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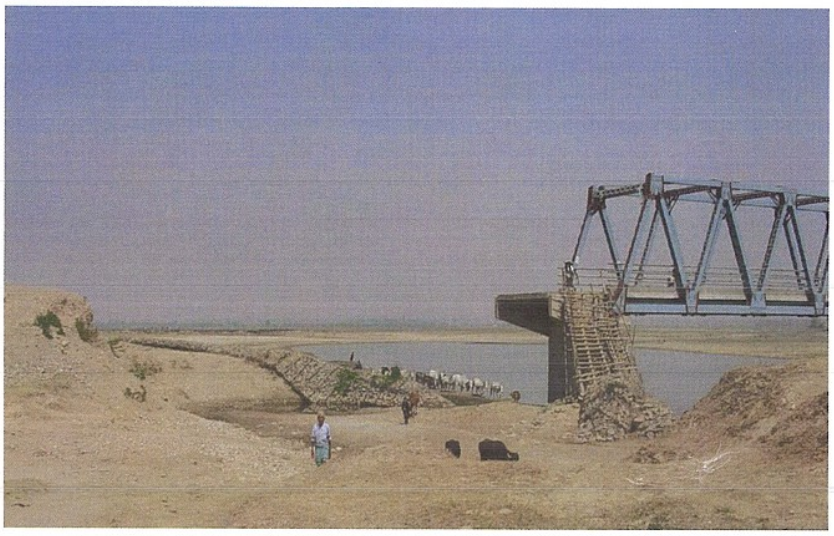
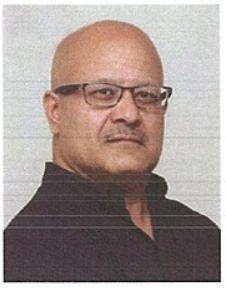
Ministry of Physical infrastructure and Transport

Department of Roads



Guidelines on Hydrologic and Hydraulic Analysis and River Training Works for Bridge Design

AUTHOR'S NOTE



Department of Roads (DoR) is responsible for planning, implementing, operation and maintenance of strategic Roads and Bridges across the country. It plans and implements more than hundreds of bridges every year. In this process it has been a challenge to DoR to select a suitable site at which a bridge can be built economically, at the same time satisfying the demands of traffic, the stream safety and aesthetics. The issues of outflanking, bank erosion, overtopping, undermining of foundations are the main concern in bridges at the plains of the country. These had resulted in bridge failure in some cases. Similarly choking of bridge opening and estimation of scour had been apprehension during design of bridge across gravel bed and boulder Stage Rivers of Nepal.

In the efforts towards standardization of hydrologic and hydraulic analysis and related river training works, the objective of this guideline is to explain comprehensively the steps required for a sound bridge design in plains, hills and mountain environment of Nepal based on critical review of existing practice, international existing guidelines, and local experiences. This guideline covers the topics of determination of design discharge, high flood level, scour estimation, related example calculations and river training works.

I am sure that this guideline would provide all relevant information at one place to the bridge design Engineers dealing with hydrological and hydraulic analysis. Finally I would like to express my sincere thanks to Department of Roads for entrusting me to carry out this works.

.....

Prof. Dr. Narendra Man Shakya

FOREWORD

Hydrological Investigation is one of the important aspect in design of river Bridges. Selection of economic bridge site with due consideration of traffic demands, stream characteristics, safety and aesthetics is a technically challenging job in itself. At present times, when large number of bridges are being built across the nation, DoR, as a Government body responsible for design and construction of Bridges along Strategic Road Networks feels the need for uniformity in various aspects of Bridge design. Guidelines on hydrological and hydraulic Analysis and River Training Works for Bridges Design shall enhance the bridge design and maintain uniformity as well.

This guideline is based on the latest IRC codes with added methodologies and explanations wherever necessary. It aims to address the hydrological dilemmas in the bridge design with proper explanations and sample calculations.

I would like to express my gratitude towards Prof. Dr. Narendra Man Shakya, Hydrologist for his valuable efforts in developing this guideline and to the entire team of Bridge Branch for their continued support.

I believe this guideline shall be a milestone in design of bridges for all the agencies in Nepal.

Thank you.

.....

Arjun Jung Thapa
Director General
Department of Roads.

ACKNOWLEDGEMENT

Guidelines on Hydrologic and hydraulic Analysis and River Training works for Bridge Design is prepared to bring uniformity in the hydrologic and hydraulic aspects of bridge design. This guideline is based on the latest publications of IRC. Various methods and empirical formulas suitable in the Nepalese context have been recommended to avoid any ambiguity in the design process. This guideline shall be reviewed periodically as per the need and feedbacks/reviews from the users of the guideline.

I would like to express my special thanks of gratitude to Prof. Dr. Narendra Man Shakya for helping us develop this guideline with his knowledge and years of experience. The guideline would not have been possible without the sincere trust and support of Mr. Arjun Jung Thapa, Director General, Department of Roads. I would also like to extend my gratitude to all the Senior Divisional Engineers and Engineers of the Bridge Branch for their incessant efforts for the preparation of this guidelines.

I believe this guideline shall be a helpful tool for everyone associated with Bridge Design in Nepal.

Thank you.

.....

Dipak Shrestha
Deputy Director General
Bridge Branch

TABLE OF CONTENT

BACKGROUND 1
1. HYDROLOGICAL INVESTIGATIONS 3
1.1 DETERMINATION OF DESIGN DISCHARGE 3
1.1.1 Ungauged Sites 3
1.1.2 Gauged Sites 9
1.1.3 Divided flows 10
1.1.4 Consideration Of Sediment Concentration 11
1.1.5 Consideration Of Climate Change 11
1.2 DETERMINATION OF WATER WAYS 11
1.2.1 General Rules 11
1.2.2 The General Instruction for Alluvial Streams 11
1.3 CROSS SECTIONAL SURVEY 13
1.4 HYDRAULIC ANALYSIS 15
1.4.1 General Instruction 15
1.4.2 Subdivision and Channel Bank Locations 16
1.4.3 Water Surface profile 16
1.4.4 Calculation of Conveyance 18
1.4.5 Manning's n 19
1.4.6 Calculation of Mean Kinetic Energy Head 20
1.4.7 Computation of Water Surface Profile 23
1.5 BRIDGE SCOUR 27
1.5.1 Degradation/Aggradation 27

1.5.2 General And Local Scour 27
1.5.3 Computation Methods 28
1.6 RIVER TRAINING WORKS 42
1.6.1 General 42
1.6.2 Design of Guide bund 43
1.6.3 Length 45
1.6.4 Crest Height 46
1.6.5 SHAPE and Size 46
1.6.6 Downstream Extent 46
1.6.7 Rip Rap 46
1.6.8 OTHER Design Concerns 47
ANNEX 52

LIST OF FIGURES

TITLE **PAGE NO.**

Figure 1. Logarithmic plot of intensity and duration curves 8
Figure 2. Cross sectional information u/s and d/s of bridge site 14
Figure 3. Representations of Terms in the Energy Equation 17
Figure 4. Conveyance Sub-division Method 18
Figure 5. Mean Energy Head Calculation 21
Figure 6. Plan and sectional view of cross sections for hydraulic computation 24
Figure 7. Blench curve 30
Figure 8. Cross section parameters for the component velocity approach of Maza alverez and Echavarria Alfaro (1973) 31
Figure 9. Hydraulics of uniform flow through long rectangular contraction 34
Figure 10. Typical Guide Bank 44
Figure 11. Bradley's curve 48

LIST OF TABLES

TITLE **PAGE NO.**

Table 1. Run off Coefficient 8
Table 2. Manning's rugosity coefficient n 20
Table 3. Subcritical Flow Contraction and Expansion Coefficients 22
Table 4. Increase over design discharge 27
Table 5. Lacey's silt factor as function of grain size 28
Table 6.Critical Tractive Force 33
Table 7. Values of coefficients K1, K2, K3 36
Table 8. Correction factor for pier nose (K1) 37
Table 9. Correction for angle of attack of flow (K2) 37
Table 10. Correction factor for bed condition (K3) 37
Table 11. Coefficient (K1) for abutment shape 40
Table 12. Shape factors for uniform piers and shorter abutments 42

BACKGROUND

Department of Roads (DoR) is responsible for planning, implementing, operation and maintenance of strategic Roads and Bridges across the country. During recent years DoR had intensified its activities in planning and implementation of bridges. It plans and implements more than hundreds of bridges every year. In this process it has been challenge to DoR to select a suitable site at which a bridge can be built economically, at the same time satisfying the demands of traffic, the stream, safety and aesthetics. The success of the final design will depend on the thoroughness of the information furnished during investigation. In this regard DoR has been experiencing some inconsistencies in estimation of hydrologic and hydraulic design parameters of bridges. This has lead either under or highly over sizing of bridge elements. In particular, the issues of outflanking, bank erosion, overtopping, undermining of foundations are the main concern in bridges at the plains of the country. These had resulted in bridge failure in some cases. Similarly choking of bridge opening and estimation of scour had been apprehension during design of bridge across gravel bed and boulder Stage Rivers of Nepal.

The most common cause of bridge failures is floods with the scouring of bridge foundations being the most common cause of flood damage to bridges. The hydraulic design of bridge waterways is typically based on flood frequencies somewhat less than those recommended for scour analysis. The main concern to DoR is just non-availability of such comprehensive guidelines for hydrologic and hydraulic analysis and river training works. The need to minimize future flood damage to the nation's bridges requires that additional attention be devoted to developing and implementing, improved procedures for hydraulic designing of bridges for scour and training works. Such guideline will serve as yardstick to DoR for maintaining consistencies in hydraulic design of bridge and related structures which otherwise is widely different among the consultants hired for such jobs. It is therefore necessary to look for standardization of such analysis so as to reduce the time spent on project preparation as well as to assure the design safety and stability of stream in the vicinity of bridge site.

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Following articles are comprehensive guidelines for standardization of hydrologic and hydraulic analysis and related river training works required for a sound bridge design in plains, hills and mountain environment of Nepal based on critical review of existing practice, international existing guidelines, and local experiences.

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1. HYDROLOGICAL INVESTIGATIONS

The hydrologic analysis process for sizing the bridge water way shall include the evaluation of design discharge at the site.

1.1 DETERMINATION OF DESIGN DISCHARGE

The design discharge for which the waterway of the bridge is to be designed, shall be based on maximum flood discharge of 100 years return period. In case where the requisite information is not available, the design discharge shall be the maximum estimated discharge determined by consideration of the following or any other rational method.

For ungauged site (catchment area ≤ 1000 km^2) from the rainfall and other characteristics of the catchment:

i) By the use of an empirical formula, applicable to that region, or

ii) By a recognised method, provided it is possible to evaluate for the region concerned the various factors employed in that method.

For the gauged sites from the records available, if any, of discharge observed in the stream at the site of the bridge, or at any other site in its vicinity.

1.1.1 Ungauged Sites

(i) For catchment area ≤ 1000 km^2

Although records of rainfall exist to some extent, actual records of floods are seldom available in such sufficiency as to enable the engineer accurately to infer the worst flood conditions for which provision should be made in designing a bridge. Therefore, recourse has to be taken to theoretical computations from rainfall and runoff relationships

1.1.1.1 The catchment area of the river at bridge site should be demarcated on 1: 25000 topographic map, google map and Digital elevation model. The differences in the measured area from those maps should not be more than 5% (See Figure 1 of Annex). The length of the river L, length to the centre from the bridge site L_c, Maximum elevation H_{max}, and the minimum elevation (H_{min}) should all be read from the map. The locations of rain gauge stations within and in the periphery of the basin should be marked in the same map.

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1.1.1.2 The annual maximum daily rainfall values of the relevant station shall be collected and 24 hours maximum rainfall for 100 years returns periods should be calculated by frequency analysis. Frequency analysis shall be carried out using five of the most widely used fitting distributions: Gumbel Max, Log Pearson 3, Lognormal and Gamma and GEV (Generalized Extreme Values) distributions. Goodness of fit like Chi-squared and Kolmogorov Smirnov tests shall be applied for selecting best fitting distribution in conjunction with efficiency of observed and estimated values of cumulative frequencies of variates.

Peak discharge for various return period shall be estimated by using various approaches such as, WECS, Modified Dicken's, B.D. Richard's, Snyder's method, Rational formula method as described below and example computation are shown in Tables 1 through 4 of ANNEX.

(a) Water and Energy Commission Secretariat (WECS) Approach

Water and Energy Commission Secretariat has published a regional hydrological analysis report according to which the flood flows in any river of catchment area below 3000m of elevation, is given by

$Q_2 = 1.8767\ (A+1)^{0.8783}$

$Q_{100} = 14.63\ (A+1)^{0.7342}$

Where the subscript 2 and 100 stand for the return periods in number of years, and A=Catchment Area in sq. Km.

The flood flows for other return period R is then given by

$Q_R = \exp(\ln Q_2 + s\sigma)$

Where $\sigma = \ln(Q_{100}/Q_2)/2.326$ is the standard deviation of the natural logarithms of annual floods and s stands for a standardized normal variate for a particular return period R. The values of s for different return period of 2 yr, 5yr, 10, 20, 50, 100, and 200 years are 0, 0.842,1.282, 1.645, 2.054, 2.326, and 2.576 respectively.

(b) Modified Dicken's Method

Dicken's method is an empirical one where the peak discharge is given by

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$Q_T = C_T A^{3/4}$

where C_T for the return period T is given by

$C_T = 2.342 \log(0.6T)\log(1185/p)+4$

with $p = 100(A_S + 6)/A$

A_S is the snow covered area out of total catchment area A.

