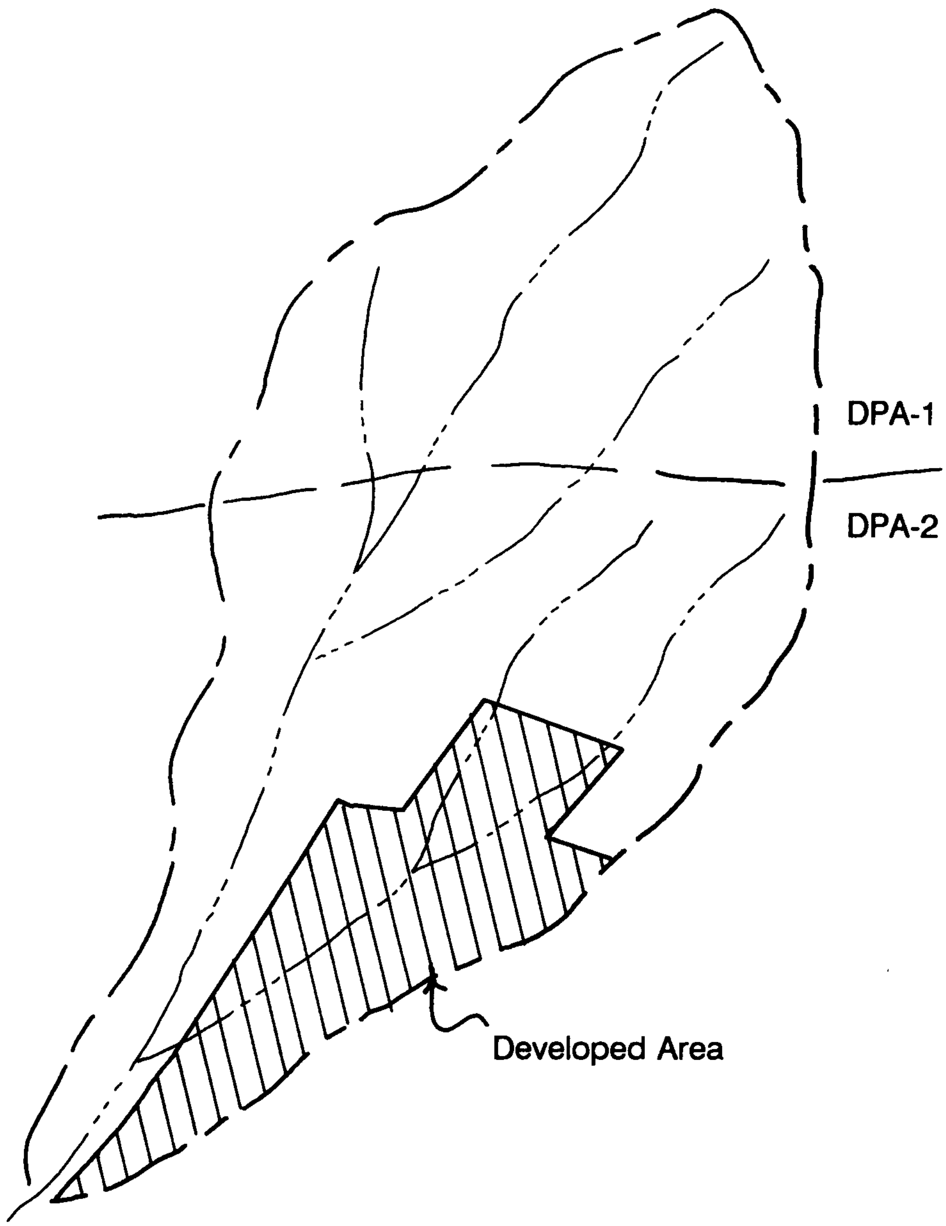
# APPENDIX D

## <u>Sedimentation Examples</u>

*(Refer to the Sedimentation Manual Text)*

# Debris Production & Bulked Q Example

**WORK MAP**




| Initials<br>HK | Los Angeles County<br>Department of Public Works |
|---|---|
| Date<br>6/17/91 | Project Name<br><br>DEBRIS PRODUCTION & BULKED Q<br>EXAMPLE PROBLEM |
| Scale<br>NONE | |

LAP MRHNDO1.FRM 12/01/90
SEDEXPO1.FRM

## EXAMPLE 1

DEBRIS PRODUCTION AND BULKED Q EXAMPLE

Given:

Burned **Q** = 348 cfs
Total Area = 224 acres = 0.35 sq. miles
Developed Area = 45 acres = 0.07 sq. miles

Problem:

Find the total debris production and the bulked Q.

Answer:

1. Calculate the areas for each DPA zone.

| DPA Zone | Drainage Area | | Developed Area |
|---|---|---|---|
| | Acres | Square Miles | Square Miles |
| 1 | 77 | 0.12 | 0 |
| 2 | 147 | 0.23 | 0.07 |
| Total | 224 | 0.35 | 0.07 |

2. Determine the debris production rate (DPR) for each DPA zone from Appendix B-3 corresponding to the total drainage area.

For DPA zone 1 (total area = 0.35)
DPR = 155,000 cubic yards per sq. mile
DPA zone 1 (total area in DPA 1 - developed area = 0.12)
DPR = 222,000 cubic yards per sq. mile
DPA zone 2 (total area = 0.35)
DPR = 98,000 cubic yards per sq. mile
DPA zone 2 (total area in DPA 2 - developed area = 0.23 - 0.07 = 0.16)
DPR = 123,000 cubic yards per sq. mile

3. Calculate the total debris production as follows:

$$DP = 155{,}000\ (0.12 - 0)\left(\frac{0.12 - 0}{0.12 + 0.23}\right) + 222{,}000\ (0.12 - 0)\left(\frac{0.23 + 0}{0.12 + 0.23}\right)$$

$$+\ 98{,}000\ (0.23 - 0.07)\left(\frac{0.23 - 0.07}{0.12 + 0.23}\right) + 123{,}000\ (0.23 - 0.07)\left(\frac{0.12 + 0.07}{0.12 + 0.23}\right)$$

$$= 41{,}734.8\ yd^3$$

4. Determine the peak bulking factor for each DPA zone from Appendix B-6 and find the $Q_{bulked}$ $(Q_B)$ as follows:

In DPA zone 1:

