# Performance review of Jamuna Bridge River Training Works 1997-2009

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ABSTRACT: Recently a review was made of the performance of the Jamuna Bridge River Training Works, (RTW) .crossing the braided Brahmaputra River in Bangladesh. The RTW are functioning since 1997. The review was carried out to identify any improvements which could possibly be made both in the design proper and in the supporting studies for forthcoming bridge projects. In the review period of 13 years, the RTW were not fully tested, as during none of the years was the design discharge approached or was the upstream channel pattern characterized by extreme outflanking. Although the yearly maintenance required until 2009 is on the average only 0.13% of the capital costs of the RTW, still some unexpected phenomena were observed. The design scour depths were exceeded substantially, and scour holes of almost 45 m below average flood levels were observed. The scour occurs during outflanking and additional studies are needed to understand the cause and possible remedies. Locally substantial shifting of the rock of the slope protection is observed, leaving the underlying geotextile exposed. Regular inspection and maintenance as specified is needed to keep the RTW in optimal condition.

## 1 INTRODUCTION

In Bangladesh only one bridge over the Jamuna River (the local name of the mighty Brahmaputra River) connects the Eastern and Western part of the country (Figure 1). This Jamuna Multipurpose Bridge (JMB, sometimes called Bangabandhu Bridge) was constructed in the period 1994-1998. The river training works (hereafter referred to as RTW), an integral and essential part of the bridge, have as main purpose to ensure that the 10-20 km wide Jamuna River (local name of the Brahmaputra River) passes under the bridge with a total span of 4.8 km only. The design of the RTW was done in the period 1987-1990 but was adjusted prior to and during the construction. The RTW were already completed by mid 1997 and since then served their purpose and have required limited maintenance.

Recently it was decided to construct a second bridge connecting the two parts of the country; this time over the Padma River (see Figure 1), which carries the combined flow of the Brahmaputra and the Ganges Rivers. Within the framework of the design of this second bridge, BBA (Bangladesh Bridge Authority) considered it appropriate to review the performance of the Jamuna Brdige RTW. Specifically this review should deal with the designs in light of the design rules applied and the performance of the RTW in the period since 1997, but also it should address the design boundary conditions used.

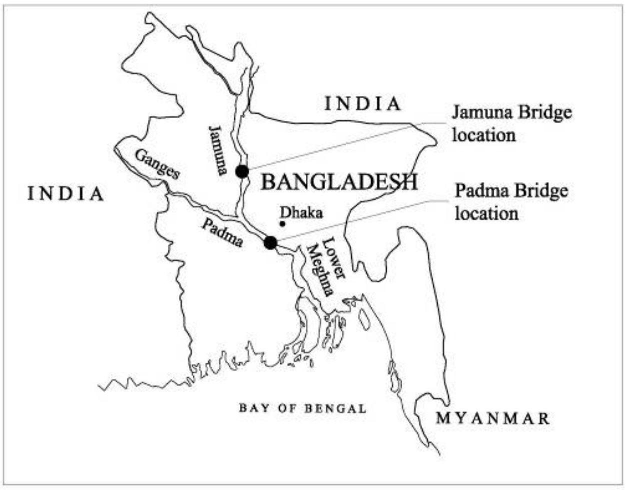



Figure 1: Location of Jamuna Bridge and of proposed Padma Bridge in Bangladesh

Furthermore the different studies (mathematical and physical model studies and otherwise) carried out to support the design should be reviewed as well, to in hindsight assess their relevance.

Hence the consulting firm in charge of the detailed design of RTW for the Padma Bridge (NHC, North-West Hydraulic Consultants, Canada) invited two former NEDECO staff members to carry out this review. One (the second author of this paper) was on behalf of Royal Haskoning Project Manager Phase II design and Dy. Project Director for construction supervision of Jamuna Bridge RTW and the other (the first author) was on behalf of

WL/Delft Hydraulics Senior Morphologist and Modelling Expert in charge of the supporting model studies, the remote sensing study and special surveys.

The present paper summarizes the findings of this review, giving due attention to the performance of the structures and the support provided by model studies. A more elaborate report presenting much more information, including a number of recommendations for the design, construction and maintenance of similar bridges over large braided rivers and the related modeling support, is available (MAUNSELL & AECOM, 2010, Annex F).

Before the design of the RTW of the Jamuna bridge is discussed, it is appropriate to summarize here the main characteristics of the Jamuna River the bridge is crossing. The Jamuna River is a very large braided river with discharges varying between 4,000 and 65,000 m3/s and more, a slope of about 7 cm/km, fine sand as bed material ($D_{50}$ about 0.2 mm), a total width of 10 to 20 km, 2 to 3 channels per cross-section and a bed material load of about 200 million tons/year. Local floodplain level is about PWD + 12 m, where PWD is the local reference level approximately corresponding to mean sea level. For more information on the Jamuna River and its characteristics see Coleman (1969), Klaassen et al (1988) and Klaassen & Vermeer (1988a, 1988b).

