DETERMINATION OF WATERWAY UNDER A BRIDGE IN HIMALAYAN REGION - SOME CASE STUDIES

S.K. MAZUMDER
Individual Consultant
(Former AICTE Emeritus Professor, Delhi College of Engg.
& Dean, Faculty of Technology, University of Delhi)
E-mail: somendrak64@rediffmail.com

ABSTRACT

Determination of waterway under a bridge requires proper investigation and data collection at site for economy, efficiency and safety. Waterway is governed principally by the design peak flood discharge which can be computed by several methods as recommended in IRC/IS/RDSO codes/guidelines. Limitations of different methods of flood estimation have been discussed. Waterway is dependent also on the type of terrain through which river passes near the bridge site. Although Lacey's waterway acts as a guideline, actual waterway to be provided under a bridge may substantially differ from Lacey's waterway, depending on the terrain condition. Procedure for computing waterway for bridges in mountainous, trough, meandering and deltaic terrains have been narrated. For a new bridge, designer has freedom to provide waterway as required. But in case a bridge is to be widened, designer is sometimes compelled to provide waterway same as in the existing one, unless the existing bridge is to be dismantled. It is possible to reduce afflux by providing suitable transitions connecting the existing bridge with the new one and retain an existing bridge with highly restricted waterway. Some case studies to illustrate computation of waterway for some new and existing bridges under different terrains in the Himalayan region are given at the end.

1. INTRODUCTION

A large numbers of bridges are being constructed all over India by the railways and roads authorities for better and faster communication and connectivity to the different parts of the country. Some of the roads and road bridges are new; but a large numbers of existing bridges are being widened from 2- lanes to 4-lanes. In the case of new bridges, designer has a freedom to provide adequate waterway required for smooth passage of design flood without causing any harmful afflux (IRC-5, 1998). In the case of widening of existing bridges, however, the designer is sometimes compelled to provide waterway under the new bridge same as in the existing one, unless the new road is far away from the existing one or it is decided to demolish the existing bridge due to its age and poor structural conditions. Low height submerged bridges/causeways are to be demolished in favor of all weather road. Usually, the gap between the new and the old bridge is kept same as the median width between the roads. The gap between the bridges is also decided by the existing well foundation as well as the new wells to be constructed for supporting the new bridge. In any case, this gap is usually inadequate for providing a suitable transition (Mazumder & Dhiman, 2003) to connect the existing short span bridge with the new one requiring substantial increase in waterway due to hydrologic/hydraulic requirement as per provisions made in IRC/BIS/RDSO codes.

When the existing bridge can not be dismantled because of prohibitive costs, often the hydrological/hydraulic aspects are over ruled and the waterway for the new bridges is kept the same as that of the existing ones. An argument often put forward by the bridge engineers in favor of retaining the existing waterway is that in spite of providing less waterway (as per hydrologic/hydraulic requirement), the existing bridge is not damaged/washed out. Usually, a bridge engineer is concerned more about the structural and foundation requirement than the hydrologic/hydraulic requirement. Inadequacy in structural or foundation design may lead to immediate collapse of the bridge. Inadequacy of hydrologic/hydraulic design, however, may not cause immediate failure since the design flood may not occur immediately after construction of the bridge. But there is always a risk of failure of the bridge due to inadequacy of waterway. Apart from risk of failure, there are several undesirable long term problems e.g. submergence of land, meandering and erosion of river banks, damage of properties in the adjoining areas, costly maintenance / protection / river training etc. throughout the life span of the bridge. Several adverse impact of over constricted bridges have been discussed in an earlier paper by the author (Mazumder et al., 2002)

Any unwise decision regarding waterway may lead to loss of river regime, instability of river, formation of hydraulic jump and degradation downstream, change in fluvial processes and many other unforeseen situations which can be studied and resolved by a designer with deep knowledge and insight in hydrologic/hydraulic/morphological sciences. Considering the uncertainty, risks and the long term costs of maintenance involved, it is almost an universal practice to design waterway of a bridge for a peak flood of 50 years return period under normal conditions and 100 years return period under exceptional circumstances. If the exiting waterway is insufficient, it results in high afflux, inadequate freeboard, overtopping of the road, aggradations (upstream) and degradation (downstream) and possible outflanking of the bridge. Costly river training measures and annual maintenance will be needed for the safety of the bridge and the adjoining approach road.

Computation of waterway under the bridges either for the new or for the existing ones to be widened has to be made very scientifically for their safety as well as economy. Underestimation of waterway may result in outflanking of a bridge and other problems discussed above. Overestimation of waterway, on the other hand, will not only increase the cost of the bridge, it will also provide an opportunity to the river to play in its meandering belt under the bridge causing non-uniformity of flow distribution which results in high scour under some of the spans and silting in some others.

Most of the rivers, especially those in the north and north-east of India, pass through a varieties of terrains e.g. hilly and mountainous, sub-hilly and trough, braided and meandering zones with wide flood plains, deltaic and tidal reaches etc. Fixing waterway for a bridge under different terrains requires an intimate knowledge of morphology, hydrology, hydraulics and river-mechanics of the rivers and their alluvial stream processes (Garde and Rangaraju, 2000). Any arbitrary decision regarding waterway under a bridge without considering its past history and behavior of the river in the near and far field may create unforeseen problems in future during the life span of the bridge.(Mazumder-2004). In this paper, an attempt has been made to discuss the different aspects of hydrologic/hydraulic considerations needed for determining waterway for a bridge under different terrain conditions – both for the new bridges and the existing bridges to be widened for accommodating more lanes. Some case studies for determination of waterway under the bridges in the Himalayan region are furnished with computations involved in each case.

2.0 INVESTIGATIONS NEEDED FOR DETERMINING WATERWAY

A number of routine investigations are to be carried out for determining the safe waterway for a bridge. These are narrated in IRC/BIS codes. They are briefly mentioned below:

2.1 Topographic Investigation:

Depending on the size of the catchment area, toposheets of either1:25,000 or 1:50,000 or 1:2,50,000 scales are used for finding the catchment area , terrain slope, river course and its tortuisity, land use, soil and cover conditions etc. When toposheets are not readily available due to classified nature of certain catchments (restricted areas), satellite imageries obtained from Google Earth can be used. Use of digital terrain maps by GPS/GIS are very useful to obtain topographic information and in visualizing terrain condition and the river systems.

2.2 Hydrologic Investigations:

Hydrologic investigations e.g. rainfall characteristics, stream flow characteristics, design flood and flood history of the river, dominant flow, stream forms and their tributaries, sediment characteristics, debris flow etc. are vitally needed for determining location and waterway for a bridge and to avoid future problems of failure and maintenance. Different types of hydrologic investigations are briefly discussed in the following paragraphs.