Peak bulking factor for area $A_1 + A_2 = BF_{1(A1+A2)} = 2$

Peak bulking factor for area $A_1 - Ad_1 = BF_{1(A1-Ad1)} = 2$

In DPA zone 2:

Peak bulking factor for area $A_1 + A_2 = BF_{2(A1+A2)} = 1.81$

Peak bulking factor for area $A_2 - Ad_2 = BF_{2(A1-Ad2)} = 1.81$

$$Q_B = 2.00\left(\frac{348\,(0.12 - 0)}{0.12 + 0.23}\right)\left(\frac{0.12 - 0}{0.12 + 0.23}\right) +$$

$$2.00\left(\frac{348\,(0.12 - 0)}{0.12 + 0.23}\right)\left(\frac{0.23 + 0}{0.12 + 0.23}\right) + \left(\frac{348\,(0)}{0.12 + 0.23}\right) +$$

$$1.81\left(\frac{348\,(0.23 - 0.07)}{0.12 + 0.23}\right)\left(\frac{0.23 - 0.07}{0.12 + 0.23}\right) +$$

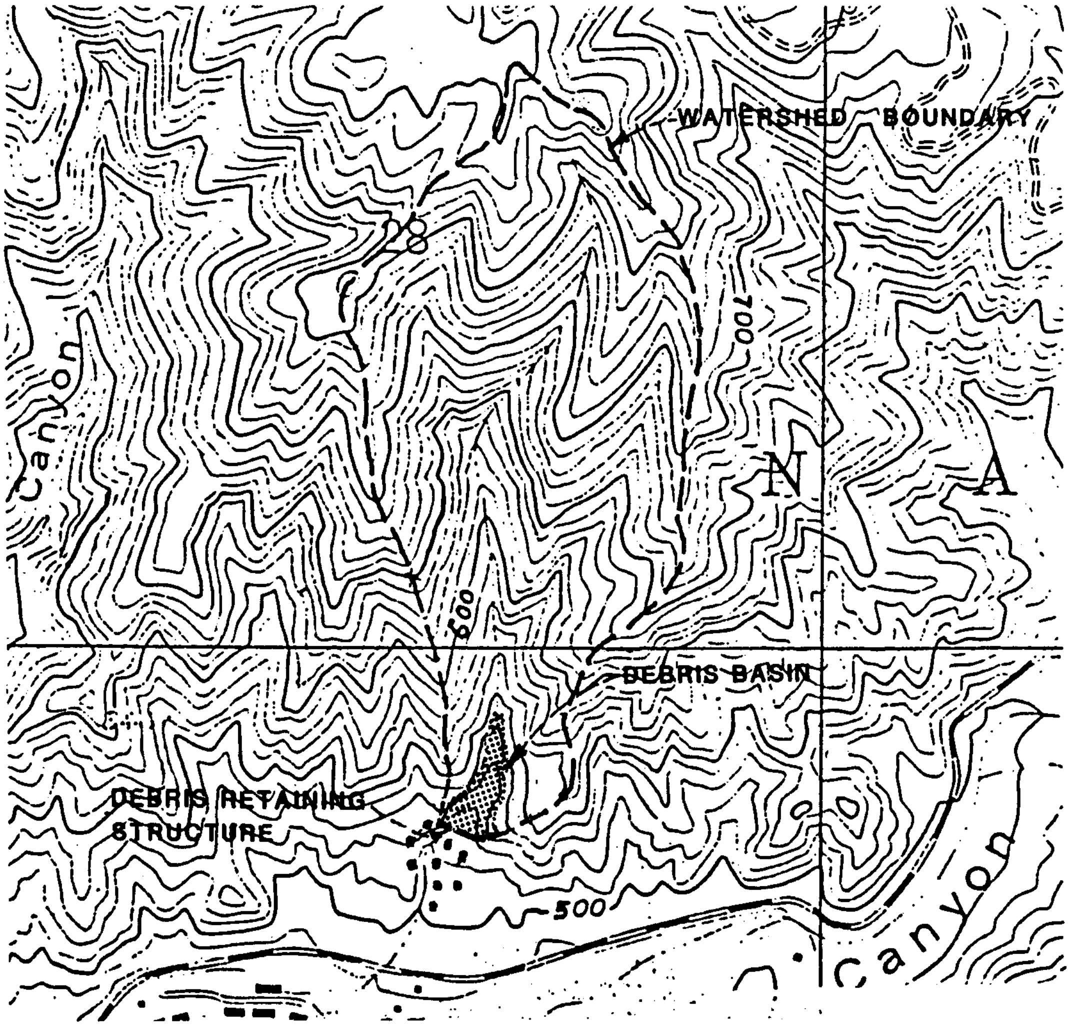
$$1.81\left(\frac{348\,(0.23 - 0.07)}{0.12 + 0.23}\right)\left(\frac{0.12 + 0.07}{0.12 + 0.23}\right) + \left(\frac{348\,(0.07)}{0.12 + 0.23}\right)$$

$$= 596\ cfs$$

# Debris Basin Design Example

EXAMPLE 2

DEBRIS BASIN DESIGN EXAMPLE PROBLEM




Design a debris basin for the watershed shown above.

The given watershed is in DPA zone 6 and has a tributary area of 0.2 square miles.

The debris production rate for the given watershed is 37,500 cubic yards (see curve on Appendix page B-1.

The Design Debris Event (DDE) is 37,500 $x$ 0.2 = 7,500 cubic yards

The DDE is the basis for capacity in designing the Debris Basin. At this point a large scale (20 or **40** feet per inch) topographic map is required. The actual calculation is a trial and error procedure. The designer must select a height for the structure, $H_S$. Setting this height determines the other dimensions of the basin such as the height of cone, $H_C$, the natural slope $S_N$, and the debris slope, $S_D$.