(c) B.D. Richards' Method

Another widely used method of flood estimation using rainfall and catchment characteristics is the B.D.Richards' method, which has been comprehensively used in Mahakali Irrigation Project. The procedure involved is as follows:

Initially certain value is assumed for T_c, then

$F = 1.09352-0.06628 \ln (a)$

$D = 1.102L^2/ (S.F)$

$R_{TC} = R_T \text{ X } 22.127\ T_c^{0.476577}/100$

$I = R_{TC} / T_C$

$K_R = 0.651\ (T_C + 1)$

$C_{KR} = 0.95632/ K_R^{1.4806}$

$T_{C3} = D * C_{KR}$

$T_{C2}= (TC3/0.585378)^{1/2.17608}$

Where,

T_c= Time of concentration in hr.

A = Catchment area in sq. km.

L = Catchment length in km.

S = Catchment slope in % (= $(H_{max}-H_{min})/L \times 100$)

F = Areal reduction factor

R_T = 24 hour rainfall for return period T

I = Rainfall intensity corresponding to the time of concentration T_c

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T_{c2} = Second estimate of time of concentration

If the difference between the first estimate T_c and the resulting second estimate T_{c2} is greater than 5%, the whole process is repeated with $T_c = T_{c2}$

To calculate the flood discharge then, the following formula is used

$Q = 0.222AIF$ in m^3/s.

(d) Snyder's Method

Another well-established method widely used in Nepalis the Snyder's method. For an ungauged river, the flood flow estimation by deriving a synthetic unit hydrographs based on known physical characteristics of the basin is resorted to. The method is based on the analysis on large number of hydrographs from drainage basins ranging in area from 25 sq. km to 25000 sq. km. The method which has been used in Nepal to estimate the flood discharge hydrographs in Nepalese rivers is based on the following relationships.

$t_{pr} = 0.75\ C_t(L\ L_c)^{0.3}$

$t_{pr} = t_{pr} /5.5$

$t_{pR} = t_{pr} + 0.25\ (t_R - t_r)$

$q_{pR} = 2.78\ C_p / t_{pR}$

$Q_{pR} = q_{pR} * (C_A * A) * R$

Where,

t_{pr} = lag time from mid-point of effective rainfall of duration t_r to peak of unit hydrograph in hours.

t_r = Standard duration of effective rainfall in hours.

C_t and C_p values for a given catchment shall be obtained from the maps (Annex Figure 2, 3) prepared for Nepal.

(e) Rational method

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The simplest method to estimate the maximum of run-off is the Rational-Formula which is

$$Q = \frac{CIA}{360}$$

Where, Q is the rate of run-off in cubic meters per second, I is intensity in mm per hour, A is the catchment in hectares, and C is a dimensionless constant, the run-off coefficient is estimated by identifying land use/land covers and soil type of each watersheds. This is done by overlaying land use/ land cover and soil map onto the watershed maps (see Table 1.1.1).

This method is applicable only in case when time of concentration is less than duration of rainfall whose intensity can be considered constant and uniformly distributed over the catchment. Therefore, this method may be applicable only for small watershed having time of concentration less than one hour.

Here I is obtained from intensity-duration-frequency curve corresponding to t_c, the time of concentration, which is given by

$$t_c = \{0.87L^3/H\}^{0.385}$$

where, t_c is in hr, L, the longest dimension of stream or watershed is in km, H, the stream fall of vertical drop of the flow form farthest point to the outlet point is in m. If the time of concentration calculated from above equation is less than 15 minutes then it is assumed to be 15 minutes as recommended by ASCE.

Based on the maximum daily rainfall R_{24} as tabulated earlier, the Rainfall Intensity for any duration is obtained from highest of the following two methods:

(A) Mononobe equation which is generally applied in mountainous catchment

$$I_{D,T} = \frac{R_{24,T}}{24}\left(\frac{24}{D}\right)^{\frac{2}{3}}$$

Where T is the retum period in years and D the duration which is taken as time of concentration t_c in hr. Here $I_{D,T}$ is in mm/hr and $R_{24,T}$ is the maximum daily rainfall of return period T.

(B) Based on local observations

7

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For three stations in hills of Nepal (Kathmandu, Okhaldhunga and Pokhara), maximum rainfall observations of 5 min, 10 min, 30 min, 1 hr, 2hr, 6 hr , 12 hr , 1 day are plotted in terms of logarithmic of duration in hours versus logarithmic of intensity in mm/hr. They plot in a parallel sets of lines (see fig). Make a parallel line on the graph with a point [Log (24), Log ($R_{24,T}/24$)] until desired time of concentration is reached to get the Log ($I_{D,T}$).

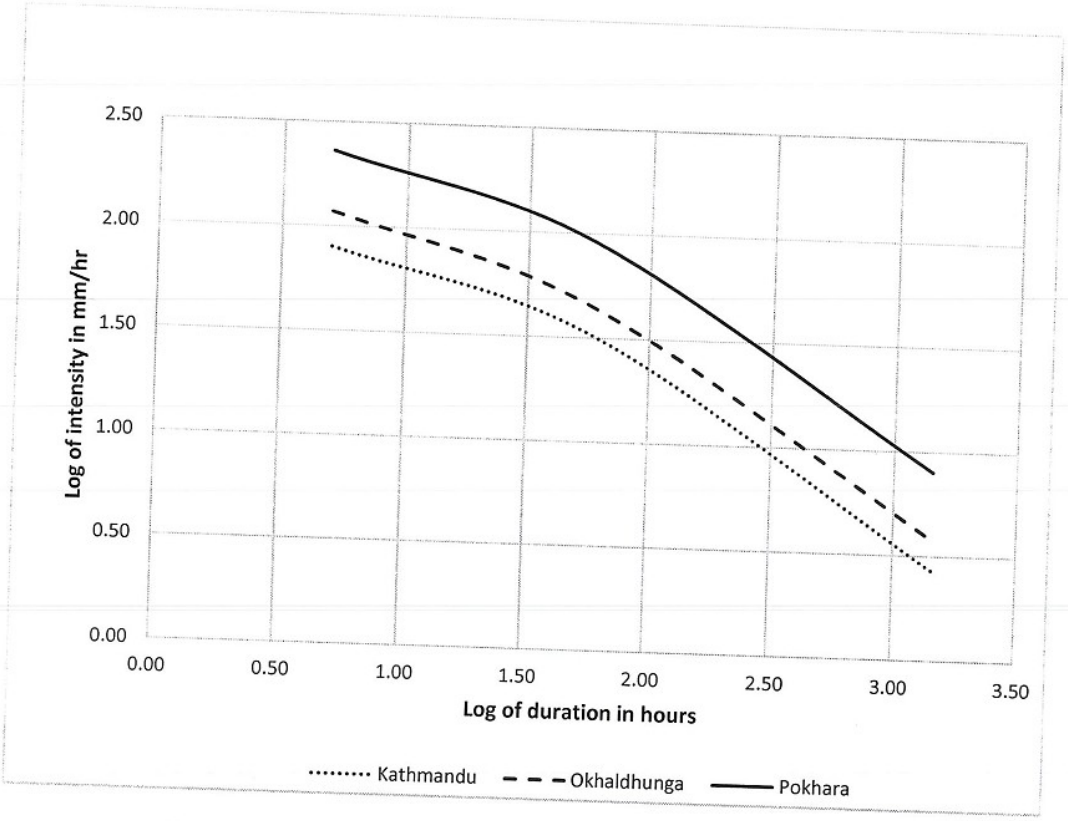

Figure 1. Logarithmic plot of intensity and duration curves

Table 1. Run off Coefficient

Catchment Characteristics	C
Steep, bare rock	0.90
Rock, steep but wooded	0.80

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Plateaus lightly covered, ordinary ground, bare	0.70
Clayey soils, stiff and bare	0.60
Clayey soils, lightly covered	0.50
Loam, lightly cultivated or covered	0.40
Loam, largely cultivated	0.30
Sandy soil, light growth	0.20
Sandy soil, covered, heavy bush	0.10
Jungle areas	0.10-0.20

1.1.1.3 The values obtained from above mentioned methods should be compared. The highest of these values should be adopted as the design discharge Q, provided it does not exceed the next highest discharge by more than 50 percent. If it does, restrict it to that limit.

(ii) For catchment area > 1000 km^2

1.1.1.4 If the ungauged part of the catchment is greater than 1000 km^2 in addition to methods described for catchment area ≤ 1000 km^2, $R_{24,T}$ shall be disaggregated to hourly values using alternate block method. That hourly event shall then be routed to obtain peak discharge using calibrated parameters of appropriate catchment model fitted to similar gauged catchment. The peak discharges obtained from the methods explained in clauses 1.1.1.1 through 1.1.1.4 should be compared. The highest of these values should be adopted as the design discharge Q, provided it does not exceed the next highest discharge by more than 50 per cent. If it does, restrict it to that limit.

1.1.2 Gauged Sites

In any case the analysis of peak discharge shall be carried out using flood frequency method if the river is gauged in the vicinity of bridge site. For the location with catchment area greater than 3000 km^2, flood frequency analysis using gauged data of more than 25 years in the vicinity of bridge site is must.

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In such situation the design discharge shall be based on flood frequency analysis of observed instantaneous annual maximum flow records of the river.

Flood frequency analysis shall be carried out at gauging site using 4 of the most widely used fitting distributions: Gumbel Max, Log Pearson 3, Lognormal and Gamma and GEV (Generalized Extreme Values) distributions. Goodness of fit like Chi-squared and Kolmogorov Smirnov tests shall be applied for selecting best fitting distribution in conjunction with efficiency of observed and estimated values of cumulative frequencies of variates.

When the catchment area at bridge site is greater than that of gauging station of the river, peak discharge of return period T at bridge site is estimated as whichever is greater of following:

- Catchment ratio times the calculated peak discharge for the same return at gauging site.
- Calculated peak discharge for the same return at gauging site plus the adopted peak discharge using methods described in clause 1.1.1 for the area downstream of gauging site.

When the catchment area at bridge site is less than that of gauging station of the river, peak discharge of return period T at bridge site is estimated as whichever is greater of following:

- Catchment ratio times the calculated peak discharge for the same return at gauging site.
- Calculated peak discharge for the same return at gauging site minus the adopted peak discharge using methods described in clause 1.1.1 for the area between gauging and bridge site.

1.1.3 Divided flows

When the river main channel divides into two distinct permanent channels down streams in plains for example Tinau/Danab river at Butwal, peak discharge at bridge site located downstream of junction at one of the divided channel shall be estimated as peak discharge calculated using above mentioned methods minus bank full carrying capacity of another divided channel.

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1.1.4 Consideration Of Sediment Concentration

The design discharge shall be increased by 10% in case the river reach is subject to enormous quantity of sediment transport.

1.1.5 Consideration Of Climate Change

Most of the climate change models project increase in rainfall during monsoon in Nepal. The design discharge shall be increased by 10% to consider the impact of climate change in flood.

1.2 DETERMINATION OF WATER WAYS

1.2.1 General Rules

The hydraulic opening of a waterway bridge is determined based on a combination of the following:

- The contributing watershed area,
- The magnitude and frequency of the design flood,
- The roadway alignment and profile, and
- Physical and environmental constraints.

The hydraulic design process for sizing the bridge waterway opening shall include the evaluation of water surface elevations in the main channel for existing conditions and for proposed conditions. A comparison of the elevations between these two conditions shall be made to identify the effects of the bridge on the waterway. These results shall clearly identify and mitigate any backwater effects caused by the project.

1.2.2 The General Instruction for Alluvial Streams

1.2.2.1 The linear waterway of a bridge across an exclusively alluvial stream should normally be kept equal to the width required for stability that it is certain to come to regime according to Lacey. It will then be stable and have a section and slope conforming to his equations. For wide alluvial streams the stable width W can be taken equal to the wetted perimeter

i.e. $W = P = 4.8\ Q^{1/2}$

1.2.2.2 Unstable Meandering Streams:

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A large alluvial stream, meandering over a wide belt, may have several active channels separated by bars, island or shallow sections of nearly stagnant water. The total width of such streams may be much in excess of the regime width required for stability. In bridging such a stream it is necessary to provide training works that will contract the stream. The cost of the latter, both initial and recurring, has to be taken into account infixing the linear waterway.

In the ultimate analysis it may be found in some such cases, that it is cheaper to adopt a linear waterway for the bridge somewhat in excess of the regime width. But as far as possible, this should be avoided. When the adopted linear waterway exceeds the regime width it does not follow that the depth will become less than the regime depth. Hence, such an increase in the length of the bridge does not lead to any countervailing saving in the depth of foundations. On the contrary, an excessive linear waterway can be detrimental in so far as it increases the action against the training works.

In such cases, the waterway shall be constricted to the extent in which safe passage of design flood can be managed by less than 75m of quarter ellipse guide bank calculated using nomogram due to Bradley (Figure 1.6.2). The required length of the guide bund is estimated from the nomogram using following hydraulic characteristics of the bridge site.

Q = Total discharge of the stream, m^3/s

Q_f = Discharge intercepted by embankments on either flood plains, m^3/s

Q_{30m} = Discharge in 30 m of stream adjacent to the abutment, m^3/s

A_{n2} = Cross-sectional flow area at the bridge opening (constricted waterway) at design flood stage, m^2

V_{n2} = Q/A_{n2} = Average velocity through the bridge opening, m/s

Q_f/Q_{30m} = Guide bank discharge ratio

1.2.2.3 Contraction to be avoided:

The linear waterway of the bridge across an alluvial stream should not be less than the regime width of the stream. Any design that envisages contraction of the stream beyond the regime width, necessarily has to provide for much deeper foundation. Much of the saving in cost expected from decreasing the length of the bridge may

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be eaten up by the associated increase in the depth of the substructure and the size of training works. Hence, except where the section of the stream is rigid, it is generally troublesome and also ineffective from economy consideration to attempt contracting the waterway.

1.2.2.4 The General Instruction for Semi Alluvial Streams:

When the banks of a stream are high, well defined, and rigid (rocky or some other natural hard soil that cannot be affected by the prevailing current) but the bed is alluvial, the linear waterway of the bridge should be made equal to the actual surface width of the stream, measured from edge to edge of water along the HFL on the plotted cross-section at which the designed maximum discharge determined in accordance with Clause 1.1, can be passed without creating harmful afflux.