2.2.1 Peak Flood and High Flood Level (HFL)

Peak flood discharge and corresponding high flood level data are necessary for fixing the waterway and the deck level of a bridge. In the case of major bridges on large river, where gauging (stage-discharge) data are readily available for frequency analysis, design peak flood for a return period of 50 or 100 years is computed by Gumbel's method of probability analysis or similar other methods of frequency analysis. At least 15 to 20 consecutive years of annual peaks and the corresponding high flood levels are required.

In the case of medium and minor bridges or major bridges where gauging data are not available, peak flood is usually estimated by several indirect methods (IRC-SP: 13, 2004 & IRC-5, 1998). However, these methods of indirect determination of peak flood have some inherent drawbacks some of which are pointed out under the head estimation of design discharge.

2.2.2 Rainfall data

Where peak flood is to be estimated, rainfall data in the catchment is of vital importance. Depending upon the return period of peak flood, maximum probable rainfall of either 50 year or 100 year return period is found from frequency analysis of rainfall data. Since continuous recording type rain gauges are now installed in many parts of our country, it is desirable to use such rainfall records of different storm durations. for estimation of peak flood by rational formula (for small catchments only)

$$Q_p = 0.0278 \text{ f P } I_c \text{ A} \tag{1}$$

Where

Q_p = peak flood in cumec

P = permeability of the catchment area

F = spread factor

A = catchment area in hectare

I_c = rainfall intensity in cm/hr given in the IRC: SP-13 as

$$Ic = F/T \text{ [} (T+1)/ (t_c +1)] \tag{2}$$

Where

F = total rainfall in cm/hr

T = duration of total rainfall in hours

t_c = time of concentration in hours

Use of such empirical equation (eq.2) may result in considerable error (positive) when 24 hours (T) total rainfall (F) is used for computing peak design flood from a small catchment (upto 25 sq. km) having small concentration time (t_c) .It is desirable to use storm distribution curves from continuous rain gauge data available with Indian Meteorologica Department (IMD) and Central Water Commission (CWC.) and Ministry of railways (1990).

For medium catchments varying from 25 to 2500 square km , flood estimation reports for different regions in India - published by CWC in association with MOSHRT, IMD and RDSO.-- gives 24 hour rainfall of different return periods as well as rainfall intensities of different storm periods for different regions in India. The reports also give the procedure for finding peak flood by synthetic unit hydrograph method by use of rainfall data published in the report. They are reliable and scientific.Time of concentration (t_c) can be estimated either by IRC formula or by other improved formula (CBIP-1989) or by estimating time of flow from the hydrologically most remote point in the catchment.

.

2.2.3 Run off Data

Best method of finding design peak flood for determining waterway is to collect run-off data of the stream from gauging station near the bridge site. Unfortunately, however, gauging data are not available in all streams. Central water Commission, Govt. of India, has gauging sites only in major streams at selected locations as per their requirement and they publish the gauge-discharge data periodically. Stream gauges are installed by state Govt. also for finding run-off from streams only at such locations where some hydraulic structures are proposed to be constructed by the Govt. Even if run-off data are collected by the central or state govt., they are reluctant to part with them, especially to private consultants. A large number of streams and their catchments are classified. The toposheets needed for finding catchments areas and other information as well as the run-off data of these streams are not made available on the plea of secrecy (although available in websites of foreign countries) even though requisitions are signed by clients like NHAI/MOSHRT etc. A lengthy, bureaucratic and time consuming procedure is required to be completed. Collection of run-off data independently by a consultant is prohibitively costly, time consuming and beyond the scope of TOR for highway projects.

2.2.4 Sediment Data

Sediment data, e.g. bed and suspended loads, size and distribution of sediments etc. are needed for finding scour depth, aggradations or degradation in a river. Such information, even if available with CWC or other Govt. agencies are seldom available for reasons already stated under 2.2.3. However, river bed samples (collected from local bore holes at the bridge site) are analyzed to determine type of river bed material, sieve size distribution, non uniformity coefficient etc. for computation of maximum scour depth and design of protection works etc.

2.3 River Survey Data

River survey data e.g. contour plan, L-section, cross-sections, HFL etc. are vitally required for fixing the Location of the bridge and waterway required for the bridge. IRC-5, 1998 and IRC Pocket book for bridge engineers (IRC Handbook, 2000) gives the details of river survey data to be collected at a bridge site.

2.4 Morphologic investigations

Morphologic investigations in regard to the history of river behavior in the vicinity of the bridge site in the past as well as future change in flow pattern should be carried out. Morphologic behavior of a river is governed both by the flow of water and sediments in the river. Lane (1957) , Lacey (1929), Garde & RangaRaju (2000), Schumm (1980) and many other river engineers have developed procedures for determining stability and regime characteristics of rivers. When the river is in meandering state, prediction of migration rate of meander and the effect of bridge on the change in meander pattern and its effect on the bridge, river training measures (e.g. approach embankments, guide bunds, spurs etc.) that may be needed are to be ascertained for deciding waterway. Many a times, fixing waterway without proper morphological study has resulted in excessive cost of maintenance and other problems related to safety of the bridge and the approach embankments.

2.5 Site Investigations

Site investigations for foundation, bank and bed materials, availability of construction materials, river behavior, flood plain characteristics, effect of afflux on the adjoining areas upstream etc. are some of the vital information required in selection of location of a bridge and fixing its waterway.

3.0 ESTIMATION OF DESIGN FLOOD FOR DETERMINING WATERWAY

Since waterway under a bridge depends mainly by the magnitude of peak discharge to be passed under the bridge without creating any harmful afflux, design flood has to be determined carefully. Any under estimation of flood will lead to excessive afflux and damage to the bridge and appurtenant works. Whereas, overestimation of design flood will result in longer waterway and increased cost. From safety and economic considerations, IRC recommends a design flood of 50 years return period. For very important bridges, Peak flood of 100 years return period is recommended. This is more or less an universal practice.

3.1 Computation of Design Flood

Detailed procedures for computation of design flood have been outlined in IRC-5, 1998. for major bridges and IRC SP:13, 2004 for minor bridges. Various methods of estimation of design flood are:

(i) Empirical method
(ii) Rational method
(iii) Weir-Orifice method
(iv) Slope-Area method
(v) Unit hydrograph method
(vi) Flood frequency method

The methodology of computation of peak flood by different methods are available in the IRC/IS/RDSO codes/guidelines and flood estimation report by CWC. Author, however, wishes to point out some of the limitations of the different methods below.

3.2 Limitations of Different Methods for Flood Estimation

In empirical method e.g. Dicken's or Ryve's formulae, the value of C (Dicken's Eq. $Q = C\,A^{3/4}$) may vary widely from place to place and it may give wrong result unless C-value is known correctly. Usually, C-value is found to decrease with increase in catchment area (A). Rational method should be used only for small catchment area, say up to a maximum of 25 sq. km. This is because of the fact that in the rational formula ($Q_p = 0.0278\ f\ P\ I_c\ A$), it is assumed that the critical rainfall intensity (I_c) occurs uniformly over the entire catchment area. It is also difficult to determine the permeability coefficient (P) precisely unless one knows the exact land use, terrain slope, soil and vegetative cover etc.