**EXAMPLE 2-A**

DEBRIS SLOPE CALCULATION

Given:

| | | | |
|---|---|---|---|
| Spillway crest elevation, | El, | = | 530.7 ft |
| Original ground elevation below the spillway, | $El_g$ | = | **517.3** ft |

$$H_s = El, - El_g = 530.7 - 517.3 = 13.4\,ft$$

The cone height:

$$\begin{aligned} H_c &= 2H_s \\ &= 2(13.4) = 26.8\,ft \end{aligned}$$

The elevation of this point is: 517.3 + 26.8 = ***544.1 ft***

To determine the length, $L$ (needed to calculate $S_N$), scale along the stream's natural flow line from the point below the spillway to elevation **544.1** (a distance of **410** feet). Therefore:

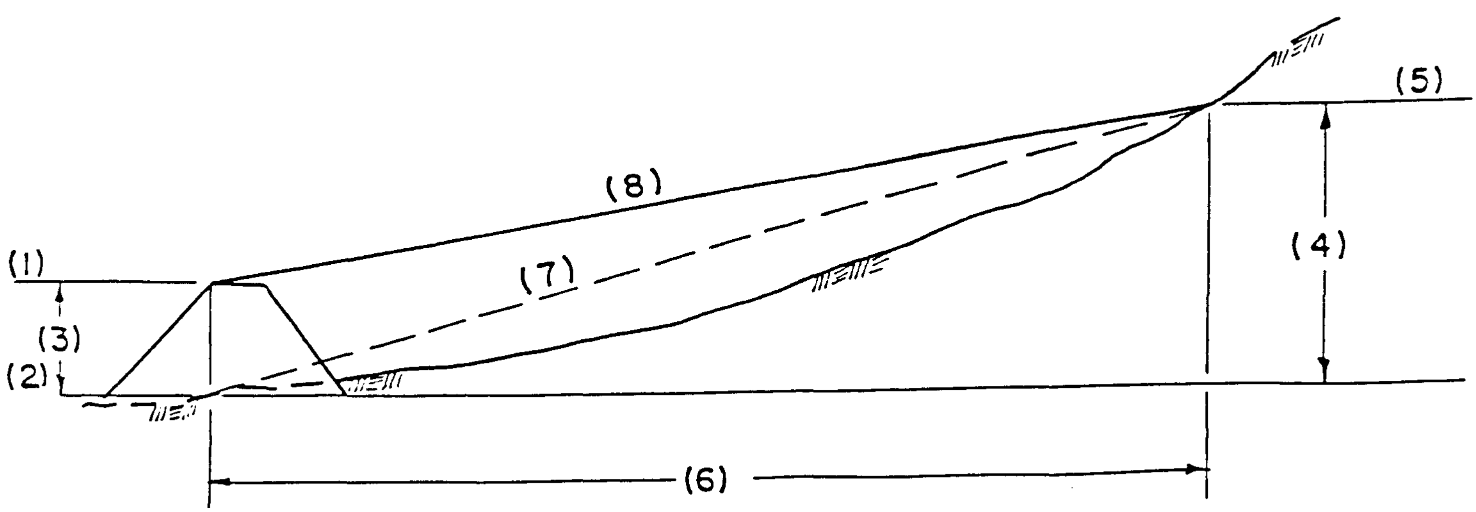
$$S_N = \frac{H_C}{L} = \frac{26.8}{410} = 0.0654$$

The cone slope

$$\begin{aligned} S_D &= 0.5 \times S_N \\ &= 0.5 \times 0.0654 = 0.033 \end{aligned}$$

## DEBRIS SLOPE CALCULATION

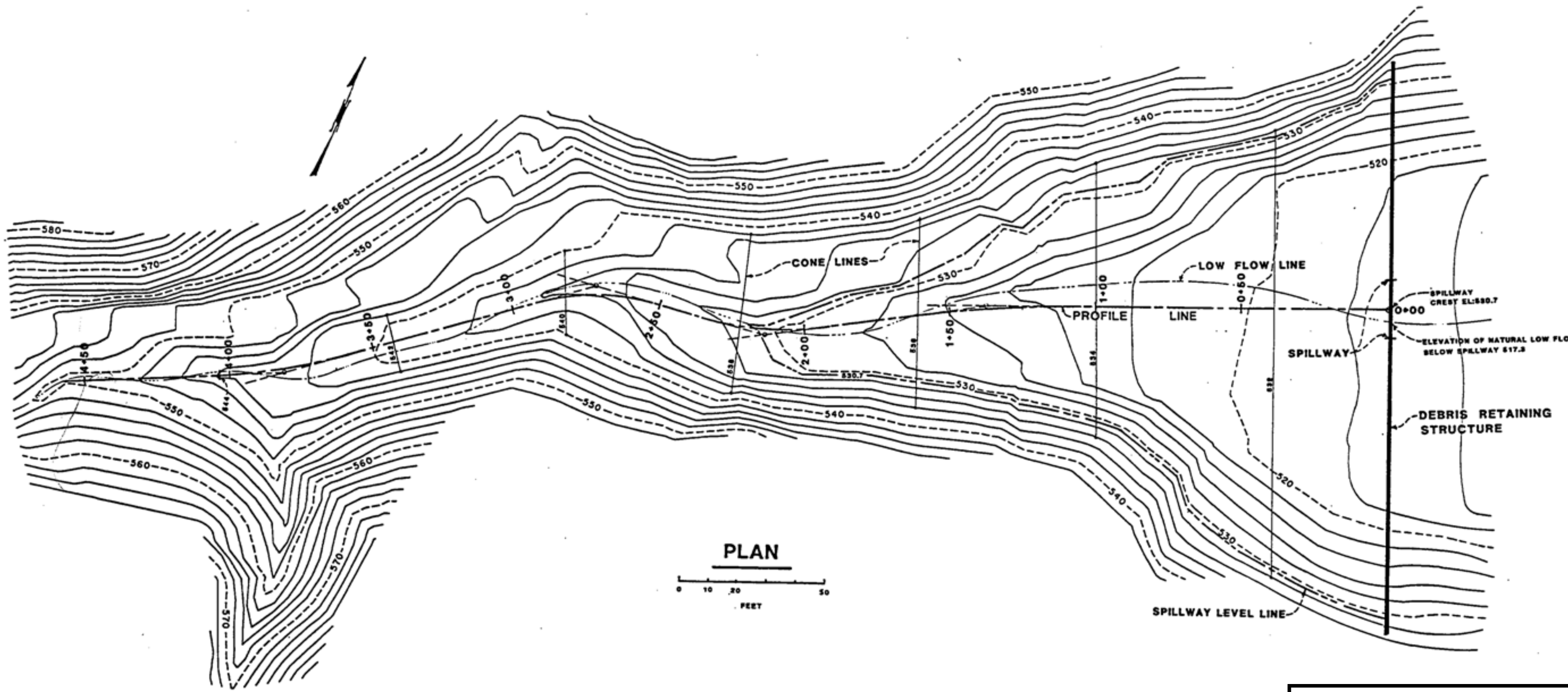
| | | |
|---|---|---|
| (1.) | Elevation of spillway crest (El,) | 530.7 |
| (2.) | Prior elevation of natural ground below spillway ($El_g$) | 517.3 |
| (3.) | Height of spillway above natural ground ($H_S$): (I) - (2) = | 13.4 |
| (4.) | $\Delta$EL ($H_C$) = 2 x (3) = | 26.8 |
| (5.) | Upstream cone elevation = (2) + (4) = | 544.1 |
| (6.) | Horizontal distance from spillway crest to upstream cone elevation (measured along low flow path) (L) = | 410 |
| (7.) | Natural slope ($S_N$) = {(4) ÷ (6)) x 100 = | 6.54% |
| (8.) | Debris cone slope ($S_D$) = 0.5 x (7) = | 3.27% (Say 3.3%) |



