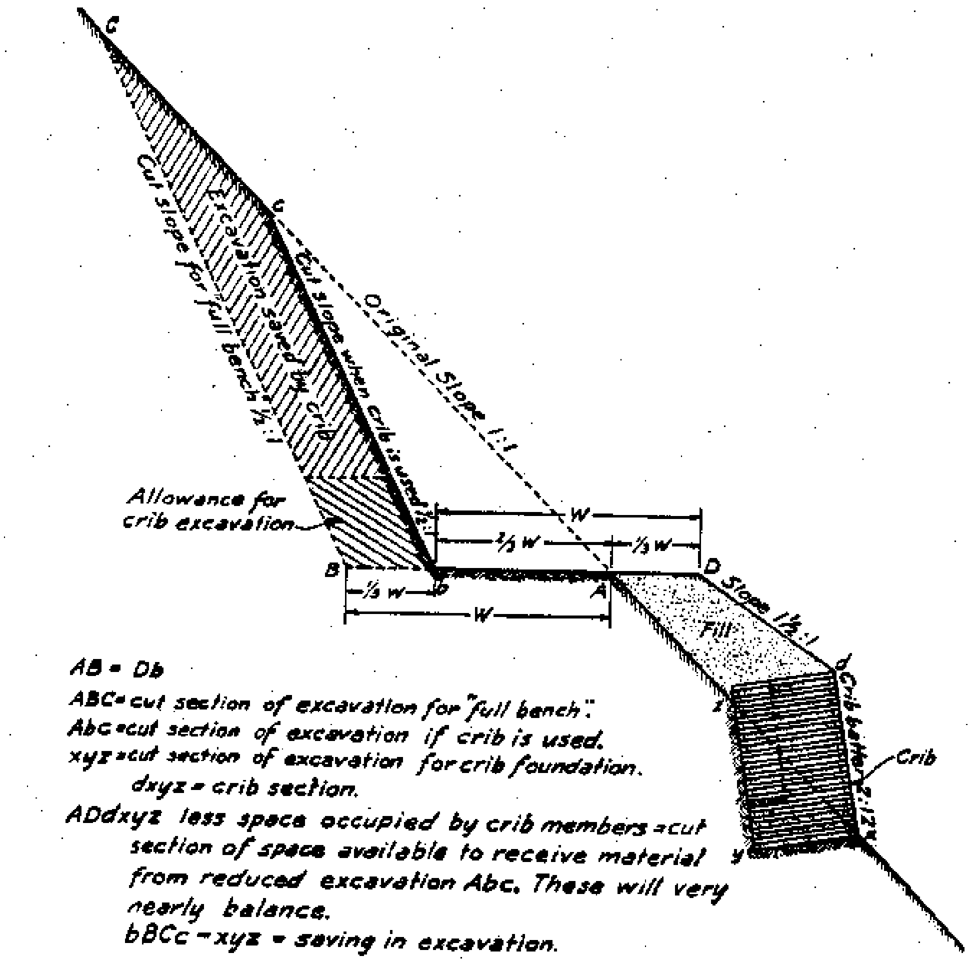


Figure 55. Illustration of the very considerable reduction in excavation made possible on a steep slope by the use of cribbing. Crib proportions shown are suitable for log construction; if crib was built of concrete or steel, shorter spreaders could be used in upper 3 m as indicated by the dashed line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