The weir-orifice formula used for finding afflux in a bridge (IRC: SP-!3) can be used in flood estimation provided the exact amount of afflux is known at the bridge site. There is hardly any gauge record in majority of bridges. Moreover, the coefficient of discharge (C_d) in the equation ($Q = C_d L H^{3/2}$) varies substantially, depending on submergence, approach and exit conditions, bridge geometry etc.

Slope-Area method is a very popular method of estimating flood discharge. In this method, Manning's equation ($Q = 1/N^* A_f R^{2/3} S^{1/2}$) is used for computing design flood from the known flow area (A_f), hydraulic mean depth (R) and the longitudinal bed slope (S). Although the HFL u/s of bridge is recorded from local enquiry for finding A_f,, the discharge so computed can not be given any return period, unless a continuous record of annual peak gauge levels are available at the bridge sites. At least 10 to 15 consecutive years' peak flood levels are needed in order to find the probable HFL with a frequency of 50 years. With high afflux and wide flood plains, Manning's method is not applicable since the flow in such situation is not uniform. In Manning's equation, S is energy slope which may be different from the longitudinal bed slope of river used for computation. In a flat terrain, it is very difficult to determine S-value correctly unless L-section is prepared for a long reach of river Values of roughness coefficient (N) also varies from place to place, depending on bed conditions, geometry and vegetative condition of main channel and flood plain, meander characteristics etc.. Moreover, the flow area A_f is usually computed from the bed profile surveyed during lean flow season. Exact bed profile during the passage of flood is unknown since there is hardly any such survey data available during flood.

Unit hydrograph method can be used in flood estimation provided actual flood hydrograph for an isolated storm is available and the design rainfall distribution corresponding to 50-year return period is known. For medium catchments up to 2500 sq. km., CWC flood estimation report recommends use of synthetic unit hydrograph for estimation of design flood from the published rainfall of 50-year/100year return period and their distribution with time. Details are available in the reports prepared separately for different regions in India . It is, however, assumed that all the streams in a given region will have similar hydro-meteorological characteristics which may not be always true.

Flood frequency analysis, from gauged peak floods of at least 15 years, is a reliable method of determining design flood. But the gauged discharges may not be correct, especially where floats are used in a limited section of waterway for finding surface velocities and then corrected by multiplying with a coefficient (about 0.87) which may vary from place to place. Zero gauge level and current meters need frequent calibration. Use of modern equipments e.g. ADCP (Accoustic Doppler Current Profiler) mounted on boat fitted with GPS is a very efficient and quick method of finding river flow. But these are costly devices and need trained manpower.

4.0 DETERMINATION OF WATERWAY FOR A NEW BRIDGE

When a new bridge is to be constructed, a designer has all the freedom to provide waterway as required. As per IRC-5, waterway (W) should be equal to Lacey's regime waterway (P) given by the equation:

$$P = W = 4.8\, Q_d^{1/2} \tag{3}$$

where

Q_d = design flood discharge in m^3/sec,

P = Wetted perimeter in meter.

W = Linear waterway in metre (for wide river W is almost equal to P)

The code also stipulates that the waterway so found should also be compared with linear waterway at HFL corresponding to design flood discharge and the minimum of the two should be adopted as the clear waterway under the bridge. The methodology for determining waterway under different situations is discussed briefly underneath.

4.1 In a Hilly Terrain

In a hilly or mountainous terrain where the river flows in gorges with steep bed slope, the flow is usually in supercritical state when depth (y) is small and velocity of flow (V) is very high. In supercritical flow, Froude's number of flow, defined as $Fr = V/ (gy)^{1/2}$, is more than one. Lacey's waterway in such situation is very high compared to linear waterway at HFL. Thus the minimum waterway under the bridge will be determined by the linear waterway at HFL. and not by Lacey's regime waterway. In fact, Lacey's regime condition is not valid in such a terrain at all. Waterway under the bridge in supercritical flow should not be less than the linear waterway at HFL. Because any restriction of normal waterway under a bridge in supercritical flow will result in the formation of hydraulic jump upstream of the bridge which is not desirable. Moreover, restriction of normal waterway will affect free movement of gravels and boulders which move along the river bed during flood season. In other word, the clear span under the bridge should be equal to or more than the linear waterway at HFL so that the river continues to flow in its normal waterway under the bridge without affecting the natural movement of water and sediments, as it used to carry before the construction of the bridge. Obviously, if there is a single span bridge, afflux will be zero. In multi span bridges, however, obstruction due to piers will result in formation of shockwaves at the pier front, if it is blunt or semicircular. Computations of the height of such shockwaves and their control is beyond the scope of this paper. However, streamlining of pier nose will considerably reduce the height of shockwaves around the pier faces. Procedure for determining waterway in a few bridges in supercritical flow is given under the head case studies. In a hilly terrain where the road is to cross the river at higher elevation due to approach condition, the span of the bridge will be determined by the elevation of approach road at the crossing point and as such the waterway will be more than the normal waterway of the river.

4.2 In a sub-hilly/ Trough Terrain

In a sub-hilly/trough region, slope of river bed and stream power per unit width and unit weight (QS_o) reduce drastically resulting in deposition of the sediments brought from the mountainous stretch. In this stretch, the river is found to be unstable and changing its course periodically. As a result, a fan shaped delta-type formation occurs. It is better to avoid construction of any hydraulic structure including bridges in such region and shift it either upstream or downstream of the unstable region since there is always a risk of outflanking of the bridge due to its shifting course. If it is not possible, the past history of river behavior

(Mazumder,2004) in the area must be studied carefully to select appropriate location of the bridge so that the cost of the bridge is less but at the same time bridge safety is ensured. In such stretches, Lacey's waterway is only a guideline but the actual waterway to be provided may be much more depending on width of the fan shaped braided area. which may be several times more than Lacey's waterway.

4.3 In a Meandering Flood Plain

As the rivers in the Himalayan region descend further downstream in the flood plains, longitudinal bed slope reduces further. In this region, the river bed and bank consists of fine alluvial soil which can be as easily be eroded as deposited. Due to an inherent instability (Mazumder, 1993) of any natural stream like a river, the river flow is hardly axial. During high flow or flow at bankful stage, the river erodes its outer bank and the eroded materials get deposited on the inner bank opposite to the eroded one. It is due to this process of simultaneous erosion and deposition on alternate banks, rivers flow in meandering bends as illustrated in Fig.1. Wang (1992) developed a mathematical model of the meandering process to prove that the typical cross – slope observed in a meander (Fig.1) with lower bed elevation on the outer side of the bend (due to erosion) and higher bed elevation on the inner bank side (due to deposition) arises out of secondary current which is essentially needed for the river stability. A lot of study on meandering bends have been made by Oddgard (1986), Rozovsky (1957) and other river scientists.