To determine the capacity of the basin thus defined, measure the area behind the dam delineated by each full contour. Maximum contour intervals should be 2 ft for small basins and 5 ft for large basins of 50,000 cubic yards or more. The areas of two contours are averaged, then multiplied by the contour interval to calculate the volume of that slice. Starting at the bottom of the basin, these volumes of the slices are summed until spillway crest is reached. The total volume constitutes the Level Capacity.

The cone lines are located by measuring along the profile which has a slope of 3.3% in this example. To calculate the volume of the debris cone, begin by selecting the cone line at the spillway crest (530.7). The second cone line is selected at elevation 532.0 feet. The first contour interval is thus 1.3 feet. The third and following cone lines will be selected at 2 foot contour intervals and so on until elevation 544.0 feet. Since the highest cone line daylights at elevation 544.1, the last contour interval is only 0.1 foot. After selecting the cone lines, the contour area at each cone line will be planimetered. The areas of two contours are averaged, then multiplied by the contour interval (typically 2 feet in this example, except for the very first and last contour intervals) to calculate the volume of that slice. Starting at the spillway crest, all volumes of slices are summed until the top of cone. The total volume constitutes the cone capacity. The cone capacity is added to the level capacity for the total capacity of the basin (see attached plan on page D-8).

If the volume reached is not sufficient for the DDE, then a higher dam must be used, and the calculations repeated as before.




Los Angeles County
Department Of Public Works

Debris Storage Capacity
Calculations

## STORAGE CAPACITY CALCULATIONS

| ELEVATION (FT) | CONTOUR INTERVAL | AREA (SQ.FT) | MEAN AREA (SQ.FT) | VOLUME (CU.YD) | TOTAL VOLUME (CU.YD) |
|---|---|---|---|---|---|
| 517.3 | | 0 | | | |
| | 0.7 | | 556 | 14 | 14 |
| 518.0 | | 1112 | | | |
| | 2.0 | | 3083 | 228 | 242 |
| 520.0 | | 5054 | | | |
| | 2.0 | | 6685 | 495 | 737 |
| 522.0 | | 8316 | | | |
| | 2.0 | | 9393 | 696 | 1433 |
| 524.0 | | 10470 | | | |
| | 2.0 | | 11539 | 855 | 2288 |
| 526.0 | | 12608 | | | |
| | 2.0 | | 13713 | 1016 | 3304 |
| 528.0 | | 14818 | | | |
| | 2.0 | | 16052 | 1189 | 4493 |
| 530.0 | | 17286 | | | |
| | 0.7 | | 17740 | 460 | 4953 |
| 530.7 spwy | | 18195 | | | |
| | 1.3 | | 15655 | 754 | 5707 |
| 532.0 | | 13116 | | | |
| | 2.0 | | 10489 | 777 | 6484 |
| 534.0 | | 7863 | | | |
| | 2.0 | | 6285 | 466 | 6950 |
| 536.0 | | 4706 | | | |
| | 2.0 | | 3696 | 274 | 7224 |
| 538.0 | | 2687 | | | |
| | 2.0 | | 1953 | 145 | 7369 |
| 540.0 | | 1220 | | | |
| | 2.0 | | 889 | 66 | 7435 |
| 542.0 | | 558 | | | |
| | 2.0 | | 280 | 21 | 7456 |
| 544.0 | | 2 | | | |
| | 0.1 | | 1 | --- | --- |
| 544.1 | | 0 | | | |
| | | | | Storage Capacity | ≈7500 |

EXAMPLE 2-B

DEBRIS SLOPE CALCULATION FOR A TWO-BRANCH STREAM

| | | |
|---|---|---|
| (1.) | Elevation of spillway crest (El,) | 993.0 |
| (2.) | Prior elevation of natural ground below spillway ($El_g$) | 981.6 |
| (3.) | Height of spillway above natural ground ($H_S$): (1) - (2) = | 11.4 |
| (4.) | $\Delta$EL ($H_C$) = 2 x (3) = | 22.8 |
| (5.) | Upstream cone elevation = (2) + (4) = | 1004.4 |

(6.) Horizontal distance from spillway crest to upstream cone elevation (measured along low flow path) (L)