1.2.2.5 The General Instruction for Streams with Rigid Boundaries:

In wholly rigid streams the rule for semi alluvial streams applies, but some reduction in the linear waterway may, across some streams with moderate velocities, be possible and may be resorted to, if in the final analysis it leads to tangible savings in the cost of the bridge.

1.2.2.6 The General Instruction for flashy Rivers

If the river is of a flashy nature and the bed does not submit readily to the scouring effects of the flood, the waterway should be determined by the area velocity method taking into account the design flood level and its water spread, the characteristics of the bed materials and the water surface slope.

1.3 CROSS SECTIONAL SURVEY

Survey is a critical component of any bridge hydraulic investigations. The recommended survey requirements for a simple water course include cross sections for the reach both downstream and upstream of the crossing. An example of the required cross sections for a simple analysis can be seen below in fig.1.3.1

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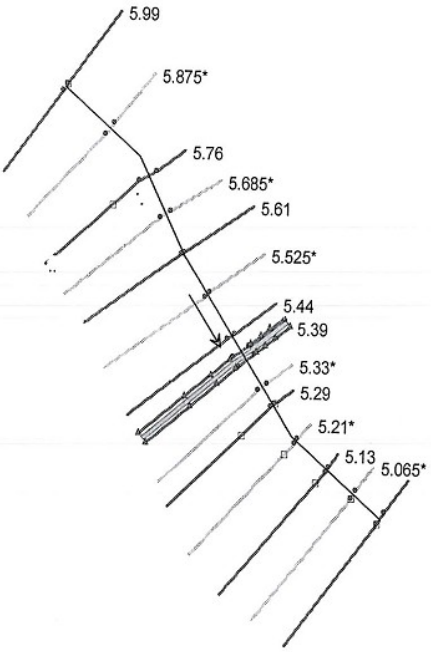

Figure 2. Cross sectional information u/s and d/s of bridge site

Downstream of the crossing, the general requirement is to have at least 10 cross sections spaced approximately 50 m apart. These cross sections should be at right angles to the flow and should extend to higher than the highest expected flood level, so that the model can represent all of the flow. The maximum surveyed level is usually higher than the bank level of the stream, since over-bank or floodplain flow is common. The cross sections should represent the downstream reach sufficiently to allow the backwater model to calculate the flood level immediately downstream of the bridge accurately. In some circumstances, some changes to this guideline may be needed if the water course is small or large or there are other special features. These 10 cross sections however are generally adequate for most requirements.

Upstream of the crossing, at least 10 cross sections are also suggested, also spaced about 50 m apart. These upstream cross sections ensure that the afflux or the impact of the crossing on flood levels can be calculated accurately. As well as the cross sections, the stream bed profile over the reach of the river upstream and downstream of the bridge site (included with the cross sections above) should also be surveyed. This provides assistance with the estimation of the flood profile and also helps to identify locations where there are changes

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in bed slope, which is useful in the hydraulic modelling. It is important that the cross sections should extend up to above the level of the highest expected flood. If the cross sections do not extend sufficiently high, the calculated flood levels and velocities will be too high because the cross sections are extended vertically in the hydraulic modelling. Photographs of the locations of the surveyed cross-sections should be taken to aid in hydraulic analysis of the site, especially if a site visit by the hydraulic specialist is unlikely. As well as these cross sections that are used for the hydraulic analysis, additional survey data may also often be useful. In some cases, survey of the floor levels of potentially affected buildings or other infrastructure should be taken to assist in assessing afflux at these buildings.

1.4 HYDRAULIC ANALYSIS

1.4.1 General Instruction

Hydraulic analysis is the calculation of flow patterns, flood levels and flow velocities for the bridge and nearby river reach. This information is the data used directly in the analysis of the bridge and the impacts of the bridge on local flooding in the water course. The hydraulic analysis relies on the survey data described above, which is used to control the flood flow patterns in the model.

Hydraulic calculation usually works upstream from a downstream boundary (for subcritical flow, which is the most common in natural channels), and this downstream level needs to be carefully assessed. The downstream boundary will usually be based on a downstream slope or the level of the receiving water. The downstream boundary should be sufficiently far downstream that the level does not affect the tail water of the bridge, but in some cases, careful analysis and sensitivity testing may be needed.

In case the bridge site is located at tributary which joins main stream at downstream not far away from the bridge site. The analysis should also consider the possibility of backwater from a downstream main stream since HFL of main stream could be quite higher than that of tributary. If these conditions seem likely, careful analysis of the downstream conditions is needed to check for the possibility of backwater.

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1.4.2 Subdivision and Channel Bank Locations

Hydraulic Analysis require the location of bank stations that separate the main channel area from the overbank or floodplain areas of the cross section. Sometimes it is hard to delineate the channel from the overbanks. A good rule of thumb is to partition the cross section into areas of similar "n" values and then determine the channel and overbank limits. Note: For alluvial channels, the channel limits tend to be at the same elevation since the water that the vegetation (with resulting roughness) uses to grow is also at the same elevation across the watercourse. This requires a field visit. A secondary consideration is to locate the bank limits where there are obvious breaks in the geometry (e.g., change in bank slope). The location of the bank stations can have an impact on the development of a floodway. If significant areas have differing roughness (say over 10% of the flow area), subdivide the overbanks and vary "n" by distance across the cross section. Doing this is particularly important if more than one discharge is to be calculated and the roughness changes as the floodplain increases in elevation going away from the channel. One of the main reasons for subdivision is to separate areas of the cross section where flow is more or less uniform (e.g., main channel flow versus floodplain flow). One recommendation is to subdivide a cross section with water in both the main channel and the floodplain when the depth of water in the floodplain is greater than half of the maximum depth measured in the main channel. In any case, the criteria used for subdivision of cross sections should be consistent for the entire reach under calculation.

1.4.3 Water Surface profile

Hydraulic computational procedure employs the Standard Step Method for profile computations. The standard step method is based on the principle of conservation of energy, i.e., the total energy head at an upstream section must equal to the total energy head at the downstream section plus any energy losses that occur between the two sections.

The energy equation can be written as follows:

$Z_2 + Y_2 + \alpha_2 V_2^2/2g = Z_1 + Y_1 + \alpha_1 V_1^2/2g + h_e$ Refer column 35 (Table 5 of Annex)

Where

Er. Arjun Jung Thapa
Director General

Z_1, Z_2 = Elevation of the main channel inverts

Y_1, Y_2 = Depth of water at cross sections

V_1, V_2 = Average velocities

α_1, α_2= Velocity weighing coefficients

g = Gravitational acceleration

h_e= Energy head loss

A diagram showing the terms of the energy equation is shown in following Figure 1.4.1

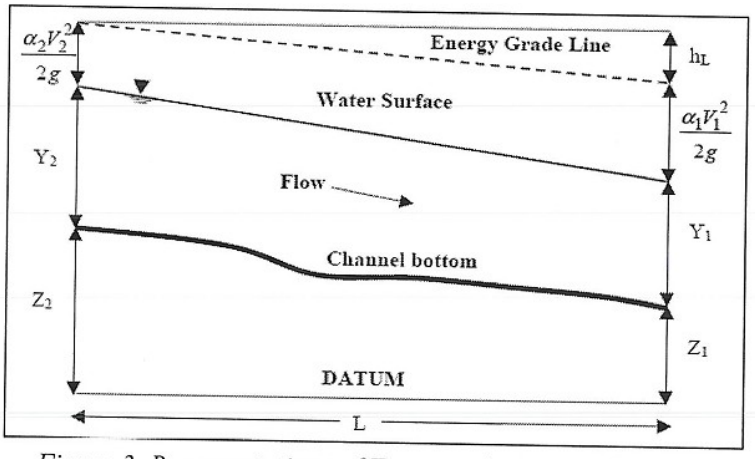

Figure 3. Representations of Terms in the Energy Equation

The energy head loss (h_e) is expressed as

$h_e - LS_f + C[\alpha_2 V_2^2/2g - \alpha_1 V_1^2/2g]$ Refer column 39, 40 and 41 (Table 5 of Annex)

Where

L = Discharge weighted reach length

S_f= Representative friction slope between two sections

C= Expansion or Contraction loss coefficient

The distance weighted reach length, L, is calculated as:

$L = (L_{lob}\, Q_{lob} + L_{ch}\, Q_{ch} + L_{rob}\, Q_{rob}) / (Q_{lob} + Q_{ch} + Q_{rob})$ Refer column 37 (Table 5 of Annex)

Er. Arjun Jung Thapa
Director General

Where,

L_{lob}, L_{ch}, L_{rob}= Cross-section reach length specified for flow in the left overbank, main channel and right overbank respectively

Q_{lob}, Q_{ch}, Q_{rob} = Arithmetic average of the flows between sections for the left overbank, main channel and right overbank respectively

1.4.4 Calculation of Conveyance

The determination of total conveyance and the velocity coefficient for a cross-section requires that flow be sub-divided into units for which the velocity is uniformly distributed (Figure 1.4.2). The approach shall be to sub-divide flow in the overbank areas using the input cross-section n-value break points (location where n-values change) as the basis for sub-division. Conveyance shall be calculated within each sub-division forms the following form of Manning's equation:

$Q = KS_f^{1/2}$

$K = 1/nAR^{2/3}$ Refer column 22, 23, 24 and 30 (Table 5 of Annex)

Where,

K = Conveyance for sub-division

n = Manning's roughness coefficient for sub-division

A = Flow area for sub-division and R = Hydraulic radius for sub-division

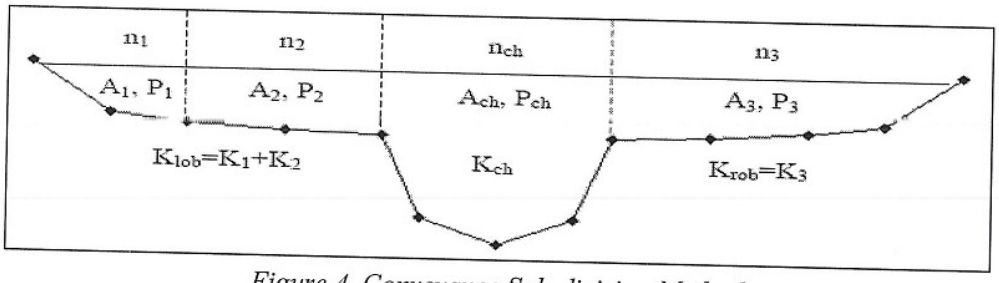

Figure 4. Conveyance Sub-division Method

All the incremental conveyances in the overbank are summed to obtain a conveyance for the left and the right overbank and the total conveyance for the cross-section is obtained by summing the three subdivision conveyances (left, channel and right).

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1.4.5 Manning's n

Selection of an appropriate value for Manning's n is very significant to the accuracy of the computed water surface profiles. The value of n is highly variable and depends on a number of factors including: surface roughness; vegetation; channel irregularities; channel alignment; scour and deposition; obstruction; size and shape of the channel; stage and discharge; seasonal changes; temperature; suspended material and bed load. Manning's n for most common types of channels, are shown in Table 1.4.1 below.

Er. Arjun Jung Thapa
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Table 2. Manning's rugosity coefficient n

Type of Channel and Description	Minimum	Normal	Maximum
A. *Natural Streams*			
1. **Main Channels**			
a. Clean, straight, full, no rifts or deep pools	0.025	0.030	0.033
b. Same as above, but more stones and weeds	0.030	0.035	0.040
c. Clean, winding, some pools and shoals	0.033	0.040	0.045
d. Same as above, but some weeds and stones	0.035	0.045	0.050
e. Same as above, lower stages, more ineffective slopes and sections	0.040	0.048	0.055
f. Same as "d" but more stones			
g. Sluggish reaches, weedy. deep pools	0.045	0.050	0.060
h. Very weedy reaches, deep pools, or floodways with heavy stands of timber and brush	0.050	0.070	0.080
	0.070	0.100	0.150
2. **Flood Plains**			
a. Pasture no brush			
1. Short grass	0.025	0.030	0.035
2. High grass	0.030	0.035	0.050
b. Cultivated areas			
1. No crop	0.020	0.030	0.040
2. Mature row crops	0.025	0.035	0.045
3. Mature field crops	0.030	0.040	0.050
c. Brush			
1. Scattered brush, heavy weeds	0.035	0.050	0.070
2. Light brush and trees, in winter	0.035	0.050	0.060
3. Light brush and trees, in summer	0.040	0.060	0.080
4. Medium to dense brush, in winter	0.045	0.070	0.110
5. Medium to dense brush, in summer	0.070	0.100	0.160
d. Trees			
1. Cleared land with tree stumps, no sprouts	0.030	0.040	0.050
2. Same as above, but heavy sprouts	0.050	0.060	0.080
3. Heavy stand of timber, few down trees, little undergrowth, flow below branches	0.080	0.100	0.120
4. Same as above, but with flow into branches	0.100	0.120	0.160
5. Dense willows, summer, straight	0.110	0.150	0.200
3. **Mountain Streams, no vegetation in channel, banks usually steep, with trees and brush on banks submerged**			
a. Bottom: gravels, cobbles, and few boulders	0.030	0.040	0.050
b. Bottom: cobbles with large boulders	0.040	0.050	0.070

1.4.6 Calculation of Mean Kinetic Energy Head

Mean kinetic energy head for each cross-section is obtained by computing the flow weighted kinetic energy heads for three sub-sections of the cross-sections (Left Overbank, Channel, and Right Overbank). Figure 1.4.3 illustrates the mean kinetic energy calculation process for a cross-section with a main channel and right over bank.

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All the incremental conveyances in the overbank are summed to obtain a conveyance for the left and the right overbank and the total conveyance for the cross-section is obtained by summing the three subdivision conveyances (left, channel and right).