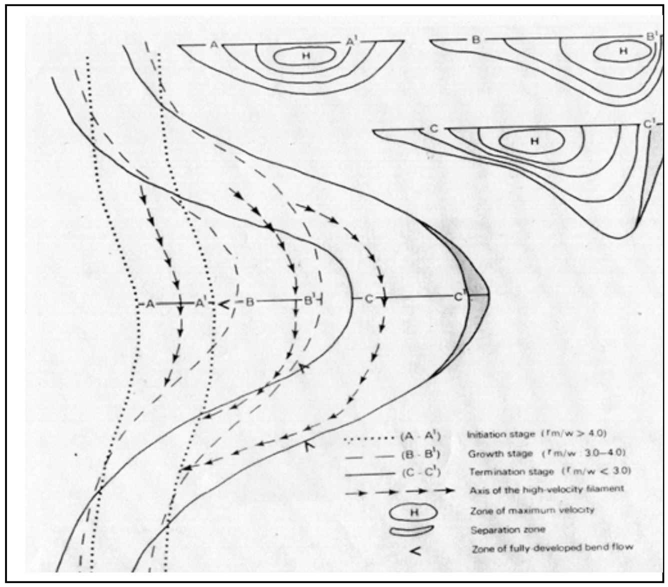


Fig.1 Development of Meander in a Flood Plain

In the meandering stretch, the river develops a wide flood plain known as meander belt over the years. The river is often found to change the meander pattern subjecting both the banks to either erosion or deposition In a meandering belt, it is customary to provide waterway equal to Lacey's regime waterway (P) with guide bund and approach embankment in the flood plain. in case of major bridges as illustrated in Fig.2..

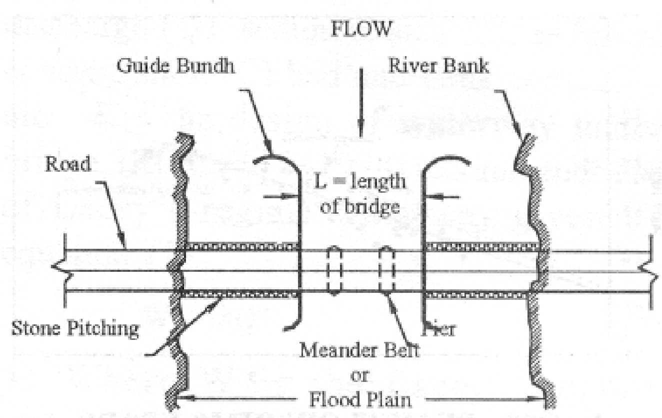


Fig.2 Guide Bund and Approach Road with Pitcing

In case of medium and minor bridges, however, it is customary to provide waterway under the bridge less than normal / Lacey's waterway by contraction of normal / Lacey's waterway with a view to reduce cost. The IRC code (IRC Handbook, 2000) permits a maximum amount of restriction of up to

1/3 rd of normal / Lacey's waterway (i.e. a fluming ratio of about 0.67) with a rider that the afflux due to such restriction should not be more than 15 to 20 cm. In many of the existing bridges, however, restriction is found to be more than 33% of normal / Lacey's waterway resulting in excessive afflux and other problems discussed by the author in his earlier papers (Mazumder et al, 2003).

4.4 In a Deltaic Region

In the deltaic stretches, longitudinal bed slope of the river becomes extremely small varying from 1 in 10,.000 to 1 in 20,000 or even less. In such a flat. terrain, the stream power reduces to such an extent that even the fine sediments like fine silts and clays start depositing in the channel beds and banks. With reduction in conveying capacity (due to siltation), river divides and starts flowing in multiple channels forming deltas (like Sunderban, Mahanadi etc). The large volume of water carried by the rivers from its catchment can not be conveyed with very little conveying capacity at their bankful stage. As a result, flow from one channel often shifts to another channel and as such prediction of design flood in any particular channel becomes difficult.

Many of the rivers in their deltaic stretch are also subject to backflow during high tides. Thus, determination of waterway in deltaic channels is a very difficult task due to unsteady varying flow over time, unless river is trained with flood embankments to follow a steady course. Provisions are to be made for dredging to maintain the conveying capacity and navigability (where there is inland transport) to carry the design flood without spilling of the banks .of the river.

The flood embankments are usually constructed at a sufficient distance away from the natural banks of the river main channel in order to avoid damage to the embankments. Complete submergence of the area in between the embankments occurs due to storage of incoming flood water during tidal lockage period at high tides when sea level rises above the river high flood level at its outfall. In the absence of any road, the stored flood water upstream flows downstream through the main channel and flood plains (in between the flood embankments).as the sea level falls during low tides. When an all weather road is be constructed in such tidal stretch, waterway under the bridge across the river should be sufficient enough in order to avoid undue afflux above normal high flood level (assuming free fall in to the sea) just upstream of the proposed bridge. Usually, Lacey's waterway corresponding to design flood will be adequate. It will, however, be necessary to compute maximum possible afflux due to restriction of flood plain width. If the afflux is high (say more than 10 cm or so), waterway should be increased. AASTHO-type equation (6) should be used to compute afflux instead of Molesworth equation (7) which is not applicable for rivers with wide flood plains and non-uniform approach flow. In case waterway is less resulting in higher afflux, there is likelihood of outflanking of the bridge. Elliptical type guide bunds as per Lagasse's design criteria (Lagasse, 1995) on either side of the bridge should also be provided for the safety of the bridge. Other things remaining same, the guide bunds reduce afflux and provide .an uniform flow under the bridge without any obliquity/skew flow. They also ensures safety to the bridge reducing the risk of outflanking.

.

5.0 DETERMINATION OF WATERWAY WHERE AN EXISTING BRIDGE IS TO BE WIDENED

For a new bridge, a designer has the freedom in deciding waterway as required in various types of terrains discussed under clause 4.0. Where a road is to be widened and the bridge is to be constructed adjacent to the existing bridge, it is customary to provide same waterway under the new bridge, unless it is decided to dismantle the old bridge due to a number of reasons e.g.

(i) the existing bridge is very old and structurally unsafe

(ii) the bridge deck is at low level resulting in submergence during flood season

(iii) there is high restriction of normal waterway resulting in excessive afflux, submergence of valuable land upstream, overtopping of approach road, scouring of bed and banks and high maintenance cost .

(iv) formation of sharp meander in the vicinity of the bridge (due to inadequate waterway) with high flow obliquity and river flowing almost parallel to the approach embankments causing scour and possible breach of road embankment and outflanking and requiring costly protection measures.