WEST FORK L = 333 ft
SOUTH FORK L = 211 ft

(7.) Natural slope ($S_N$) = {(4) ÷ (6)) x 100

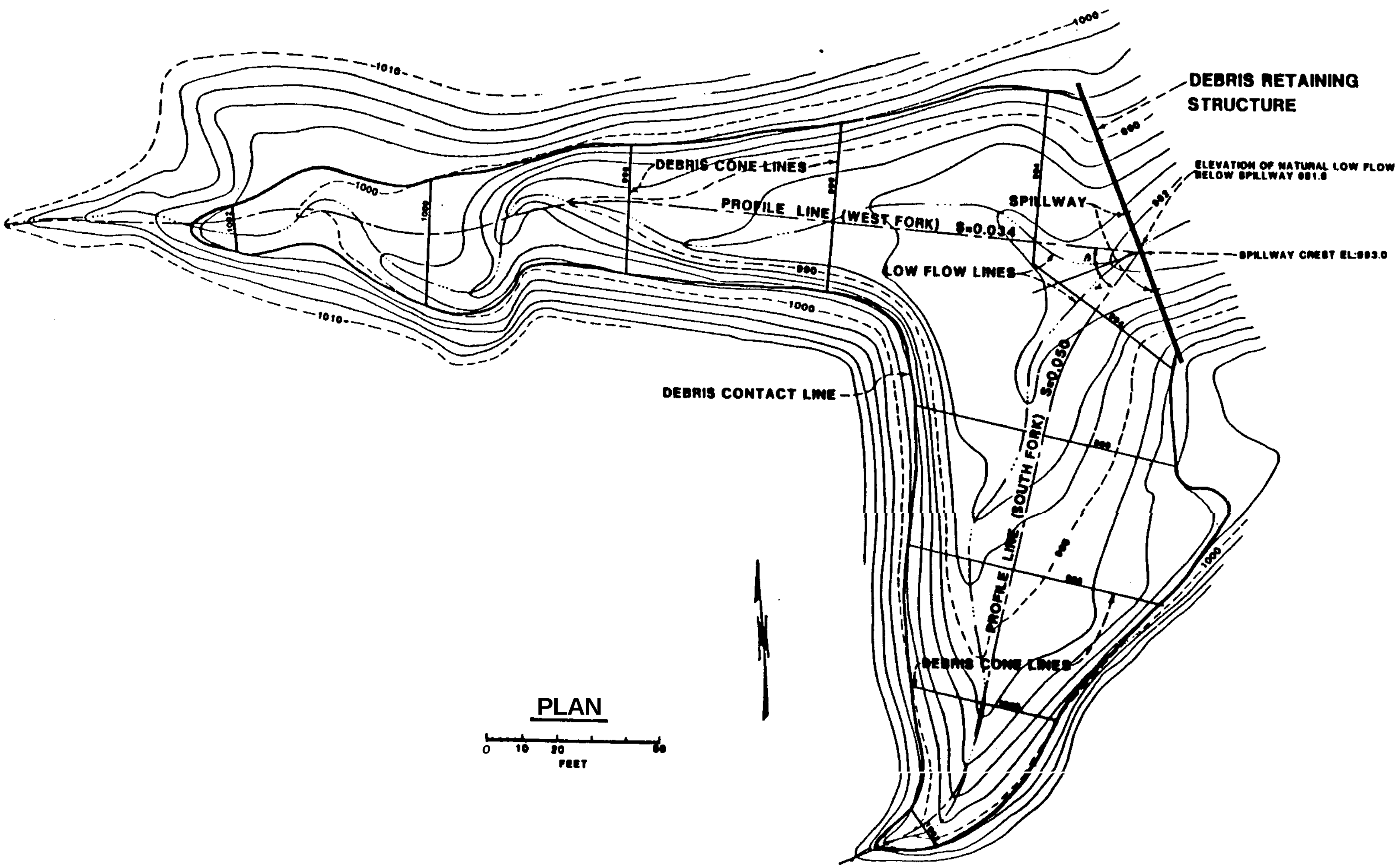
WEST FORK $S_N$ = 6.85%
SOUTH FORK $S_N$ = 10.8%

(8.) Debris cone slope ($S_D$) = 0.5 x (7)

WEST FORK $S_D$ = 3.4%
SOUTH FORK $S_D$ = 5.4%

Max. Allowable $S_D$ = 5.0%; therefore, SOUTH FORK $S_D$ = 5.0%

Proceed with the rest of the calculations as in Example 2-A.




Los Angeles County
Department Of Public Works

Debris Slope Calculation
And Representation On Plan

# Levee & Bridge Design Example

EXAMPLE 3

## LEVEE AND BRIDGE DESIGN EXAMPLE PROBLEM

The following information is given for a proposed channel:

| | | |
|---|---|---|
| Capital peak discharge, $Q_{cap}$ | = | 23,500 cfs |
| Flow Top width, W | = | 450 ft |
| Radius of curvature, $r$ | = | 2400 ft |
| Length of improvement, L | = | 3500 ft |
| Distance from downstream end of project to the closest grade control | = | 1000 ft |
| Existing slope, $S$ | = | 0.007 |
| Channel side slope | = | 1½ (H):1(V) |
| Velocity under existing conditions for $Q_{cap}$ | = | 11.8 fps |
| Velocity under existing conditions for 25% of $Q_{cap}$ | = | 7 fps |

I. LEVEE DESIGN

1. Levee toe-down (See section 5.A-2 of Sedimentation Manual)

Flow velocity (V) and depth (Y) in the proposed channel were determined by performing a hydraulic analysis using a low Manning's $n$ of 0.025 for the Capital Flood $Q$ and 25 percent of Capital Flood $Q$.

$V_{cap}$ = 12.4 fps
$Y_{cap}$ = 4.2 ft
$V_{25\% \text{ of } cap}$ = 7.35 fps

a. Long Term Degradation ($Z_{deg}$)

The first step is to determine if invert stabilization is required.

Percent Increase in Velocity for Capital Flood Q
= 100 ((12.4 - 11.8) / 11.8) = 5.1%
Percent Increase in Velocity for 25% of Capital Flood Q
= 100 ((7.35 - 7) / 7) = 5.0%

From the graph in Appendix C-2 for natural slope of 0.007, 5.1% velocity increase, and assuming no reduction in sediment supply, the point of intersection falls below the curve that represents no reduction in sediment supply. Therefore, invert stabilization is not required within the project reach.

The second step is to determine the equilibrium slope for the channel with no reduction in sediment supply. The equilibrium slope for natural slope of *0.007* can be computed by interpolation between $S = 0.005$ and $S = 0.01$.

For $S = 0.005$ From Appendix C-1A
$S - S_{eq} = 0.00035$
$S_{eq} = 0.005 - 0.00035 = 0.00465$
For $S = 0.01$ From Appendix C-1B
$S - S_{eq} = 0.0007$
$S_{eq} = 0.01 - 0.0007 = 0.0093$
By interpolation with $S = 0.007$, $S_{eq} = 0.0065$

Therefore, with natural slope equal to *0.007,* the maximum long term degradation in the channelized reach can be computed as follows:

$$Z_{deg} = (3500 + 1000)(0.007 - 0.0065) = 2.25 \text{ ft}$$

b. General Scour ($Z_{gs}$)

From Appendix C-3 for velocity of *12.4* fps:

$$Z_{gs} = 2.9 \text{ ft}$$

c. Local Scour ($Z_{ls}$)

A typical value of *2* ft was used. (See Section 5.2(c) .)

d. Bend Scour ($Z_{bs}$)

$(V)(Y) = (12.4)(4.2) = 52.08$
$W/r = 450/2400 = 0.19$
Assume uniform flow conditions. Therefore $S_e = S_{eq} = 0.0065$ (Refer to Section 5.2(d) ):