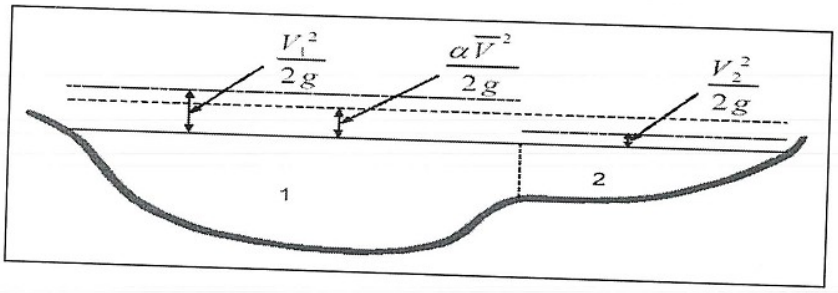

Figure 5. Mean Energy Head Calculation

To compute the mean kinetic energy it is necessary to obtain the velocity head weighting coefficient alpha. Alpha can be calculated by using following equation.

$\alpha = A^2_t\,[K^3_{lob}/A^2_{lob} + K^3_{ch}/A^2_{ch} + K^3_{rob}/A^2_{rob}] / K^3_t$ Refer column 31 (Table 5 of Annex)

Where,

A_t = Total flow area of cross-section

A_{lob}, A_{ch}, A_{rob} = Flow areas of left overbank, main channel right overbank respectively

K_t = Total conveyance of cross-section

K_{lob}, K_{ch}, K_{rob} = Conveyance of left overbank, main channel and right over bank, respectively

Based on the energy equation parameters shown in Figure 1.4.1, the water surface elevation is the sum of Y and Z. The change in energy head between adjacent cross-sections is equal to the head loss:

$H_2 = H_1 + h_L$ Refer column 42 (Table 5 of Annex)

Where,

H_1= energy head at cross-section 1 (m)

H_2= energy head at cross-section 2 (m)

h_L= energy head loss (m)

Er. Arjun Jung Thapa
Director General

The head loss between the two cross-sections is the sum of friction head loss and flow contraction/expansion head loss. Friction losses result from shear stresses between the water and channel bed and banks:

$Hf = L\overline{S_f}$ Refer column 39(Table 5 of Annex)

Where,

h_f= friction head loss (m)

L = distance between adjacent cross-sections (m)

Contraction/expansion head losses can occur due to the formation of eddies wherever there is a contraction or expansion of the channel.

$$h_0 = C\left|\frac{\alpha_2 V_2^{\,2}}{2g} - \frac{\alpha_1 V_1^{\,2}}{2g}\right|$$ Refer column 41(Table 5 of Annex)

Where,

h_o= contraction or expansion head loss (m)

C = contraction or expansion coefficient

Typical values for contraction and expansion coefficients for subcritical flow are shown in Table 1.4.2

Table 3. Subcritical Flow Contraction and Expansion Coefficients

Type of Channel Transition	Contraction	Expansion
None	0.0	0.0
Gradual	0.1	0.3
Typical Bridge Section	0.3	0.5
Abrupt	0.6	0.8

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1.4.7 Computation of Water Surface Profile

(a) For sections upstream and downstream of bridge site

The basic computational procedure is based on the iterative solution of the energy equation. Given the flow and water surface elevation at one cross-section, the goal of the standard step method is to compute the water surface elevation at the adjacent cross-section. For subcritical flow, the computations begin at the downstream boundary and proceed upstream; for supercritical flow, the computations begin at the upstream boundary and proceed downstream. At the boundary, the flow and water surface elevation must be known. The procedure is summarized below (assuming subcritical flow).

1. Assume a water surface elevation at cross-section 1.
2. Determine the area, hydraulic radius, and velocity of cross-section 1 based on the cross-section profile.
3. Compute the associated conveyance and velocity head values.
4. Calculate friction slope, friction loss, and contraction/expansion loss.
5. Solve the energy equation for the water surface elevation at the adjacent cross-section.
6. Compare the computed water surface elevation with the value assumed in step
7. Repeat steps 1 through 6 until the assumed and computed water surface elevations are within a predetermined tolerance.

The calculation shall be presented in the tabular format similar to table 5 through 7 of ANNEX.

(b) At sections in the vicinity of bridge location

Energy loss caused by bridge shall be computed in three parts. One part consists of losses that occur in the reach immediately downstream from the bridge, where expansion of flow generally take place. The second part is the losses at the structure itself. The third part consists of losses that occur in the reach immediately upstream of the structure, where the flow is generally contracting to get through the opening. These losses shall be computed using four cross sections as shown in fig.1.4.4

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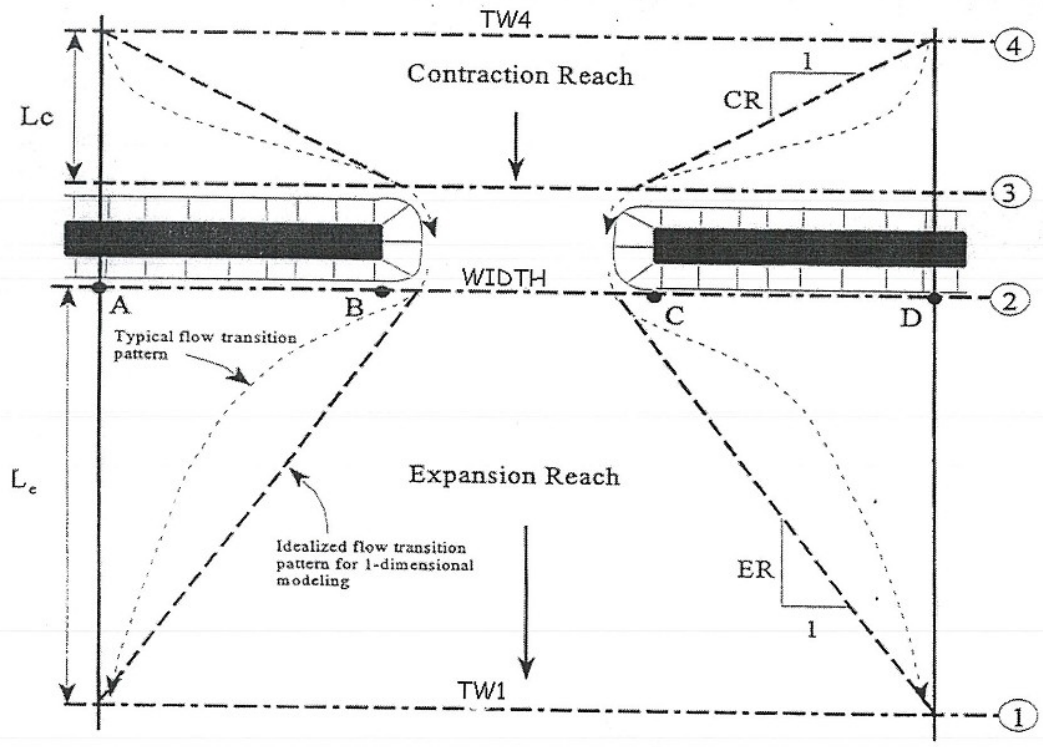

Le = ER * (TW1 - WIDTH) / 2
Lc = CR * (TW4 - WIDTH) / 2

Rule of thumb: ER = 2
CR = 1

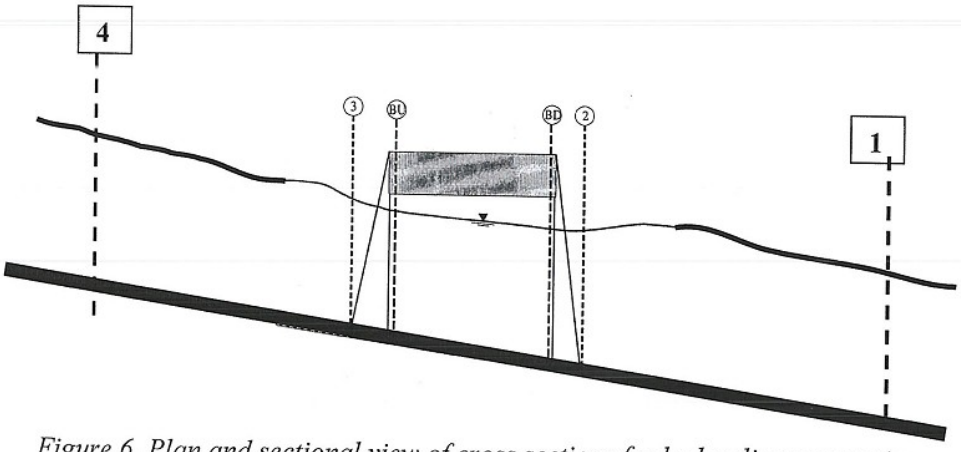

Figure 6. Plan and sectional view of cross sections for hydraulic computation

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The water surface profile through a bridge is computed by solving the energy equation. The method is an iterative solution performed from the exit cross section (1) to the approach cross-section (4). The energy balance is performed in steps from the exit section (1) to the cross section just downstream of the bridge (2) form just downstream of the bridge (2) to inside of the bridge at the downstream end (BD); from inside of the bridge at the downstream end (BD) to inside of the bridge at the upstream end (BU); Form inside of the bridge at the upstream end (BU) to just upstream of the bridge (3); and from just upstream of the bridge (3) to the approach section (4). A general energy balance equation form the exit section to the approach section can be written as follows:

$$h_4 + \frac{a_4 V_4^2}{2g} = h_1 + \frac{a_1 V_1^2}{2g} + h_{4-1}$$

Where: h_1 = Water surface elevation at section 1

V_1 = Velocity at section 1

h_4 = Water surface elevation at section 4

V_4 = Velocity at section 4

h_L = Energy losses from section 4 to 1

The incremental energy losses from 4 to 1 are calculated as follows

<u>From Section 1 to 2</u>

Losses from section 1 to section 2 are based on friction losses and an expansion loss. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between sections 1 and 2. The following equation is used for friction losses from 1 to 2:

$$h_{f_{1-2}} = \frac{BQ^2}{K_2 K_1}$$ Refer column 39 (table 6 of Annex)

Where B is the flow weighted distance between sections 1 and 2, and K1 and K2 are the total conveyance at sections 1 and 2 respectively. The expansion loss from section 2 section 1 is computed by the following equation:

$$h_e = \frac{Q^2}{2gA_1^2}\left[2\beta_1 - \alpha_1 - 2\beta_2\left(\frac{A_1}{A_2}\right) + \alpha_2\left(\frac{A_1}{A_2}\right)^2\right]$$

Er. Arjun Jung Thapa
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Where α and β are energy and momentum correction factors for non-uniform flow. α_1 and β_1 are computed as follows:

$$\alpha_1 = \frac{\Sigma(K_i^3/A_i^2)}{K_T^3/A_T^2}$$

$$\beta_1 = \frac{\Sigma(K_i^2/A_i)}{K_T^2/A_T}$$

α_2 and β_2 are related to the bridge geometry and are defined as follows:

$$\alpha_2 = \frac{1}{C^2}$$

$$\beta_2 = \frac{1}{C}$$

Where C is an empirical discharge coefficient for the bridge.

<u>From section 2 to 3</u>

Losses from section 2 to section 3 are based on friction losses only. The energy balance is performed in three steps: from section 2 to BD; BD to BU; and BU to 3. Friction losses are calculated using the geometric mean friction slope times the flow weighted distance between section. The following equation is used for friction losses from BD to BU:

$$h_{f(BU-BD)} = \frac{L_B Q^2}{K_{BU} K_{BD}}$$ Refer column 39 (table 6 of Annex)

Where K_{BU} and K_{BD} are the total conveyance at sections BU and BD respectively, and L_B is the length through the bridge. Similar equations are used for the friction losses from section 2 to BD and BU to 3.

<u>From Section 3 to 4</u>

Energy losses from section 3 to 4 are based on friction losses only. The equation for computing the friction loss is as follows:

$$h_{f(3-4)} = \frac{L_{AV} Q^2}{K_3 K_4}$$ Refer column 39 (table 6 of Annex)

Where L_{AV} is the effective flow length in the approach reach, and K_3 and K_4 are the total conveyances at section 3 and 4. The effective flow length is computed as the average length equal conveyance stream tubes.

Er. Arjun Jung Thapa
Director General

1.5 BRIDGE SCOUR

The estimated scour depth is a function of

- degradation or aggradation,
- general scour, and
- local scour

To provide for an adequate margin of safety, the scour for foundation shall be designed for a larger discharge over the design discharge determined as given in table 1.5.1.

Table 4. Increase over design discharge

Catchment area in km^2	Increase over design discharge in per cent
0 - 3000	30
3000 - 10000	30 - 20
10000 - 40000	20 - 10
Above 40000	10

For intermediate values of catchment area, linear interpolation may be adopted.

1.5.1 Degradation/Aggradation

Degradation and aggradation are the lowering and raising of the bed, respectively over the relatively long reaches and long time periods which can be judged qualitatively only at this stage. If the long term degradation of the channel bed is noted, the estimated scour depth resulting from the summation of the local and general scour components should be increased.

1.5.2 General And Local Scour

General scour refers to a more localised vertical lowering of the channel bed over relatively short time periods. For the purpose of this manual, general scour will be restricted to scour resulting from the contraction of the channel due to the encroachment of bridges or protection works. Local scour results from local disturbances in the flow such as the scour occurring at bridge piers and abutments.

Er. Arjun Jung Thapa
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1.5.3 Computation Methods

Following paras describe various methods to be adopted for the scour calculation. The values obtained from above mentioned methods should be compared. The highest of these values obtained from the combination of methods should be adopted as the design scour depth, provided it does not exceed the next highest value by more than 50 per cent. If it does, restrict it to that limit. However, the bottom level of foundation for pier and unprotected abutment from the lowest bed level of the river should be at least 5m. Sample example calculation are shown in Tables 8 and 9 of ANNEX.

1.5.3.1 General Scour

Mean scour depth shall be calculated with respect to predominant concentration of flow in any part of waterway by dividing the waterway into compartments as per the concentration of flow.