(v) high cost of river training to control meander, reduce scour and for restoring the original course of river flowing at right angle to bridge axis.

Afflux is due to head losses within and outside a bridge. Most of the existing. medium and minor bridges are provided with abrupt entry and exit since the return wings are either at right angle to the abutments or at. 45^0 with the abutment. In both cases, flow separates from the sides and there is high head losses in entry or exit. Where flow is choked due to high restriction of waterway, hydraulic jump occurs downstream of the bridge resulting in considerable head loss. Entry and exit losses can be reduced by providing smooth inlet and outlet transitions i.e. by designing the wings such that flow separation can be eliminated. Choking of flow and jump formation can be eliminated by lowering bed level under the bridge with paved bed. In several minor bridges with highly restricted waterway, smooth transitions can be provided, both upstream and downstream of the bridges (without dismantling) to reduce head loss and afflux. With such smooth transitions, the water surface profile lowers considerably at the entrance to the bridge thereby increasing freeboard.

6.0 SOME CASE STUDIES

Examples of determining waterway for a few bridges under different situations, as discussed above, are furnished for both the cases i.e. for new bridge as well existing bridges where widening is required as waterway under the existing bridge is found to be inadequate or the bridge is found submerged during high flood.

6.1 Computation of Waterway for New Road Bridges

As already discussed, a designer has full freedom to provide waterway as required for a new bridge and also where an old bridge is to be dismantled for different reasons. Computation of waterway for a new bridge is illustrated by typical examples under different terrain conditions.

6.1.1 In Hilly/Mountainous Terrain

Waterway for the bridge on NH-44 over river Tamung in Meghalaya

There are 29 minor bridges in a stretch of 70 km on the road. Cross-section of a typical stream, Tamung, is shown in Fig. 3 (a) The river runs at a longitudinal bed slope of 1 in 4.9 at the proposed bridge site as shown in fig. 3 (b). Estimated design discharge of 50 year return period is found to be 110 cumec at a flow depth 2m. Mean velocity and Froude's no. of approach flow at the proposed bridge site at the design discharge are found to be 15 m/sec and 2.82 respectively, indicating supercritical flow. Lacey's waterway (Eq.3) and regime depth by Blench's equation as per IRC-5 (Eq.4) are found to be 51m and 5m respectively

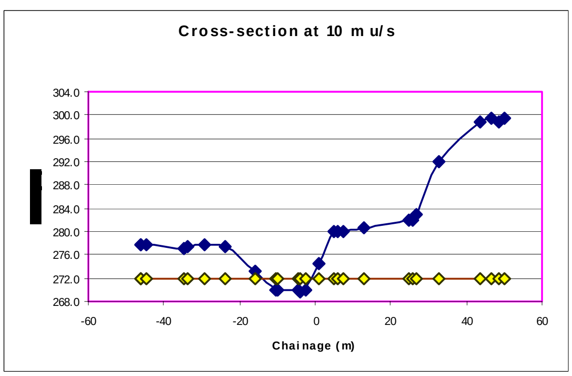
$$R=!.35\ [(q^2/f)^{1/3}] \qquad (4)$$

Actual linear waterway at design normal HFL is, however, found to be only 15m.as shown in fig. 3(a).

Regime depth computed by Lacey's original regime formula given by Eq 5 (not mentioned in IRC Code) is found to be 2.5m.only

$$R= 0.475\ (Q/f)^{1/3} \qquad (5)$$

Since the flow is supercritical and the terrain is hilly, IRC formulae either for waterway or for regime depth is not applicable. Considering the linear waterway at HFL and freeboard requirement, a single span of 18m (which is only 0.35 of Lacey waterway) was recommended for the proposed bridge. Since there is no restriction of waterway (due to supercritical flow), there is no afflux. Maximum scour depth below river bed is recommended as 2.5m only with provision for open foundation.



.Fig.3(a) Cross-Section of Tamung River

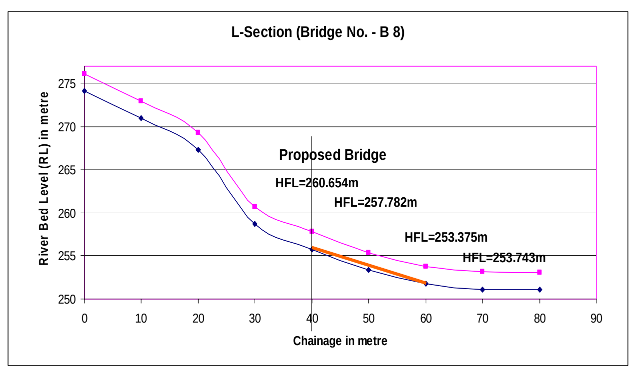


Fig.3(b) L-Section of Tamung river at Bridge Site

6.1.2 In a Sub-Hilly / Trough Type Terrain

Waterway for bridge on NH-31 over river Torsha in West Bengal

Originating from Tibet. at an altitude of about 6000 meter, river Torsha (also called Siltorsha ln India and Amu Chu in Tibet) flows through Tibet, Bhutan and West Bengal before joining river Brahmaputra.. Total catchment area up to the proposed bridge site is about 3950 sq. km out of which 2056 sq km is rainfed and the rest under permanent snow cover lying mostly in Bhutan and Tibet. After traversing a distance of about

225 km in the mountainous terrain of Himalayas, it descends to the plain near Hasimara in West Bengal where it deposits a large amount of sediments .consisting of sands, gravels and. boulders The river in this trough region is unstable and found to often change the course. Due to sediment deposition in its bed, the flood water spills over its banks and a number of tributaries (like Buri Torsa, SonapurTorsa etc) originate from Torsha and rejoins it, resulting in a fan shaped delta like structure in the area. near the bridge site. Low bed level of the river is 54.631 m and the bed slope is 0.00089 i.e 1 in 1124 Design discharge (of 50 year return period) at the bridge site is estimated as 4,481 cumec and the corresponding design HFL is 59.651m. Lacey's regime waterway (Eq 3) corresponding to design discharge is found to be 301 m.. But the observed linear waterway at design HFL is found to be 600m. A clear waterway of 567m is recommended for the bridge considering the periodic shift of the main river course in this trough zone. Mean velocity of approach flow and corresponding Froude's number. of flow are found to be 1.62 m/sec and 0.232 respectively.. Considering highly sub-critical flow, the waterway could be restricted further since Lacey's waterway is 301m only. However, considering the instability of the river in the region, waterway provided under the bridge is kept as 1.88 times Lacey's waterway.