$Z_{bs}$ for $S_e = 0.0065$ can be computed by interpolation between
$S_e = 0.001$ and $S_e = 0.01$
For $S_e = 0.001$ from Appendix page C-7C
$Z_{bs} = 1.2$ ft
For $S_e = 0.01$ from Appendix page C-7B
$Z_{bs} = 0.8$ ft
By interpolation with $S_e = 0.0065$, $Z_{bs} = 0.96$ ft

To find the extent of bend scour downstream of the bend, use Appendix page C-8 with depth of flow at the bend = 4.2 ft

$$X = 170 \text{ ft}$$

e. Low Flow Incisement ($Z_i$)

Since there were no field measurements, a typical value of 2 ft was used. (See Section 5.2(e). )

f. Bedform Height *(h)*

From Appendix page C-9 bedform height h = 4.3 ft for V = 12.4 fps

Therefore the levee toe-down was computed as follows:

$$\begin{aligned} Z_{tot} &= Z_{deg} + Z_{gs} + Z_{ls} + Z_{bs} + Z_i + \frac{1}{2}h \\ &= 2.25 + 2.9 + 2 + 0.96 + 2 + 0.5(4.3) = 12.26 \approx 12.3 \ ft \end{aligned}$$

According to the cut-off depth table in the Hydraulic Design Manual, levee toe-down is 15 ft (for a curved reach and a velocity of 12 fps), therefore, use 15 ft cut-off depth.

2. Freeboard (See Section 5.2 of Sedimentation Manual)

The channel was assumed to have little vegetation, therefore a higher Manning's $n$ of 0.035 was used to evaluate maximum freeboard. By performing hydraulic analysis for the specific reach, flow velocity and depth were determined to be 11 fps and 4.8 ft, respectively.

$$Freeboard = Y_{agg} + Y_{ga} + Y_{se} + \frac{1}{2}h$$

Since this channel is degrading (see 1(a) of this example):

$$Y_{agg} = 0$$
$$Y_{ga} = 0$$

Compute superelevation, using the criteria in the Hydraulic Design Manual. The flow is subcritical and the velocity V = 11 fps is less than 35 fps, therefore:

$$Y_{se} = \frac{1.15(W)V^2}{2gr}$$
$$= \frac{1.15\,(450)\,11^2}{(2)(32.2)(2400)} = 0.41\ ft$$

where: $g$ = Acceleration of gravity = 32.2 ft/sec$^2$

From Appendix page C-9 bedform height $h$ = 3.3 ft for V = 11 fps

Therefore the freeboard was computed as follows:

$$Freeboard = Y_{agg} + Y_{ga} + Y_{se} + \frac{1}{2}h$$
$$= 0 + 0 + 0.41 + 0.5(3.3) = 2.06 \approx 2.1\ ft$$

According to section C-4.2 in the Hydraulic Design Manual, for average flow velocity of 35 fps or less, the freeboard is 2.5 ft. Therefore use 2.5 ft freeboard.

## II. BRIDGE DESIGN

Find the total pier scour and abutment scour for a proposed bridge in a straight reach of the proposed channel given the following additional information:

| | |
|---|---|
| Pier width, b | = 3 ft |
| Pier angle of attack | = 15 degrees |
| Pier length, L | = 15 ft |
| Length abutment protrudes into the flow, *a* | = 7 ft |
| Pier type | = Round Nose |

The channel was assumed to have little vegetation, therefore debris blockage was assumed to be two feet.

Flow velocity, V (13.0 feet), and depth, Y (4.0 feet), at the proposed bridge section were determined by performing a hydraulic analysis using a low Manning's n of 0.025 and pier width of 3 + 2 = 5 feet.

1. Pier scour

   From Appendix page C-4, for V = 13, pier local scour $Z_{ls}$ = 8.7 ft

The reduction factor $(K_1)$ for round nose piers from the Sedimentation Manual Table 5.1 is 0.9

$L/b = 5$
Therefore the multiplying factor $(K_2)$ for a 15° angle of attack from Appendix page C-5 = 1.7

Adjustment for debris blockage:

$$K_3 = \left(\frac{b + d}{b}\right)^{0.65}$$
$$= \left(\frac{3 + 2}{3}\right)^{0.65} = 1.40$$

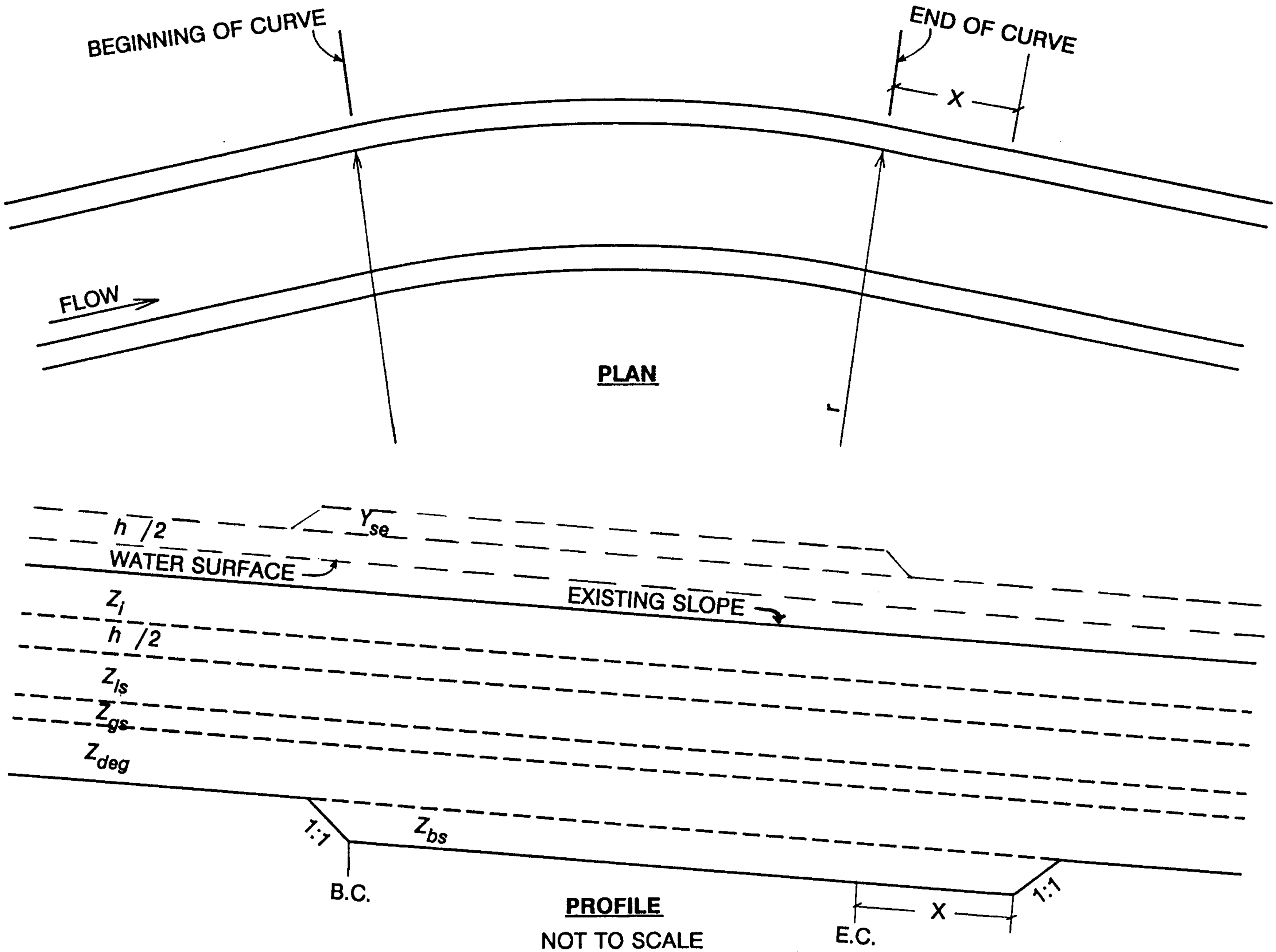
$$\textbf{\textit{Net pier local scour}} = Z_{ls} \times K_1 \times K_2 \times K_3$$
$$= 8.7 \times 0.9 \times 1.7 \times 1.4 = 18.6\ ft$$