(a) The Regime Formulation of Lacey (1930)

(i) Alluvial Stream

Mean scour depth (y_{ms}) below the highest flood level (HFL) for natural channels flowing over scourable bed can be calculated theoretically from the following equation

$y_{ms} = 1.34\ (q^2/ K_{sf})^{0.33}$

q is discharge per unit width in $m^3/s/m$

K_{sf} = Silt factor for a representative sample of bed material obtained up to the level of anticipated deepest scour

The values of K_{sf} is presented in Table 1.5.2.

Table 5. Lacey's silt factor as function of grain size

Mean grain size d_m (mm)	Lacey's Silt factor K_{sf}
0.08	0.50
0.16	0.70
0.23	0.85
0.32	1.0
0.50	1.25
0.72	1.50
1.0	1.75
1.3	2.00

K_{sf} can be described by $K_{sf} = 1.76\ d_m^{0.5}$

Er. Arjun Jung Thapa
Director General

(ii) Scour in clay: Scour in clay is generally less than scour in sand. Normally in field we get a mixture of sand and clay at many places. For the purpose of assessment following definition of sand and clay can be given.

Sand - Where (ϕ) is equal to or more than 15° even if c (Cohesion of soil) is more than 0.2 kg/cm^2

Clay - Where ϕ is less than 15° & c (Cohesion of soil) is more than 0.2 kg/cm^2

Scour in sand of above definition can be calculated by the formulae given earlier. In clay instead of silt factor (K_{sf}) clay factor (K_{sfc}) is adopted-

$K_{sfc} = F(1 + \sqrt{c})$

Where

c = Cohesion in kg/cm^2 and

F = 1.5 ϕ for $\phi \geq 10° < 15°$

= 1.75 for $\phi \geq 5° < 10°$

= 2.0 for $\phi < 5°$

Scour depth (d_{sm}) = $1.34(D_b^2/K_{sfc})^{1/3}$

(b) The regime formulation of Blench (1969)

For sand of $0.06 < d_{50}$ (mm) ≤ 2

$$y_{ms} = 1.20\left(\frac{q^{2/3}}{d_{50}^{1/6}}\right)^{1/3}$$

For Gravel $S_s = 2.65$ and d_{50} (mm) > 2

$$y_{ms} = 1.23\left(\frac{q^{2/3}}{d_{50}^{1/12}}\right)^{1/3}$$

Where, q (m^3/s/m) is the discharge per unit width, for the main channel if the channel has the flood plains; d_{50} is the sediment size for which 50% of the sediment is finer. Knowing the water surface level corresponding to q, the scoured bed level can be determined.

Er. Arjun Jung Thapa
Director General

(c) **Blench zero bed factor**

The stable hydraulic geometry of a river channel depends upon a number of factors, which include discharge, characteristics of bed and bank materials, and the amount of sediment transported by the channel. For a given flood discharge of Q, depth of flow for a channel to be in regime condition is

$$y_r = (q^2/F_b)^{0.33}$$

Where,

y_r = Average regime depth measured from the water surface, m

q is discharge per unit width in $m^3/s/m$

F_b = **Blench's** " Zero Bed Factor"

A curve that relates the zero-bed factor to the median bed material size is shown in fig…..

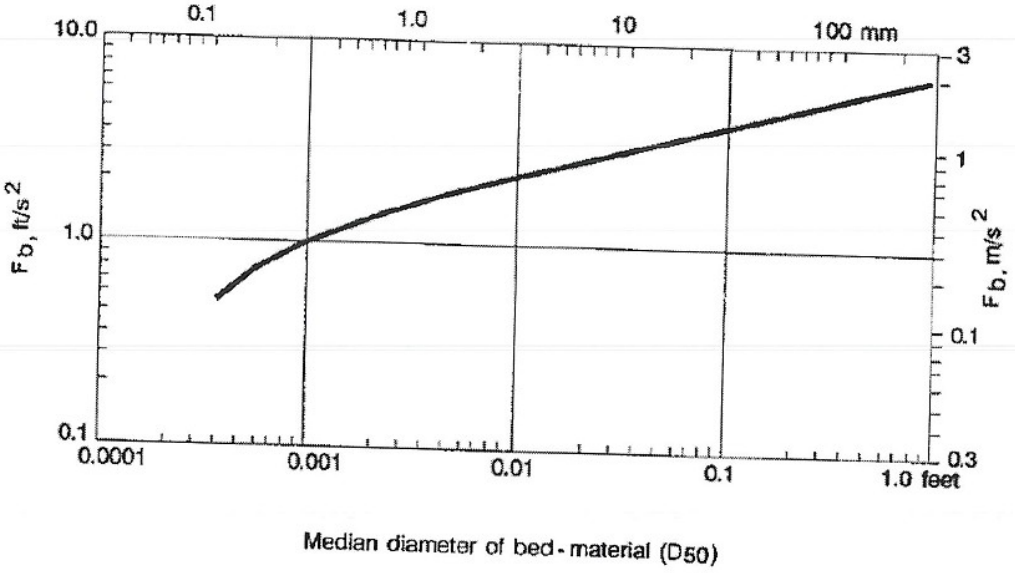

Figure 7. Blench curve

(d) The component velocities approach of Maza Alverez and Echavarria Alfaro (1973)

For sand or gravel of $d_{75} < 6$ (mm)

$$y_{ms} = 0.365\left(\frac{Q^{0.784}}{W^{0.784} d_{50}^{\ 0.157}}\right)^{1/3}$$

Er. Arjun Jung Thapa
Director General

$$y_s = \frac{y_o}{y_{mo}} y_{ms}$$

Where, y_{ms} (m) is the flow depth from the design water level to the mean scoured bed level. y_s (m) is the flow depth from design water level to the lowest scoured level; y_o (m) is the flow depth from design water level to the lowest level of the flow channel and y_{mo} is the depth from design water level to the mean bed level.

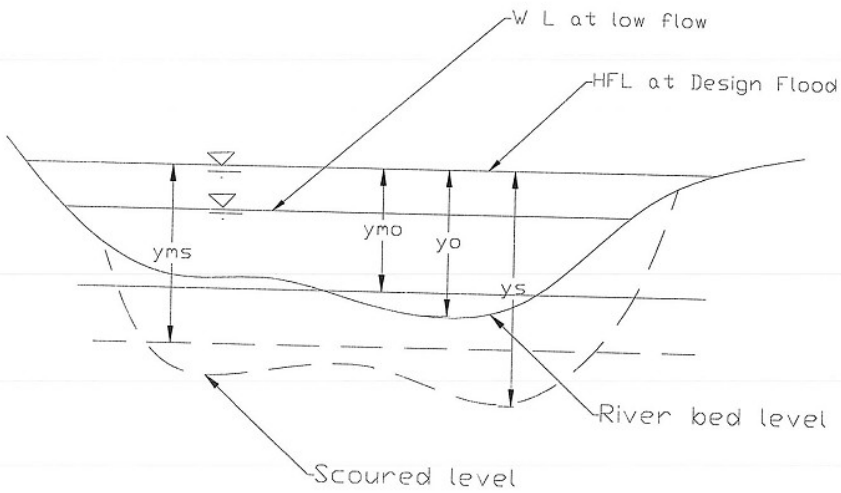

Figure 8. Cross section parameters for the component velocity approach of Maza alverez and Echavarria Alfaro (1973)

(e) The New Zealand railways formulation of Holmes (1974)

The scored depth y_s, for the combination of general scour and contraction scour is predicted as the

$$y_s = \frac{y_r v_1 K}{\sqrt{(A/W)}}$$

Where,

$$v_1 = c\left(\frac{Q}{A}\right)\left(\frac{y}{A/W}\right)^{2/3}$$

Er. Arjun Jung Thapa
Director General

y (m) is the un-scoured flow depth; y_r (m) water level raised from low water to flood stage; A (m^2) flow area of un- scoured profile; W (m) is the water way width allowing for the berm flow taken as 1.25 W_{80}; W_{80} is the width of waterway including the main channel which carries 80% of total flow; v_1 (m/s) approach velocity and the value of constant C is taken as 1.2.

(f) Armoring based design methodology of Borah (1989)

Borah (1989) evaluates the depth of scour below the original bed level δy_{as} as

$$\delta y_{as} = d_a \left[\left(\frac{1}{(1-e)P_c} \right) - 1 \right]$$

Where, e is porosity of bed materials, d_a is smallest armor size

For $u_* d_{50}/\nu \leq 10$, $\quad d_a = 68 \left(\frac{yS_f}{S_s - 1}\right)^{1.67} \left(\frac{u_*}{\nu}\right)^{0.67}$

For $10 < u_* d_{50}/\nu \leq 500$, $\quad d_a = 27 \left(\frac{yS_f}{S_s - 1}\right)^{0.86} \left(\frac{u_*}{\nu}\right)^{0.14}$

For $\frac{u_* d_{50}}{\nu} > 500$, $\quad d_a = 17 \left(\frac{yS_f}{S_s - 1}\right)$

u_* is shear velocity , $\quad u_* = (gyS_f)^{0.5}$

S_f is friction slope

ν is the kinematic viscosity of water

(g) Farraday and Charlton Equation

Basic equations:

$Y_2 = 0.38(V_1 Y_1)^{0.67} D_{50}^{-0.17}$ (Sand Bed Channel)

$Y_2 = 0.47(V_1 Y_1)^{0.8} D_{90}^{-0.12}$ (Gravel Bed Channel)

$Y_2 = 51.4\, n^{0.86}(V_1 Y_1)^{0.86} \tau_c^{-0.43}$ (Cohesive Bed Channel)

Where,

Y_2 is the average depth of general scour measured from the water surface, in metres;

Er. Arjun Jung Thapa
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Y_1 is the design depth equal to A_1/T_1, in metres;

V_1 is the design flow velocity in meters;

D_{50} is the size of the bed material, in metres, such that 50% of the stones by weight are smaller;

D_{90} is the size of the bed material, in metres, such that 90% of the stones by weight are smaller;

n is the Manning's roughness coefficient;

τ_c is the critical tractive stress for scour to occur, in Newtons/m^2, as indicated in table 1.5.3.

T_1 is the average top width for the design flow, metres; and

A_1 is the average bank full flow area for the design flow in m^2.

Table 6.Critical Tractive Force

CRITICAL TRACTIVE STRESS FOR COHESIVE BED MATERIALS				
Voids Ratio	2.0-1.2	1.2-0.6	0.6-0.3	0.3-0.2
Dry Bulk Density (kg/m^3)	880-1220	1200-1650	1650-2030	2030-2210
Saturated Bulk Density (kg/m^3)	1550-1740	1740-2030	2030-2270	2270-2370
Type of Soil	**Critical Tractive Stress N/m^2**			
Sandy Clay	1.9	7.5	15.7	30.2
Heavy Clay	1.5	6.7	14.6	27.0
Clay	1.2	5.9	13.5	25.4
Loam Clay	1.0	4.6	10.2	16.8

1.5.3.2 Contraction Scour

The degree of contraction can be considered in terms of the contraction ratio Figure 1.5.2 and equilibrium contraction scour depth is given as

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$$y_{se} = y_2 - y_1$$

$$\beta = \frac{w_1}{w_2}$$

y_{se} =depth of scour due to contraction of channel

y_2, w_2 = Flow depth and channel width at contracted section

y_1, w_1 = flow depth and channel width at natural channel (un-contracted)

β = degree of contraction

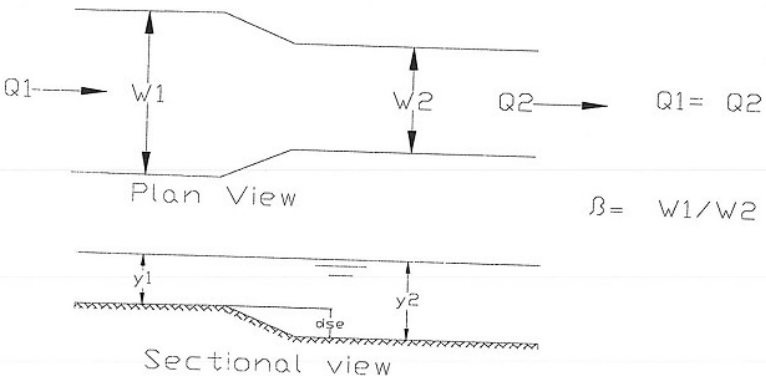

Figure 9. Hydraulics of uniform flow through long rectangular contraction

a. Parker method for gravel bed River in live bed condition

$$y_s = y_1 * \beta^{k_1} - y_1$$

Where y_s = contracted scour depth

β = degree of contraction (w_1/w_2)

k_1 = constant (ranges 0.675 to 0.825)

y_1 = depth of flood in uncontract condition

b. Gill (1981) Method

For the threshold condition in approach channel ($\frac{\tau_c}{\tau_1} = 1$),

$$\frac{y_2}{y_1} = \beta^{6/7}$$

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For the live bed condition in approach channel ($\frac{\tau_c}{\tau_1} < 1$) with high rate of sediment flow

$$\frac{y_2}{y_1} = \beta^{9/14}$$

For the clear water condition in approach channel ($\frac{\tau_c}{\tau_1} > 1$) with

$$\frac{y_2}{y_1} = \beta^{9/14}\left(\frac{\tau_c}{\tau_1}\right)^{-3/7}$$

c. Laursen (1958) Method

For Live bed Scour stream having main channel and flood channel

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_{1m}}\right)^{6/7} \beta^{k1}\left(\frac{n_2}{n_1}\right)^{k_2}$$

Where,

τ_c	=critical bed shear stress
τ_1	= bed shear stress in the approach reach
Q_{1m},	= discharge in upstream channel transporting sediment,
Q_2	=discharge in contracted channel
n_1, n_2	= Manning's Roughness coefficient at natural channel and contracted channel

For Live bed Scour stream having w_1=w_2 and n_1 =n_2

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_{1m}}\right)^{6/7}$$

For Live bed Scour stream having Q_2=Q_{1m}, contraction narrows the main channel and n_1 =n_2

$$\frac{y_2}{y_1} = (\beta)^{k_1}$$

For Clear water scour

$$\frac{y_2}{y_1} = (\beta)^{6/7}\left[\frac{{v_1}^2}{36{y_1}^{1/3}{d_{50}}^{2/3}}\right]^{3/7}$$

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Where, values of coefficients are given in Table 1.5.4.