Lacey's regime depth (R) in such a situation, where looseness factor (actual waterway / regime waterway) is more than unity, regime depth should be determined from Lacey's original equation (Eq.5 not mentioned in IRC code) and not from Blench's equation (Eq.4) given in IRC code. With 30% increase in discharge for foundation design as per IRC-78, R-values found from Eqs. 4 and 5 are 6.72m and 8.75m respectively. Therefore, MSL recommended for pier is 42.151m.(59.651- 2x8.75)

6.1.3 In a Meandering Flood Plain

<u>Waterway for the new bridge connecting Shantivan with Geeta colony(Delhi)</u>

The bridge under construction is on the d/s side of the old railway bridge over river Yamuna .which is in a meandering state near Delhi. Main course of the river shifts from right to left bank near the bridge site and the bridge is located on the main channel on east (left) bank near Geeta colony. A long approach embankment on Yamuna flood plain connects the bridge with Shantivan with 140m long guide bunds on either side. The left bank on the outer side of meandering bend is protected with spur field to avoid erosion and prevent further lateral migration of the river towards left bank side. Some of the salient design parameters of the new bridge are given below:

i) Design Discharge for Waterway /Guide bund -12,750 cumec (4.5 lakh cusec)

(ii) Corresponding HFL - 208.21 (u/s) and 208..03 (d/s)

(iii) Design discharge for foundations – 14,866 cumec (5.25 lakh cusec).

(iv) Corresponding HFL – 208. 47 (u/s) and 208.27 (d/s)

(v)Maximum discharge intensity(q) along left bank (from model study by CWPRS) -36 cumec/m

(vi) Mean size of non-cohesive bed and bank materials - d_{50} *= 0.16 mm*

(vii) Linear waterway at design HFL (208.03m) between left and right flood embankments – 2,200 m

(viii) Lacey's Regime waterway corresponding to design flood 12,750 cumec – 542 m

(ix) Clear Waterway provided under the bridge (normal to flow axis) - 560 m

(x) Lacey's regime depth (R) corresponding to14,866 cumec - 15m as per Lacey equation (5)

(xi)Regime depth (R) as per Blench eq. (4) for maximum discharge intensity($q=36m^3$ /sec) -16m

(xii) Maximum scour depth for piers (2R) below HFL 2R=-32 m

(xiii)Maximum scour level (MSL) -176.27m (208.27- 32)

(xiv) Maximum velocity (V) considered for design of protection – 4 m/sec (as per CWPRS study).

It may be noted that although the river has a wide meander belt of about 2,200 m, waterway provided under the bridge is 560m only i.e. almost the same as Lacey's waterway as per IRC-5 recommendation..

6.2 Existing Bridges to be Widened to Accommodate more lanes

<u>Widening of bridges adjoining existing ones on NH-31C (Birpara in West Bengal to Assam border)</u>

Hydraulic computations were carried out for finding the adequacy of waterway for 72 existing bridges in a stretch of about 150 km in NH-31C.in West Bengal. It was proposed to widen the bridges from 2 lane to 4/6 lanes under the Prime Minister's 'Gram Sadak Yojna'. Afflux (h_1^*) was computed for design discharges by using equations (6), (7) & (8) given below:

AASTHO Formula

$$h^*_1 = 3(1- M)\, V^2_{n2}/2\, g \qquad \ldots\ldots\ldots\ldots\ldots \qquad (6)$$

Molesworth Formula

$$h_1^* = [V^2/17.88 + .015]\, [(A/A_1)^2 - 1\,] \qquad \ldots\ldots\ldots\ldots\ldots \qquad (7)$$

IRC: SP-13 Equations

For choked weir type flow:

$$Q = C_d\, L_{eff}\, (D_u + u^2/2g)^{3/2} \qquad \text{if } h^*_1 D_d > 0.25 \qquad 8(a)$$

For orifice type submerged flow:

$$Q = C_d\, L_{eff}\, D_d\, \sqrt{(2g.\, h_1^*)}- \qquad \text{if } h_1^*/D_d < 0.25 \qquad 8(b)$$

Where

h^*_1 = Afflux in metre

$M= Q_b / Q$

Q_b = that portion of the total discharge (Q) in the approach channel within a width equal to the projected length of the bridge in cumec

$V_{n2} = Q/A_{n2}$ in m/s

A_{n2} = gross area of waterway under the bridge opening at normal stream depth corresponding to design flood discharge in m^2

V = mean velocity of flow (in m/s) in the river prior to bridge construction at normal HFL,

A = area of flow section (in m^2) at normal HFL in the approach river section

A_1 = area of flow section (in m^2) under the bridge

C_d = coefficient of discharges for weir and orifice type flows respectively. Cd and Co values are given in the IRC: SP-13 ; D_u and D_d are the upstream and downstream depths measured from the lowest bed level under the bridge taken as datum. Afflux is given by $h1^* = (D_u - D_d)$

In 1993 flood, 5 bridges on the road were washed out. 39 bridges were found to be unsafe due to inadequate or negative free board. But most of all these bridges were in healthy condition. Under a directive from MOSRT (client), it was decided to retain all those bridges which had positive free board irrespective of magnitude of the freeboard. Free board of 17 minor bridges out of 39 bridges was increased by providing smooth transitions both upstream and downstream of the bridges. Provision of smooth elliptical type transitions reduced head loss and decreased afflux. Out of the remaining 22 bridges, 10 bridges were to be made 4 lane new bridges either due to realignment of the new road or for improvement of road geometry. In the case of remaining 12 bridges, however, afflux was found to be too high due to inadequate waterway as shown in **Table- 1**. Afflux computed by equations 6, 7 & 8 were compared with the afflux given by the difference between observed upstream and down stream HFL - both found from local enquiry as indicated in table-1.Under the above conditions, there were two possible options, as discussed below:

OptionI

One option was to provide adequate waterway for both the new two lane bridges as well as the existing two lane bridges either by adding extra spans required or by dismantling the old bridges and constructing new ones with increased waterway i.e with spans same as that required for the new bridges. Both are costly and difficult propositions. Opening new span and raising deck level for the existing old bridges are extremely difficult to execute. **Fig. 4(a)** gives a sketch indicating this option. Observed and estimated values of affluxes [by equations (6), (7) & (8)] with existing waterway are given in Table-1. Obviously, because of inadequate waterway under the existing bridges, affluxes are too high resulting in either very small or negative freeboard under the existing bridges. The existing waterway,. Lacey's waterway and the recommended waterway (if existing bridges are to be dismantled) are given in Table-2. Estimated affluxes and corresponding design HFL (u/s) for option -I are indicated in columns (6) and (7) of table-2.Due to provision of longer waterway, the affluxes and the u/s HFL were reduced in comparison to their original values given in table-1..