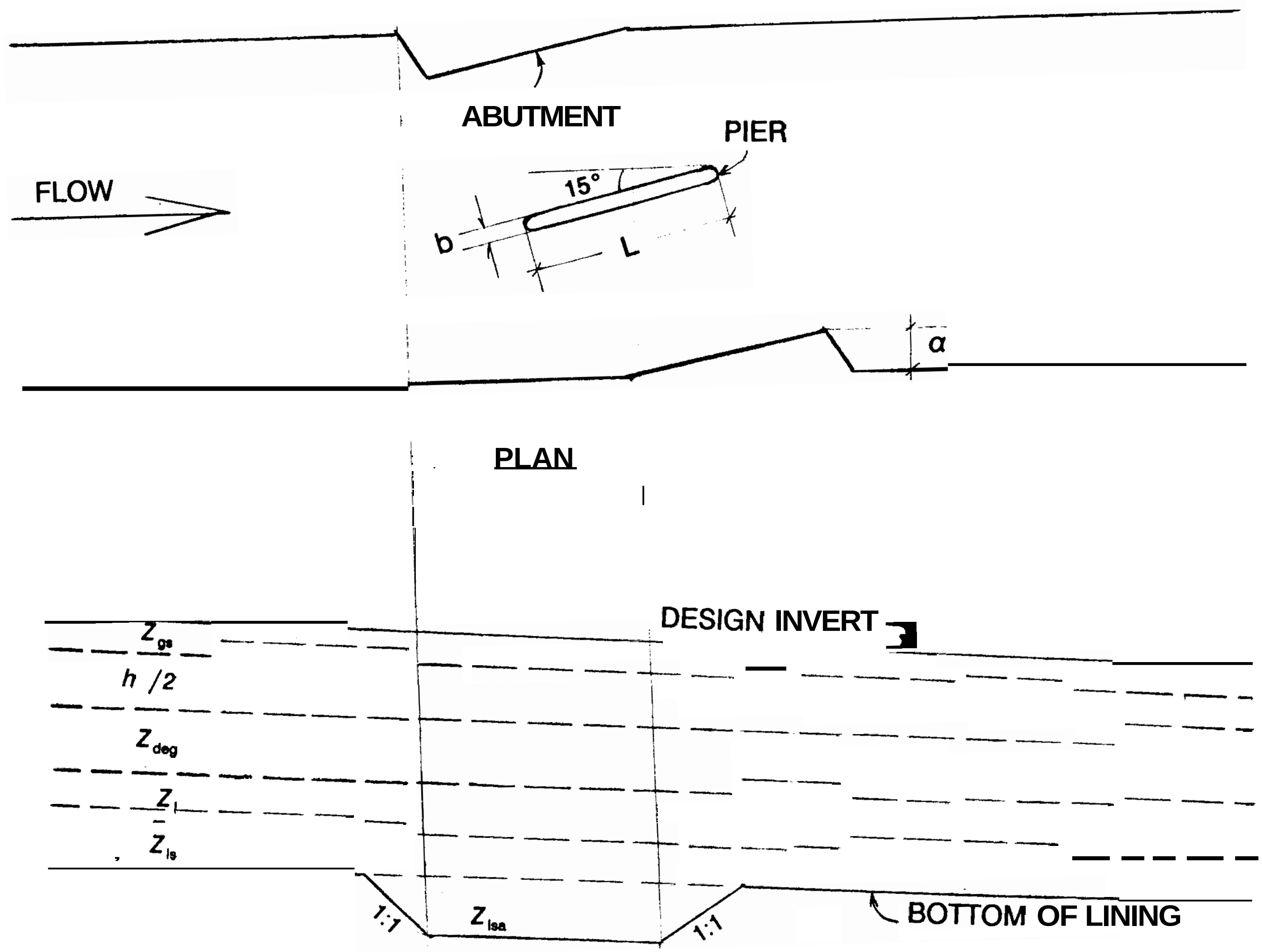
$$Z_{tot} = Z_{deg} + Z_{gs} + Z_{ls} + Z_{bs} + Z_i + \frac{1}{2}h$$
$$= 2.25 + 2.9 + 18.6 + 0 + 2 + 0.5(4.3) = 27.9$$
$$\text{use } Z_{tot} = 28\ ft$$

2. Abutment Scour

From Appendix page C-6 with $(V)(Y) = (13)(4) = 52$ and the length the abutment protrudes into the flow equal to 7 feet, abutment local scour equals 6.4 feet and:

$$Z_{tot} = Z_{deg} + Z_{gs} + Z_{ls} + Z_{bs} + Z_i + \frac{1}{2}h$$
$$= 2.25 + 2.9 + 6.4 + 0 + 2 + 0.5(4.3) = 15.7$$
$$\text{use } Z_{tot} = 16\ ft$$