Table 7. Values of coefficients K1, K2, K3

u_*/w	k_1	k_2	k_3	Mode of bed material transport
<0.50	0.59	0.066	0.25	Mostly contact bed material
0.50 – 2.0	0.64	0.21	0.21	Some suspended bed material discharge
>2.0	0.69	0.37	0.16	Mostly suspended bed material discharge

1.5.3.3 Local Scour

(a) Piers scour

Scour caused by flowing water of a stream past a bridge pier has been recognized as a major cause of bridge failure. Following are the significant points relating to parameters affecting the Pier scour depth:

(i) The greater the approach velocity greater will be the scour depth.

(ii) Similarly, scour depth is directly related to the positive power of the depth of flow.

(iii) The scour depth increases with an increase in the width of the pier.

(iv) If the approach fl ow is at an angle to the pier the projected length of the pier will come in to play and the scour increases.

(v) Shape of the pier plays an important role on the scour depth.

(vi) Size and gradation of the bed material do not have an important role on the scour depth; the time taken to achieve maximum scour depth depends on the grain size.

(vii) Lodging of debris around a pier can increase the width of pier, change its shape and its projected length with consequent increase in the maximum depth of scour

i. CSU equation

$$\frac{y_s}{y_1} = 2.0\, k_1 k_2 k_3 \left[\frac{a}{y_1}\right]^{0.65} F_{r1}{}^{0.43}$$

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L = Pier length, m

a = Pier width, m

F_{r1} = Froude Number directly upstream of the pier = $V_1/(gy_1)^{0.5}$

V_1 = Mean velocity of flow directly upstream of the pier, m/s

g = Acceleration of gravity (9.81 m/s2)

K_1 = Correction for pier nose shape

K_2 = Correction for angle of attack of flow from Table 3.5 or the equation below.

$$k_2 = \left(\cos\theta + \frac{L}{a}\sin\theta\right)^{0.65}$$

K_3 = Correction factor for bed condition

According to Federal Highway Administration (FHWA), values for correction factors are given Table 1.5.5, Table 1.5.6 and Table 1.5.7.

Table 8. Correction factor for pier nose (K1)

Shape of Pier nose	K_1
Square nose	1.1
Round nose	1.0
Circular cylinders	1.0
Group of cylinders	1.0
Sharp nose	0.9

Table 9. Correction for angle of attack of flow (K2)

Angle	L/a = 4	L/a = 8	L/a = 12
0	1.0	1.0	1.0
15	1.5	2.0	2.5
30	2.0	2.75	3.5
45	2.3	3.3	4.3
90	2.5	3.9	5.0

Table 10. Correction factor for bed condition (K3)

Bed Condition	Dune Height	K3
Clear-Water Scour	N/A	1.1

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Plane bed and Anti dune flow	N/A	1.1
small dunes	3 >H>0.6	1.1
medium dunes	9>H>3	1.2 to 1.1
large dunes	H>9	1.3

ii. Neil (1977) Method

$d_s = K_s\, b$

Where,

d_s = pier scour depth

b = diameter (width) of pier

K_s = coefficients,

1.5 for round and circular nose piers and

2.0 for rectangular piers

iii. Coleman (1971) Method

$$d_s = 0.54 * b \left(\frac{y}{b}\right)^{0.19} F_r^{1.19} y^{0.41}$$

Where,

d_s = pier scour depth

b = diameter (width) of pier

F_r = Froude Number

y = flow depth

iv. Jain and Fischer (1980) Method

Jain and Fisher as cited by Johnson developed a set of equation based on laboratory experiments

When $Fr - Fr_c > 0.2$

$$d_s = 1.86 * b \left(\frac{y}{b}\right)^{0.5} (F_r - F_{rc})^{1/4}$$

When $Fr - Fr_c < 0$

$$d_s = 1.85 * b \left(\frac{y}{b}\right)^{0.5} Fr_c^{1/3}$$

Where,

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d_s = pier scour depth

b = diameter (width) of pier

F_r = Froude Number

= $v/(gy)^{1/2}$

F_{rc} = Froude Number at contraction

= $v_c/(gy)^{1/2}$

y = flow depth

(b) Abutments Scour

Abutment scour can be computed using the modified HIRE (Richardson, 1990/FHWA 2001) or Froehlich (1989) equations. The input and data required to undertake the calculation is as follows:

(i) HIRE abutment scour equation (FHWA 2001)

$$\frac{y_s}{y_1} = 4.0{F_{r1}}^{0.33}\left[\frac{k_1}{0.55}\right]^{0.65} K_2$$

The modified HIRE equation is applicable when the ratio of projected abutment length (L) to the flow depth (y_1) is greater than 25.

(ii) Froehlich's (TRB 1989) abutments scour equation

$$\frac{y_s}{y_1} = 2.27K_1K_2\left[\frac{L'}{y_a}\right]^{0.43}{F_{r1}}^{0.61}K_2$$

L' = Length of abutment projected normal to flow, m

A_e = Flow area of the approach cross-section obstructed by the embankment, m2

F_{r1} = Froude Number of approach flow adjacent to and upstream of the abutment = $V_e / (gy_a)^{1/2}$; (Froehlich 1989, HIRE 1990)

V_e =Velocity of approach flow, = Qe/Ae; m/s

Q_e = Flow obstructed by the abutment and approach embankment, m3/s

y_1 = Average depth of flow on the flood plain or at the abutment on the overbank or in the main channel, m

y_s = Scour depth, m

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Table 11. Coefficient (K1) for abutment shape

Description	K_1
Vertical wall abutment	1.00
Vertical wall abutment with wing walls	0.82
Spill through abutment	0.55

(iii) Gill (1970) Method

$$d_s = 8.375 * L * \left(\frac{y}{L}\right)^{0.75} * \beta^{\frac{6}{7}} - y$$

Where,

L = length of abutment

y = depth of flow

1.5.3.4 Total Scour

(a) The maximum depth of scour below the Highest Flood Level (HFL) for the design of piers and abutments having individual foundations without any floor protection may be considered as follows.

(i) For piers - 2.0 Y_2

(ii) For abutments - (a) 1.27 $Y_2$with approach retained or lowest bed level whichever is deeper. (For average conditions on a straight reach of the stream)

(b) 2.00 Y_2 with scour all around. (For bad sites on curves or where diagonal current exist)

Where, Y_2 is the average depth of general scour measured from the water surface, in metres;

(b) Sum of General scour, contraction scour and local scour

(c) Melville (1997) method for total scour is given as

$$y_s = k_{yb} K_i K_d K_s K_\theta K_t K_G$$

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i. Flow shallowness – foundation size factor k_{yb}

For piers,

$k_{yb} = 2.4b$ $\quad \frac{b}{y} < 0.7$

$k_{yb} = 2\sqrt{yb}$ $\quad 0.7 < \frac{b}{y} < 5$

$k_{yb} = 4.5y$ $\quad \frac{b}{y} > 5$

Where,

b = width of pier

y = depth of flood

For Abutments,

$k_{yL} = 2.4b$ $\quad \frac{L}{y} < 1$

$k_{yL} = 2\sqrt{yb}$ $\quad 1 < \frac{L}{y} < 25$

$k_{yL} = 4.5y$ $\quad \frac{L}{y} > 25$

Where,

b = width of abutment

y = depth of flood

ii. Flow intensity factor K_i

$k_i = \frac{v-(v_a-v_c)}{v_c}$ $\quad \frac{v-(v_a-v_c)}{v_c} < 1$

$k_i = 1$ $\quad \frac{v-(v_a-v_c)}{v_c} = 1$

Where,

v_c = Critical velocity of flow

v_a = Approach velocity

v = velocity of flow

iii. Sediment size factor K_d

$k_d = 0.57 \log (2.24 \frac{b}{d_{50}})$ $\quad \frac{b}{d_{50}} \leq 25$

$k_d = 1$ $\quad \frac{b}{d_{50}} > 25$

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iv. Foundation shape factor Ks

Table 12. Shape factors for uniform piers and shorter abutments

Foundation type	Shape	Ks
Uniform Pier	Circular	1.0
	Round Nosed	1.0
	Squared Nosed	1.1
	Sharped Nosed	0.9
Abutment	Vertical wall	1.0
	Wing wall	0.75
	Spill through 0.5:1 (H:V)	0.6
	Spill through 1:1 (H:V)	0.5
	Spill through 1.5:1 (H:V)	0.45

v. Time factor K_t

$$K_t = exp\left\{-0.03\left|\frac{v_c}{v} ln\left(\frac{t}{t_e}\right)\right|^{1.6}\right\}$$

v_c= Critical velocity

v= Velocity of flow

t_e = equilibrium time

vi. Approach channel geometry factor KG

$$K_G = \sqrt{1-\left(\frac{L^*}{L}\right)\left[1-\left(\frac{y^*}{y}\right)^{5/3}\left(\frac{n}{n^*}\right)\right]}$$

1.6 RIVER TRAINING WORKS

1.6.1 General

In alluvial stream systems, it is the rule rather than the exception that banks will erode; sediments will be deposited; and floodplains, islands and side channels will undergo modification with time. Alluvial channels continually change position and shape as a

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consequence of hydraulic forces exerted on the bed and banks. These changes may be gradual or rapid and may be the result of natural causes or man's activities.

A study of the plan and profile of a stream is necessary in understanding stream morphology. Plan view appearances of streams are varied and result from many interacting variables. Small changes in a variable can change the plan view and profile of a stream, adversely affecting a bridge site or encroachment. This is particularly true for alluvial streams. Conversely, a crossing or encroachment can inadvertently change a variable, adversely affecting the stream.

If inundation, bank erosion, river channel shifting are pertinent at upstream of the bridge site, a walk through survey and study of morphological parameters of river stretch in the vicinity shall be carried out. River training requirements far beyond 100m upstream of bridge site and its planning and design shall be carried out using guideline prepared by Government of Nepal or as per respective IRC codes.

1.6.2 Design of Guide bund

When embankments encroach on wide floodplains, the-flows from these areas must flow parallel to the approach embankment to the bridge opening. These flows can erode the approach embankment. A severe flow contraction at the abutment can reduce the effective bridge opening, which could possibly increase the severity of abutment and pier scour. Guide banks (also referred to as spur dikes) can be used in these cases to prevent erosion of the approach embankments by cutting off the flow adjacent to the embankment, guiding stream flow through a bridge opening, and transferring scour away from abutments to prevent damage caused by abutment scour. The two major enhancements guide banks bring to bridge design are (1) reduce the separation of flow at the upstream abutment face and thereby maximize the use of the total bridge waterway area and (2) reduce the abutment scour due to lessening turbulence at the abutment face. Guide banks can be used on both sand- and gravel-bed streams.

1.6.2.1 **Design Considerations**

Principal factors to be considered when designing guide banks, are their orientation to the bridge opening, plan shape, upstream and downstream length, cross-sectional shape, and the crest elevation.

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Figure 1.6.1 below presents a typical guide bank plan view. It is apparent from the figure that without this guide bank overbank flows would return to the channel at the bridge opening, which can increase the severity of contraction and scour at the abutment. Note, that with installation of guide banks the scour holes which normally would occur at the abutments of the bridge are moved upstream away from the abutments. Guide banks maybe designed at each abutment, as shown, or singly, depending on the amount of overbank or floodplain flow directed to the bridge by each approach embankment.

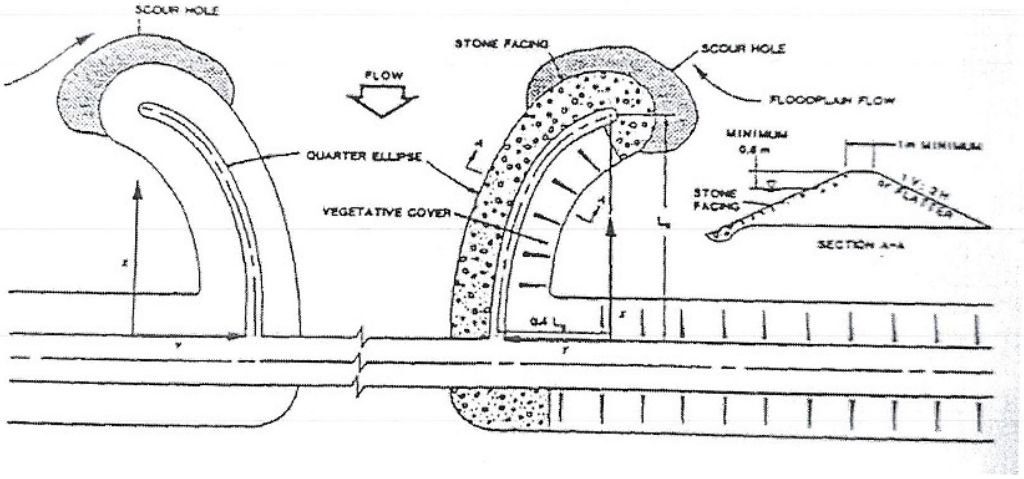

Figure 10. Typical Guide Bank

1.6.2 Orientation

Guide banks should start at and be not parallel to the abutment and extend upstream from the bridge opening. The distance between the guide banks at the bridge opening should be equal to the distance between bridge abutments. Best results are obtained by using guide banks with a plan form shape in the form of a quarter of an ellipse, with the ratio of the major axis (length L_s) to the minor axis (offset) of 2.5: 1. This allows for a gradual constriction of the flow. Thus, if the length of the guide bank measured perpendicularly from the approach embankment to the upstream nose of the guide bank is denoted as Ls, the amount of expansion of each guide bank (offset), measured from the abutment parallel to the approach roadway, should be 0.4 L_s.