Option II

Since all the existing two lane bridges were in healthy condition, the second option was to retain all the old two lane bridges on the downstream side of the new long span two lane bridges at higher elevation (as per free board requirement) and connecting the old and new bridges by a well-designed contracting transition as shown in **Fig. 4(b)**. Since the old bridges are placed downstream of the new bridges, free boards in the old existing bridges were sufficiently increased due to draw down of water surface as indicated in columns (10) and (11) of Table-2. Except two bridges, all other bridges developed weir type control at the entry to the existing bridges of shorter span resulting in large draw- down from affluxed depth (y_1) at u/s of new bridges to critical depth (y_c) at u/s of old bridges. High flood levels upstream of long span-new bridges and short span old bridges are given in columns (10) & (11) respectively. Due to lowering of HFL's (column-11), free boards under the existing bridges increased considerably as indicated in column (12) in Table-2. Even if the old bridges fail, it will not cause any damage to the new bridges which are located upstream of the old bridges.

Table –1 : Afflux and Freeboard for Existing 12 Bridges on NH-31C

Sr. No.	**Existing Bridge Details**						**Design Dischage (cumecs)**	**Estimated Afflux (m)**		
	Effective Water way (m)	**Deck Level (m)**	**HFL as per local enquiry (m)**		**Afflux from local enquiry (m)**	**Free Board (m)**				
			U/S	**D/S**				**Eq. (6)**	**Eq (7)**	**Eq. (8)**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
1	116	112.29	110.24	108.2	2.04	0.06	1594	2.56	0.72	1.95
2	57.3	108.508	106.608	104.9	1.708	0.05	1022	1.29	2.3	1.76
3	76.4	89.669	89.576	86.5	3.076	-1.71	1140	3.5	0.84	1.35
4	56	91.759	91.176	88.39	2.786	-1.42	679.5	2.16	0.85	1.2
5	6	94.997	94.406	92.673	-	0.09	61	2.59	3.82	1.733
6	48	118.282	116.15	113.36	2.79	0.03	943	1.25	2.22	3.155
7	37.8	97.337	96.137	94.35	1.787	-0.4	198	0.94	0.45	1.234
8	25	80.01	77.51	76.3	-	0.27	409	1.38	2.53	1.207
9	42.8	59.06	57.128	55.15	1.978	0.53	383.7	1.28	0.71	2.128
10	56.8	49.326	48.026	46.46	1.566	-0.6	647	1.22	0.65	0.732
11	9	51.155	51.089	48.913	-	-0.93	33.77	1.43	0.64	2.13
12	9	50.353	49.7	48.113	-	-0.25	64.32	0.81	2.13	2.12

-

Table – 2 : Afflux for different options and Freeboard under Existing Bridge

Sr. No.	Design Discharge (cumecs)	Lacey's waterway (m)	Effective waterway (m)		Option I (4 - Lane New Bridge)		Option II (2 - lane new Bridge upstream and 2- lane existing Bridge downstream connected by transition)				
			New Bridge	Existing Bridge	Estimated Afflux (m) Eq. (8)	u/s HFL (m)	Estimated Afflux (m) Eq. (8)	Soffit Level for Existing Bridge (m)	HFL u/s of new bridge (m)	HFL u/s of old bridge (m)	Free Board under existing Bridge (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
1	1594	191.64	140	116	1.4	109.002	1.90	110.19	109.200	107.839	2.351
2	1022	153.44	90	57.3	0.709	105.609	1.23	105.935	106.130	104.583	1.352
3	1140	164.09	100	76.4	0.211	87.325	1.00	87.869	87.500	85.628	2.241
4	679.5	125.12	75	56	0.585	89.048	0.80	89.759	89.190	88.790	0.969
5	61	37.96	15	6	0.35	93.027	1.36	94.497	94.033	93.336	1.161
6	943	147.4	88	48	1.308	114.715	2.71	116.182	116.070	114.486	1.696
7	198	68.39	50	37.8	0.938	95.288	0.98	95.737	95.332	94.090	1.647
8	409	97.07	45	25	0.43	76.733	0.80	77.78	77.100	75.887	1.893
9	383.7	95.2	60	42.8	0.174	55.324	1.65	57.66	56.795	54.474	3.186
10	647	123.62	75	56.8	0.36	47.070	0.67	47.426	47.130	46.795	0.631
11	33.77	28.24	18	9	0.7	49.611	1.97	50.55	50.883	49.829	0.721
12	64.32	38.98	20	9	0.53	48.638	1.88	49.453	49.993	48.870	0.583

It may be pointed out that the affluxes for the four lane bridges under option -II (table –2) was less than those in Table-1 (for the existing 2 lane bridges) due to improved bell mouth entry to the control sections (in old bridge) resulting in higher values of coefficient of discharge in equation (8).

The second option has , however, some shortcomings. The new road was on the downstream side of the existing road, due to available right of way on the down stream side. In case of option II, however, the road had to be widened on the upstream side, as the new bridges were to be constructed upstream of the old bridges on the existing road. For shifting the new road from downstream side to the upstream side of the existing road, it is necessary to provide suitable road transitions locally resulting in curved approaches to the bridges as shown in fig. 4(b). Moreover, the deck levels of the old bridges (at lower level) and new bridges (at higher level) will be different thereby introducing a vertical curve too. Both the horizontal and the vertical curves in the bridge approaches are not desirable from traffic safety point of view.

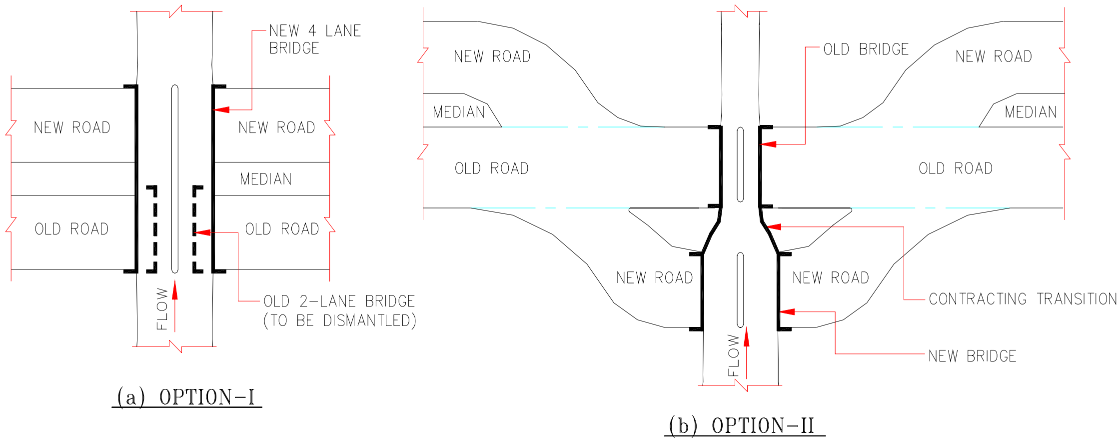


Fig.4: Showing two different Options (a) Dismantling Old Bridges (Option -I) and (b) Retaining) old Bridges (Option-II)