BEGINNING OF CURVE
END OF CURVE
X
FLOW
PLAN
r
$h$ /2
$Y_{se}$
WATER SURFACE
EXISTING SLOPE
$Z_l$
$h$ /2
$Z_{ls}$
$Z_{gs}$
$Z_{deg}$
1:1
$Z_{bs}$
B.C.
PROFILE
NOT TO SCALE
E.C.
X
1:1




PROFILE

NOT TO SCALE

**Pier & Abutment Scour**

# Invert Stabilization Example

## EXAMPLE 4

## INVERT STABILIZATION EXAMPLE FOR A SOFT BOTTOM CHANNEL

Given:

| | |
|---|---|
| Natural stream slope, S | = 0.01 |
| Peak discharge, $Q_{cap}$ | = 57,200 cfs |
| 25% of $Q_{cap}$, $Q'_{cap}$ | = 14,300 cfs |
| Natural channel bottom width | = 700 ft |
| Proposed channel bottom width, b | = 360 ft |
| Average velocity under existing conditions for $Q_{cap}$, $V_{ex}$ | = 16.8 fps |
| Velocity after project condition for $Q_{cap}$, $V_p$ | = 21.4 fps |
| Average velocity under existing condition for 25% of $Q_{cap}$, $V'_{ex}$ | = 9.7 fps |
| Velocity after project condition for 25% of $Q_{cap}$, $V'_p$ | = 12.5 fps |

An existing gravel mining operation upstream of the study reach traps approximately 2/3 of the sediment supply.

The first step is to determine if invert stabilization is required.

***Percent increase in velocity due to $Q_{cap}$, AV***

$$\Delta V = 100\left(\frac{V_p - V_{ex}}{V_{ex}}\right) = 100\left(\frac{21.4 - 16.8}{16.8}\right) = 27\%$$

***Percent increase in velocity due to $Q'_{cap}$, AV'***

$$\Delta V' = 100\left(\frac{V'_p - V'_{ex}}{V'_{ex}}\right) = 100\left(\frac{12.5 - 9.7}{9.7}\right) = 29\%$$

Since $\Delta V'$ is greater than $\Delta V$, a velocity increase of 29% is used to determine if invert stabilization is required.

From the graph in Appendix C-2, for 29 percent velocity increase and natural slope of 0.01, the point of intersection falls above the curve representing 2/3 reduction in sediment supply. Therefore, invert stabilizers such as point stabilizers or drop structures are required.

To find the equilibrium slope, use the graph in Appendix C-1 for 29 percent increase in velocity and **2/3** reduction in sediment supply to get:

$$\text{Natural slope} - \text{Equilibrium slope} = 0.0068$$
$$S - S_{eq} = 0.0068$$
$$S_{eq} = 0.01 - 0.0068 = 0.0032$$

Use Equation 5.6 to calculate the spacing of the drop structures or point stabilizers.

a. For drop structures:

Nominal height = 5 ft

$$D = \frac{H}{(S_0 - S_{eq})} = \frac{4}{(0.01 - 0.0032)} = 735\, ft$$

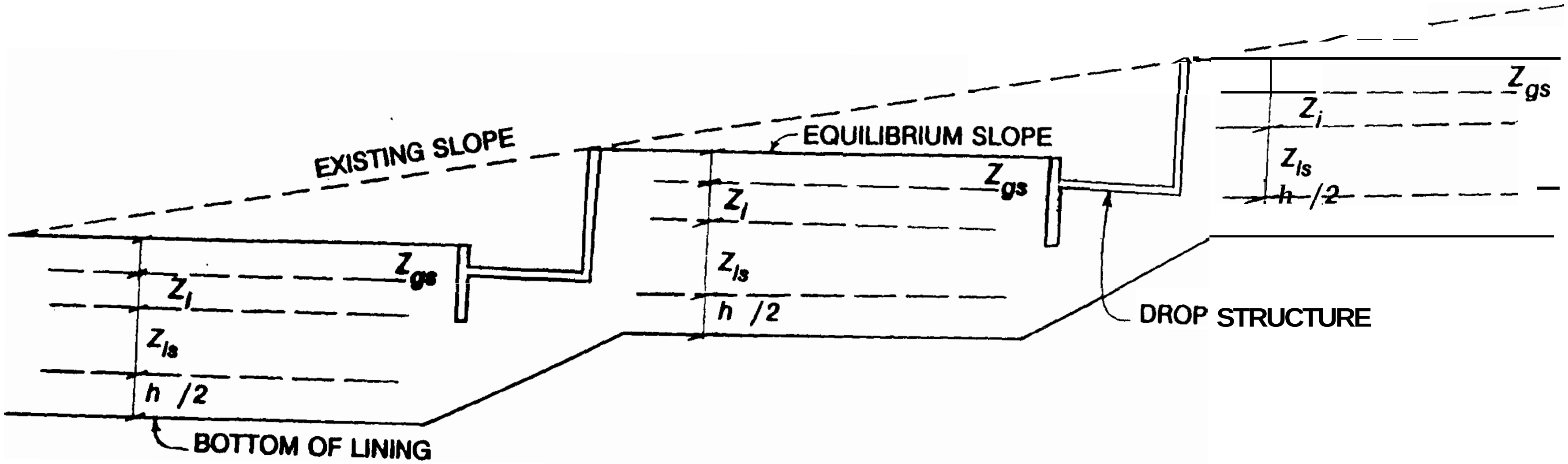
b. For point stabilizers:

Nominal height = 2 ft

$$D = \frac{H}{(S_0 - S_{eq})} = \frac{2}{(0.01 - 0.0032)} = 294\, ft$$

To find the riprap size, compute the velocities immediately downstream of the structures for $Q_{cap}$. In this example, the velocity downstream of the drop structure was 15 fps and downstream of the point stabilizer was 21 fps. This was determined through hydraulic analysis using a Manning's *n* equal to 0.025 for the invert. The velocity for the drop structure was computed using the equilibrium slope, and the natural slope was used for the point stabilizers which are constructed at grade.

From Appendix C-10 the minimum riprap diameter for the drop structure is 38 inches and for the point stabilizer is 48 inches.




**PROFILE**

NOT TO SCALE

**Drop Structure**