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The plan view orientation can be determined using equation of an ellipse with origin at the nose of the guide bank. For this equation, X is the distance measured perpendicularly from the bridge approach and Y is the offset measured parallel to the approach embankment, as shown on figure 1.6.1

It is important that the face of the guide bank match the abutment so that the flow is not

$$\frac{x^2}{L_s^2}+\frac{Y^2}{(0.4L_s)^2}=1$$

disturbed where the guide bank meets the abutment. For new bridge construction, abutments can be sloped to the channel bed at the same angle as the guide bank. For retrofitting existing bridges modification of the abutments or wing walls may be necessary.

1.6.3 Length

For design of guide banks, the length of the guide bank, Ls must first be determined. This can be easily determined using a nomograph which was developed from laboratory tests performed at Colorado State University and from field data compiled by the USGS. For design purposes the utilization of the nomograph involves the following parameters:

Q	= Total discharge of the stream, m^3/s
Q_f	= Discharge intercepted by embankments on either flood plains, m^3/s
Q_{30m}	= Discharge in 30 m of stream adjacent to the abutment, m^3/s
b	= Length of the bridge opening, m
A_{n2}	= Cross-sectional flow area at the bridge opening at normal stage, m^2
V_{n2}	= Q/A_{n2} = Average velocity through the bridge opening, m/s
Q_f/Q_{30m}	= Guide bank discharge ratio
L_s	= Projected length of guide bank, m

A nomograph is presented in figure1.6.2 to determine the projected length of guide banks. This nomograph should be used to determine the guide bank length for designs greater than 15 m and less than 75 m. If the nomograph indicates the length required to be greater than 75 m the design should be set at 75 m. It is recommended that the minimum length of guide banks be 15 m. An example of how to use this nomograph is presented below.

FHWA practice has shown that many guide banks have performed well using a standardized length of 46 m. Based on this experience, guide banks of 46 m in length should perform

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very well in most locations. Even shorter guide banks have been successful if the guide bank intersects the tree line.

1.6.4 Crest Height

As with deflection spurs, guide banks should be designed so that they will not be overtopped at the design discharge. If this were allowed to occur, unpredictable cross flows and eddies might be generated, which could scour and undermine abutments and piers. In general, a minimum of 0.6 m of freeboard, above the design water surface elevation should be maintained.

1.6.5 SHAPE and Size

The cross-sectional shape and size of guide banks should be similar to deflector, or deflector/retarder spurs discussed previously. Generally, the top width is 3 to 4 m, but the minimum width is 1 m when construction is by drag line. The upstream end of the guide bank should be round nosed. Side slopes should be lV:2H or less.

1.6.6 Downstream Extent

Guide banks are extended in some cases downstream of the abutments to minimize scour due to rapid expansion of the flow at the downstream end of the abutments. These downstream guide banks are sometimes called "heels." If the expansion of the flow is too abrupt, a shorter guide bank, which usually is less than 15 m long, can be used downstream. Downstream guide banks should also start at and start parallel to the abutment and the distance between them should enlarge as the distance from the abutment of the bridge increases.

In general, downstream guide banks are a shorter version of the upstream guide banks. Riprap protection, crest height and width should be designed in the same manner as for upstream guide banks.

1.6.7 Rip Rap

Guide banks are constructed by forming an embankment of soil or sand extending upstream from the abutment of the bridge. To inhibit erosion of the embankment materials, guide banks must be adequately protected with riprap or stone facing. Rock riprap should be placed on the stream side face as well as around the end of the guide bank. It is not necessary to riprap the side of the guide bank adjacent to the highway approach embankment. As in the case of spurs, a gravel, sand, or fabric filter may be required to protect the underlying

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embankment material. Riprap should be extended below the bed elevation extend up the face of the guide bank to 0.6 m above the design flow. Additional riprap should be placed around the upstream end of the guide bank so to protect the embankment from scour.

As in the case of spurs, it is important to adequately tie guide banks into the approach embankment. Therefore, for skewed crossings, the length of guide banks should be set using the nomograph for the side of the bridge crossing which yields the largest guide bank length.

1.6.8 OTHER Design Concerns

In some cases, where the cost of stone riprap facing is prohibitive, the guide bank can be covered with sod or other minimal protection. If this approach is selected, the design should allow for and stipulate the repair or replacement of the guide bank after each high water occurrence. Other measures which will minimize damage to approach embankments, and guide banks during high water are:

- Keep trees as close to the toe of guide bank embankments as construction will permit. Trees will increase the resistance to flow near and around the toe of the embankment, thus reducing velocities and scour potential.
- Do not allow the cutting of channels or the digging of borrow pits along the upstream side of approach embankments and near guide banks. Such practices encourage flow concentration and increases velocities and erosion rates of the embankments.
- In some cases, the area behind the guide bank may be too low to drain properly after a period of flooding. This can be a problem, especially when the guide bank is relatively impervious. Small drain pipes can be installed in the guide bank to drain this ponded water.
- In some cases, only one approach will cut off the overbank flow. This is common when one of the banks is high and well defined. In these cases, only one guide bank may be necessary.

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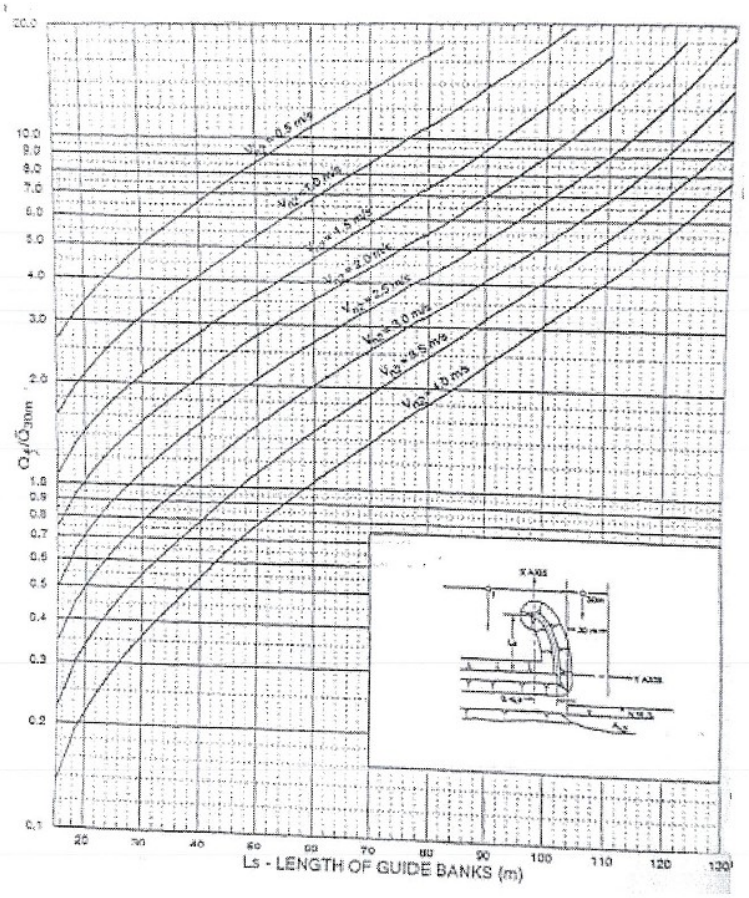

Figure 11. Bradley's curve

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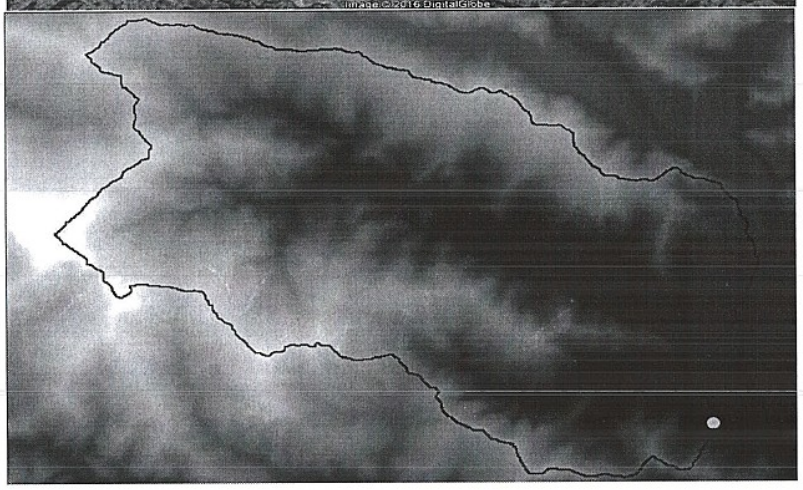
ANNEX

ANNEX

Figure 1: Catchment Area delineation

Google Earth Area= 156 km^2	
DEM (30m) Area= 157.8 km^2	

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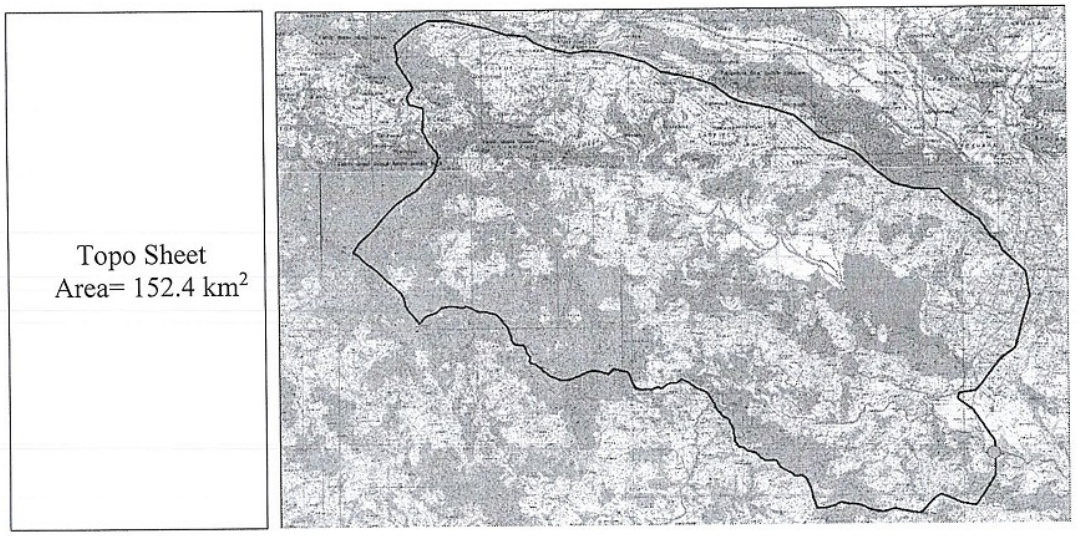

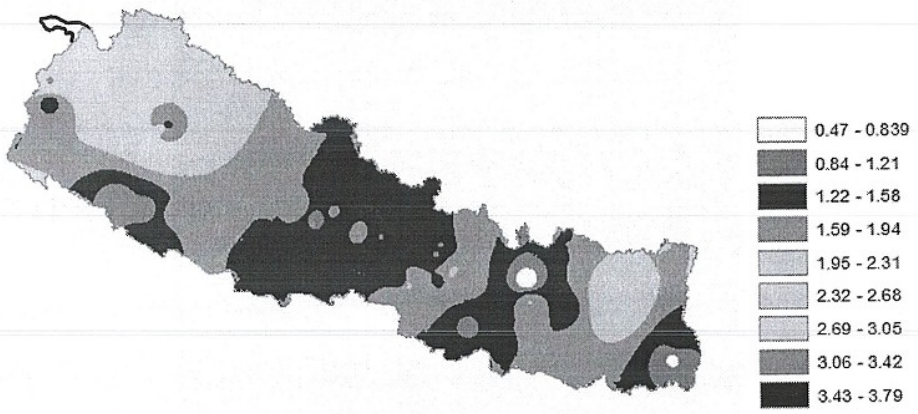

Figure 2: Spatial distribution of C_t

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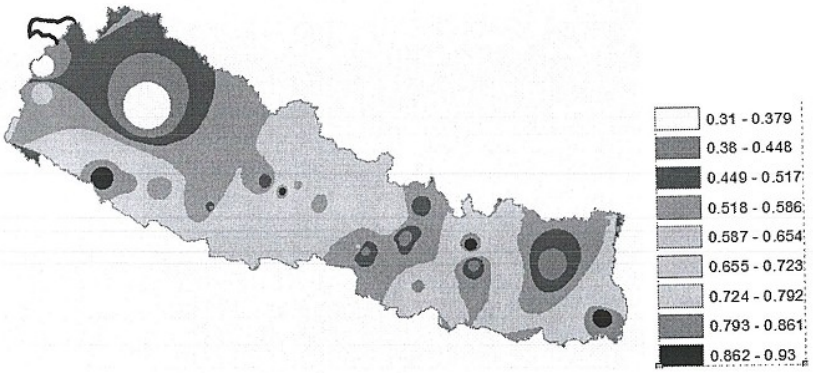

Figure 3: Spatial distribution of C_p

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Table 1: WECS Method

A	Catchment Area in sq.km	156.91	km^2
Q_2	$Q_2 = 1.8767\ (A+1)^{0.8783}$	160.05	m^3/sec
Q_{100}	$Q_{100} = 14.63\ (A+1)^{0.7342}$	601.61	m^3/sec
□	□ = $\ln(Q_{100}/Q_2)/2.326$	0.56927	

Variable	Formula	units	2yrs	5yrs	10yrs	20yrs	50yrs	100yrs	200yrs
s			0	0.842	1.282	1.645	2.054	2.326	2.576
Q_R	$Q_R = \exp(\ln Q_2 + s\square)$	m^3/sec	160.05	258.48	332.05	408.27	515.31	601.61	693.62

Table 2: Modified Dicken's Method

A	Catchment Area in sq.km	156.91	km^2
As	Snow Covered area out of total catchment area A	0	km^2
p	$p = 100(A_S + 6)/A$	3.82	