7.0 SUMMARY AND CONCLUSION

Determination of waterway for a bridge needs careful and detailed. topographic, hydrologic, morphologic investigations both in the near and far fields. of a river. Site investigations and collection of data e.g. rainfall, flood flow, sediment flow, river behavior in the past, land use etc. are vitally required in deciding waterway under the bridge. Although Lacey’s equation for waterway corresponding to design peak flood gives a guideline, actual waterway may be substantially different from Lacey’s waterway, depending on the type of terrain through which the river passes. In a hilly and mountainous terrain in the Himalayan region, where river flows at high velocity with Froude’s number more than unity, flow is supercritical.. Lacey formulae are not applicable either for waterway or for scour depth, as given in IRC codes. In such a terrain, waterway under the bridge should be equal to or more than linear waterway at design HFL. Scour depth (R) should be found from Eq.(5) and not by Eq. (4) as mentioned in the code. If the observed depth (Y) above lowest bed level is more than that found by Eq.(5), R should be taken as Y. In the sub-hilly trough terrain, river is mostly found to be unstable and the flood plain may be several times more than Lacey’s waterway. Providing Lacey’s waterway in such terrain may cause wash out /outflanking of the bridge and hence waterway needed may be much more .than Lacey’s waterway. In the meandering flood plain of a river with fine alluvial soil, meander belt is several times Lacey’s waterway. It is usual to provide waterway equal to Lacey’s waterway with guide bund and approach embankment. in case of major bridges. In medium and minor bridges, however, waterway provided is often less than Lacey’s waterway or linear waterway at HFL. IRC code allows a maximum restriction up to 2/3 of Lacey’s waterway. Wherever waterway is restricted, there will be afflux .which should be computed by different methods e.g. Eq. (6), (7), (8). Too high afflux due to excessive restriction of waterway has several ill effects as discussed by author in an earlier paper

(Mazumder.2002). Determination of waterway in a deltaic and tidal reach requires thorough study of river behavior and the effect of tide.

For a new bridge, a designer has freedom to provide waterway as required under different terrain conditions. Where an old bridge requires widening, it should be mandatory to find whether the existing waterway is adequate from the point of view of fluming, afflux and available freeboard. Where the existing bridge is to be dismantled, the new bridge should be provided with sufficient waterway to avoid excessive fluming and harmful afflux. Adequate free-board (as mentioned in IRC -5), depending on design flood, debris flow and other considerations .must be provided in fixing the deck level of a bridge.

Where an existing bridge is in healthy condition, it is difficult to provide waterway larger than the existing one, unless the increase in waterway is marginally higher or the new bridge is sufficiently away from the existing bridge. However, it is possible to improve performance and reduce afflux by providing suitable hydraulic transitions u/s and d/s of the existing/new bridge .with span. same as the existing bridge without dismantling the exiting bridge. Where the waterway under the existing bridge is such that it causes flow choking and a control, it may be possible to retain the old bridge, placed d/s of the new bridge of longer span by connecting the two by smooth transitions.

Case study of some bridges have been given to illustrate the design of waterway under different terrain conditions. separately for new bridges and bridges requiring widening.

8.0 ACKNOWLEGEMENT

Author acknowledges gratefully the help, co-operation and encouragement received from ICT authorities. Help received from his colleagues in the bridge division and water resources division of ICT where author had been earlier engaged as general manger (looking after hydrology and drainage)., is thankfully acknowledged.

9.0 REFERENCES

1.CBIP,1989 , “River Behavior, Management and Training”, Vol.I, published by Central Board of Irrigation and Power, New Delhi

2. Garde, R.J. and Ranga Raju, K.G.(2000) “Mechanics of Sediment Transport and Alluvial Stream Problems”, 3rd Ed., New Age Int. Pub. Pvt. Ltd. New Delhi.

3. IRC:78: (1980) “Standard Specifications and Code of Practice for Bridges – Section VII Foundation and Substructures, Part-I, General Features of Design”. Published by Indian Road Congress, New Delhi

4. IRC:5 (1998) “Standard Specifications and Code of Practice for Road Bridges – Section 1” Published by Indian Roads Congress, R.K.Puram, New Delhi.

5.IRC Hand Book (2000) “Pocketbook for Bridge Engineers”, The Indian Roads Congress, New Delhi

6. Lacey, G. (1929) “Stable Channel in Alluvium” J. of Instn. Of Civil Engineers, U.K., Paper no. 4736,Vol. 229, PP.

7. IRC:SP:13(2004) “Guidelines for the Design of Small Bridges and Culverts”, Published by Indian Roads Congress, New Delhi.

8. Lagasse,P.F., Schall,F. Johnson, F., Richardson, E.V. and Chang, F.,(1995), Stream Stability at Highway Structures, Department of Transportation, Federal Highway Administration, HEC-20

9. Lane (1957), “A study of the shape of channels formed by Natural Streams flowing in Erodible Material”, M.R.D. Sediment Series No. 9, U.S. Army Corps. Of Engineers, Missouri River.

10.Mazumder, S.K. (1993), “Stability of River Downstream of Hydraulic Structres” Proc. of VIII APD-IAHR .Congress, Vol II org by CW&PRS Pune, Oct. 20-23, pp 273-282

11. Mazumder, S.K., Rastogi, S.P. and Hmar, Rofel (2002) “Restriction of Waterway under Bridges”, J. of Indian Highways, Vol. 30, Nlo. 11, Nov

12.Mazumder, S.K. and Dhiman, Rajni (2003) “Computation of Afflux in Bridges with particular reference to a National Highway” Proc. HYDRO-2003, Pune (CWPRS), Dec 26-27

13. Mazumdar S.K. (2004), “River Behaviour Upstream and Downstream of Hydraulic Structures” Proc. Int. Conf. on Hydraulic Engineering : Research and Practice (ICONHERP-2004), organized by Deptt. of Civil Engineering, IIT, Roorkee, Vol. I, P.253-263 in honour of Prof. K.G. Ranga Raju.

14.Ordgaard, A.J. (1986), “Meander Flow Model-1: Development” J. of Hyd. Engg., ASCE, HY-12,

15..Rozovski, J.L. (1957) “Flow of Water in Bends of Open Channels” Acadamy of Sciences of the Ukranian SSR,. Translated in English by Prushansky, Y. Israel Programme of Scentific Translation.

16.Schumm, S.A. (1980) “Plan form of Alluvial Rivers” Proc of the International Workshop on Alluvial River Problems held at Univ. of Roorkee, March 18-20

17.Wang, P. et al (1992) “Numerical Prediction of Transverse Bed Slope and Bed Profiles in Curved Alluvial Streams” Proc. of APD – IAHR VIII Congress at CW&PRS, Pune, Vol. II, Oct 20-23