Variable	Formula	units	2yrs	5yrs	10yrs	20yrs	50yrs	100yrs	200yrs
C_T	$C_T = 2.342 \log(0.6T)\log(1185/p)+4$	-	4.462	6.784	8.540	10.296	12.618	14.375	16.131
Q_T	$Q_T = C_T A^{3/4}$	m^3/sec	197.82	300.75	378.62	456.48	559.42	637.28	715.15

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Table 3: Snyder's Method

A	Catchment Area in sq.km	156.91	km2
L	Catchment Length in km	27.72	km
Lc	Distance from the basin outlet to a point on stream nearest to the centroid of basin in Km.	14.47	km
C_t	Coefficient	1.9	
C_p	Coefficient	0.62	
C_A	Coefficient	1.135	
t_{pr}	$t_{pr} = 0.75\ C_t(L*L_c)^{0.3}$	8.60584	
t_r	$t_r = t_{pr} /5.5$	1.56470	
t_{pR}	$t_{pR} = t_{pr} + 0.25\ (t_R - t_r)$	14.2147	
q_{pR}	$q_{pR} = 2.78\ C_p/\ t_{pR}$	0.12126	

Variable	Formula	units	2yrs	5yrs	10yrs	20yrs	50yrs	100yrs	200yrs
R	24 hr rainfall in cm	cm	0.00	0.00	0.00	0.00	0.00	30.48	0.00
Q_{pR}	$Q_{pR} = q_{pR} * (C_A * A) * R$	m3/s	0.00	0.00	0.00	0.00	0.00	658.29	0.00

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Table 4: BD Richard Method

a	Catchment Area in sq.km	156.91	km2
S	Catchment Slope	4.3362	
L	Catchment Length in km	27.72	km
R_T	24 hour rainfall for T in(mm)		

Variable	Formula	Units	2yrs	5yrs	10yrs	20yrs	50yrs	100yrs	200yrs
R_T	24 hour rainfall for T in(mm)	mm	0.00	0.00	0.00	0.00	0.00	304.84	0.00
T_c	Assume	-	5.83	5.83	5.83	5.83	5.83	5.83	5.83
F	F = 1.09352-0.06628 ln (a)	-	0.76	0.76	0.76	0.76	0.76	0.76	0.76
D	$D = 1.102L^2/(S.F)$	-	257.48	257.48	257.48	257.48	257.48	257.48	257.48
R_{TC}	$R_{TC} = R_T \times 22.127\ T_c^{0.476577}/100$	-	0.00	0.00	0.00	0.00	0.00	156.25	0.00
I	$I = R_{TC} / T_C$	-	0.00	0.00	0.00	0.00	0.00	26.81	0.00
K_R	$K_R = 0.651\ (T_C + 1)$	-	4.45	4.45	4.45	4.45	4.45	4.45	4.45
C_{KR}	$C_{KR} = 0.95632/ K_R^{1.4806}$	-	0.11	0.11	0.11	0.11	0.11	0.11	0.11
T_{C3}	$T_{C3} = D * C_{KR}$	-	27.05	27.05	27.05	27.05	27.05	27.05	27.05
T_{C2}	$T_{C2} = (TC3/0.585378)^{1/2.17608}$	-	5.83	5.83	5.83	5.83	5.83	5.83	5.83
% Diff	(T_C2-T_C)/T_C	-	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%	0.00%
Remark	If(% Diff < 5,Ok, Not Ok)	-	ok	ok	ok	ok	ok	ok	ok
Q_R	$Q_R = 0.222*A*I*F$	m^3/sec	0.00	0.00	0.00	0.00	0.00	708.30	0.00

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Table 5: Downstream of bridge section

Stn	Dist	Flow (Q)	Water RL	Bed RL	Area(A)			Wetted Perimeter(P)			Mannings coefficient(n)		
					Lob	Ch	Rob	Lob	Ch	Rob	Lob	Ch	Rob
1	2	3	4	5	6	7	8	9	10	11	12	13	14
0	0	752	666.57	662.29	54.97	234.25		42.75	67.33		0.05	0.04	0.05
50	50	752	666.86	662.47		341.28			118.13		0.05	0.04	0.05
100	100	752	667.03	662.59		428.83			133.45		0.05	0.04	0.05
150	150	752	667.07	662.79		413.73			142.09		0.05	0.04	0.05

Type of Transition	Head check		Depth	Hydraulic radius (R)			Conveyance (K)			A_t	Area Change (%)	Transition type	C/E
	$\|H_1-H_2\|$	status		Lob	Ch	Rob	Lob	Ch	Rob				
15	16	17	18	19	20	21	22	23	24	25	26	27	28
			4.28	1.286	3.479	0.000	1300.02	13446.22	0.00	289.22			
	ok	0.005	4.39	0.000	2.889	0.000	0.00	17306.85	0.00	341.28	18.00	G	C
	ok	-0.003	4.44	0.000	3.213	0.000	0.00	23345.48	0.00	428.83	25.65	G	C
	ok	-0.010	4.28	0.000	2.912	0.000	0.00	21090.78	0.00	413.73	3.52	G	E

Transition	K_t	α	β	Velocity mean	$\alpha * v^2/2g$	H_1	S_f	Δx	$S_{f\,mean}$	H_f			H_2
										friction	C/E_coeff	exp_con	
29	30	31	32	33	34	35	36	37	38	39	40	41	42
	14746.24	1.17		2.60	0.405	666.97	0.00260	0					
G	17306.85	1		2.20	0.247	667.11	0.00189	50	0.002244	0.11221	0.10	0.02	667.10
G	23345.48	1		1.75	0.157	667.19	0.00104	50	0.001463	0.07314	0.10	0.01	667.19
G	21090.78	1		1.82	0.168	667.24	0.00127	50	0.001154	0.05772	0.30	0.00	667.25

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Table 6: Near the Bridge section

Stn	Dist	Flow (Q)	Water RL	Bed RL	Area(A)			Wetted Perimeter(P)			Mannings coefficient(n)		
					Lob	Ch	Rob	Lob	Ch	Rob	Lob	Ch	Rob
1	2	3	4	5	6	7	8	9	10	11	12	13	14
150	150	752	667.07	662.79		413.73			142.09		0.05	0.04	0.05
200	200	752	667.09	662.91		330.26			121.43		0.05	0.04	0.05
BD	203	752	667.07	662.91		308.57			118.09		0.05	0.04	0.05
BU	210	752	667.07	662.91		310.86			118.21		0.05	0.04	0.05
220	220	752	667.17	662.91		339.89			121.87		0.05	0.04	0.05
250	250	752	667.19	663.02		299.94			123.3		0.05	0.04	0.05

Type of Transition	Head check		Depth	Hydraulic radius (R)			Conveyance (K)			A_t	Area Change (%)	Transition type	C/E
	$\lvert H_1-H_2 \rvert$	status		Lob	Ch	Rob	Lob	Ch	Rob				
15	16	17	18	19	20	21	22	23	24	25	26	27	28
			4.28	0.000	2.912	0.000	0.00	21090.78	0.00	413.73			
	ok	0.00	4.18	0.000	2.720	0.000	0.00	16087.28	0.00	330.26	20.17	G	E
	ok	0.00	4.16	0.000	2.613	0.000	0.00	14634.82	0.00	308.57	6.57	G	E
	ok	-0.02	4.16	0.000	2.630	0.000	0.00	14806.26	0.00	310.86	0.74	G	C
	ok	0.02	4.26	0.000	2.789	0.000	0.00	16836.02	0.00	339.89	9.34	G	C
	ok	0.00	4.17	0.000	2.433	0.000	0.00	13562.98	0.00	299.94	11.75	G	E

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Transition	K_t	α	β	Velocity mean	$\alpha * v^2/2g$	H_1	S_f	Δx	$S_{f\ mean}$	H_f			H_2
										friction	C/E_coeff	exp_con	
29	30	31	32	33	34	35	36	37	38	39	40	41	42
	21090.78	1.00	1	1.82	0.1684	667.24	0.001271	0					
G	16087.28	1.00		2.28	0.2643	667.35	0.002185	50	0.00173	0.08334	0.3	0.03	667.35
G	14634.82	1.00		2.44	0.3027	667.37	0.002640	3	0.00241	0.00721	0.3	0.01	667.37
G	14806.26	1.00		2.42	0.2983	667.37	0.002580	7	0.00261	0.01827	0.1	0.00	667.39
G	16836.02	1.00		2.21	0.2495	667.42	0.001995	10	0.00229	0.02269	0.1	0.00	667.4
G	13562.98	1.00	1	2.51	0.3204	667.51	0.003074	30	0.00253	0.0743	0.3	0.02	667.52

Table 7: Upstream of Bridge Section

Stn	Dist	Flow (Q)	Water RL	Bed RL	Area(A)			Wetted Perimeter(P)			Mannings coefficient(n)		
					Lob	Ch	Rob	Lob	Ch	Rob	Lob	Ch	Rob
1	2	3	4	5	6	7	8	9	10	11	12	13	14
250	250	752	667.19	663.02		299.94			123.3		0.05	0.04	0.05
300	300	752	667.33	663.13		266.62			132.95		0.05	0.04	0.05
350	350	752	667.68	663.18		322.76			171.12		0.05	0.04	0.05
400	400	752	667.89	663.25		363.86			153.11		0.05	0.04	0.05
450	450	752	668.02	663.32	0.37	377.68		1.22	193.26		0.05	0.04	0.05
500	500	752	668.10	663.4	0.09	289.8		0.59	156.98		0.05	0.04	0.05
550	550	752	668.36	663.53	0.57	301.06		1.52	164.95		0.05	0.04	0.05
600	600	752	668.64	663.77		426.5	0.02		152.36	0.44	0.05	0.04	0.05
650	650	752	668.70	664		432.64			141.32		0.05	0.04	0.05

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Director General

Type of Transition	Head check		Depth	Hydraulic radius (R)			Conveyance (K)			A_t	Area Change (%)	Transition type	C/E
	$\lvert H_1-H_2 \rvert$	status		Lob	Ch	Rob	Lob	Ch	Rob				
15	16	17	18	19	20	21	22	23	24	25	26	27	28
			4.17	0.000	2.433	0.000	0.00	13562.98	0.00	299.94			
	ok	-0.003	4.20	0.000	2.005	0.000	0.00	10599.91	0.00	266.62	11.11	G	E
	ok	-0.011	4.50	0.000	1.886	0.000	0.00	12317.97	0.00	322.76	21.06	G	C
	ok	-0.002	4.64	0.000	2.376	0.000	0.00	16199.25	0.00	363.86	12.73	G	C
	ok	-0.006	4.70	0.303	1.954	0.000	3.34	14758.83	0.00	378.05	3.90	G	C
	ok	-0.005	4.70	0.153	1.846	0.000	0.51	10902.88	0.00	289.89	23.32	G	E
	ok	0.001	4.83	0.375	1.825	0.000	5.93	11240.71	0.00	301.63	4.05	G	C
	ok	-0.038	4.87	0.000	2.799	0.045	0.00	21178.30	0.05	426.52	41.41	G	C
	ok	-0.004	4.70	0.000	3.061	0.000	0.00	22804.23	0.00	432.64	1.43	G	C

Transition	K_t	α	β	Velocity mean	$\alpha * v^2/2g$	H_1	S_f	Δx	$S_{f\ mean}$	H_f			H_2
										friction	C/E_coeff	exp_con	
29	30	31	32	33	34	35	36	37	38	39	40	41	42
	13562.98	1.000		2.51	0.320	667.51	0.003074	0					
G	10599.91	1.000		2.82	0.405	667.74	0.005033	50	0.004054	0.20268	0.30	0.03	667.74
G	12317.97	1.000		2.33	0.277	667.96	0.003727	50	0.004380	0.21900	0.10	0.01	667.97
G	16199.25	1.000		2.07	0.218	668.11	0.002155	50	0.002941	0.14705	0.10	0.01	668.11
G	14762.17	1.001		1.99	0.202	668.22	0.002595	50	0.002375	0.11875	0.10	0.00	668.23
G	10903.39	1.000		2.59	0.343	668.44	0.004757	50	0.003676	0.18379	0.30	0.04	668.45
G	11246.63	1.002		2.49	0.318	668.68	0.004471	50	0.004614	0.23069	0.10	0.00	668.68
G	21178.35	1.000		1.76	0.158	668.80	0.001261	50	0.002866	0.14329	0.10	0.02	668.84
G	22804.23	1.000		1.74	0.154	668.85	0.001087	50	0.001174	0.05871	0.10	0.00	668.86

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Director General

Table 8: Total scour Depth

Input Data for scour Calculation

Design Discharge	Q	752	m^3/s
Span of the bridge	S	3 × 35	m
Top width	T	105	m
Flow Area	A	310.86	m2
Depth of Flow	y_1	4.18	m
Design Velocity	v_1	2.42	m/s
Specific discharge	q	10.12	m^3 /s /m
D_{50} , mm	D_{50} , mm	2.2	mm
D_{90}, mm	D_{90} , mm	12	mm
Low Flood level	LFL	663.56	m
High Flood Level (HFL) at Bridge	HFL	667.10	m

Description	Combination	Total Scour
Using 1.5.3.4 (a)	L	9.17
	B	10.08
Using 1.5.3.4 (b)	L+C+ CSU	7.39
	B+ C + CSU	7.84
	L +C+ COL	5.63
	B +C+ COL	6.08
	L +C+ J&F	6.44
	B +C+ J&F	6.89
	L +C+ LU	10.48
	B +C+ LU	10.93
	CHR+C+CSU	9.91
	NZR	9.71
Using 1.5.3.4 (b)	Melville	7.57

Method	***Abbreviation***	***Method***	***Abbreviation***
Lacey 1930	*L*	*Component velocity approach*	*CVA*
Blench 1969	*B*	*Colemen*	*COL*
contraction scour by Parker	*C*	*Jain and Fischer*	*J&F*
New Zealand Railway	*NZR*	*Melville*	*Melville*
California State University	*CSU*	*Charlton(1982)*	*CHR*

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