## 2 DESIGN RTW AND SUPPORTING STUDIES

*General*

During an early stage of the studies for the Jamuna Bridge it had already become clear that design and construction of a bridge across the Jamuna would require 'state of the art' technology such as probabilistic design methods, advanced mathematical modelling, latest design formulae and concepts in marine works and offshore structures and, last but not least, modern sophisticated construction techniques as practised by large international construction consortia operating large modern equipment. It was also clear that appropriately designed guide bunds should play a key role in the design of the River training Works.

The application of such 'state of the art' technology was, however, not encouraged by the limited knowledge on and understanding of the behaviour of braiding rivers in general, and on the Jamuna River in particular. Also the complicated and dynamic field conditions, in which such a major project would have to be built, had to be taken into account.

This was compensated by spending ample time and resources on supporting studies like remote sensing studies, hydrologic and morphological studies, physical and mathematical modelling of river behaviour during various circumstances with and without river training works, study of bank protection systems, geotechnical investigations, a visit to constructed river engineering works and hydraulic modelling centres in India, etc. subsequent a study of relevant literature on the design of river training works for bridges from India (Garg et al, 1970; Joglekar, 1971).

*1990 RTW design*

From a river engineering perspective key issues to address in the design of the RTW were the overall layout of the river training works, the length and slopes of the guide bunds and the maximum scour depth.

While designing the overall layout use was made of existing 'natural' hard points (near the selected bridge corridor): at the west bank this was the town protection of Sirajganj while a bank with slightly more clay content at the east bank at Bhuapur, more or less opposite Sirajganj, had always resisted erosion in the past. Thus, both 'hard points' locally defined the edge of the braid belt. It was decided to make use of this situation through the incorporation of both hard points into the river training works (RTW) concept and by locating the guide bunds some 5 to 10km south of these hard points.

In planform the guide bunds look like banana-shaped structures. Their function is two-fold: they protect the bridge abutments (approach viaducts, ramps) against current attack and they prevent outflanking of the bridge. In addition, they also protect the flood-free areas behind the guide bunds reclaimed by using the sand dredged from the trenches in which the guide bunds were built. These trenches were necessary to construct the underwater protected slopes of the guide bunds.

Using the results of a theoretical study (Klaassen & van Zanten, 1989), a study of satellite imagery for the period 1973-1987 supplemented with old maps plus an additional physical model study in a tilting facility a formula was developed relating the upstream length of guide bund to distance between south end of hard point and north end of guide bund. Designing the guide bunds according to this formula is expected to prevent outflanking of the bridge. The river would be narrowed to about 5 km in between the guide bunds, as only a bridge of limited length (i.e. 'limited' in relation to width of the flood plain) would provide an economically feasible solution. The supporting studies resulted in guide bunds much shorter than recommended by Indian standards (Joglekar, 1971). For the 1990 RTW layout see Figure 2.

Though extreme scour could well reach values of 43m below average flood level, it was considered impractical to extend the slope protection to such depth. First of all, no dredgers would be able to dredge to such depth (which would limit competition at tender stage), secondly, it would be very costly because of the huge volumes of sand to be dredged and thirdly, too much time would be needed for the dredging. It was ultimately decided to dredge to a depth of 28m below maximum water level during the construction, i.e. to PWD -18m and to provide a so-called falling apron of rock at the toe of the slope to cater for deeper scour. This still meant that considerable volumes of sand had to be dredged (25 million m3 or more). Based on soil investigations and subsequent laboratory testing it was concluded that a statically stable slope could be dredged, protected and maintained at a gradient of 1H:3.5V and in 1990 this gradient was selected for the guide bund slopes. Details on the 1990 design of the RTW can be found in RPT et al (1990a).

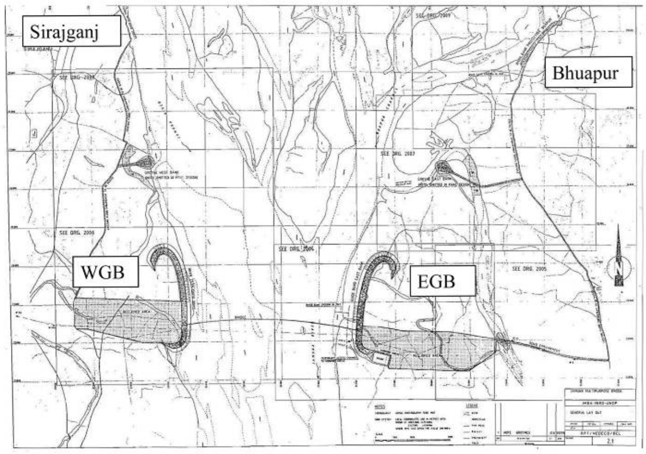
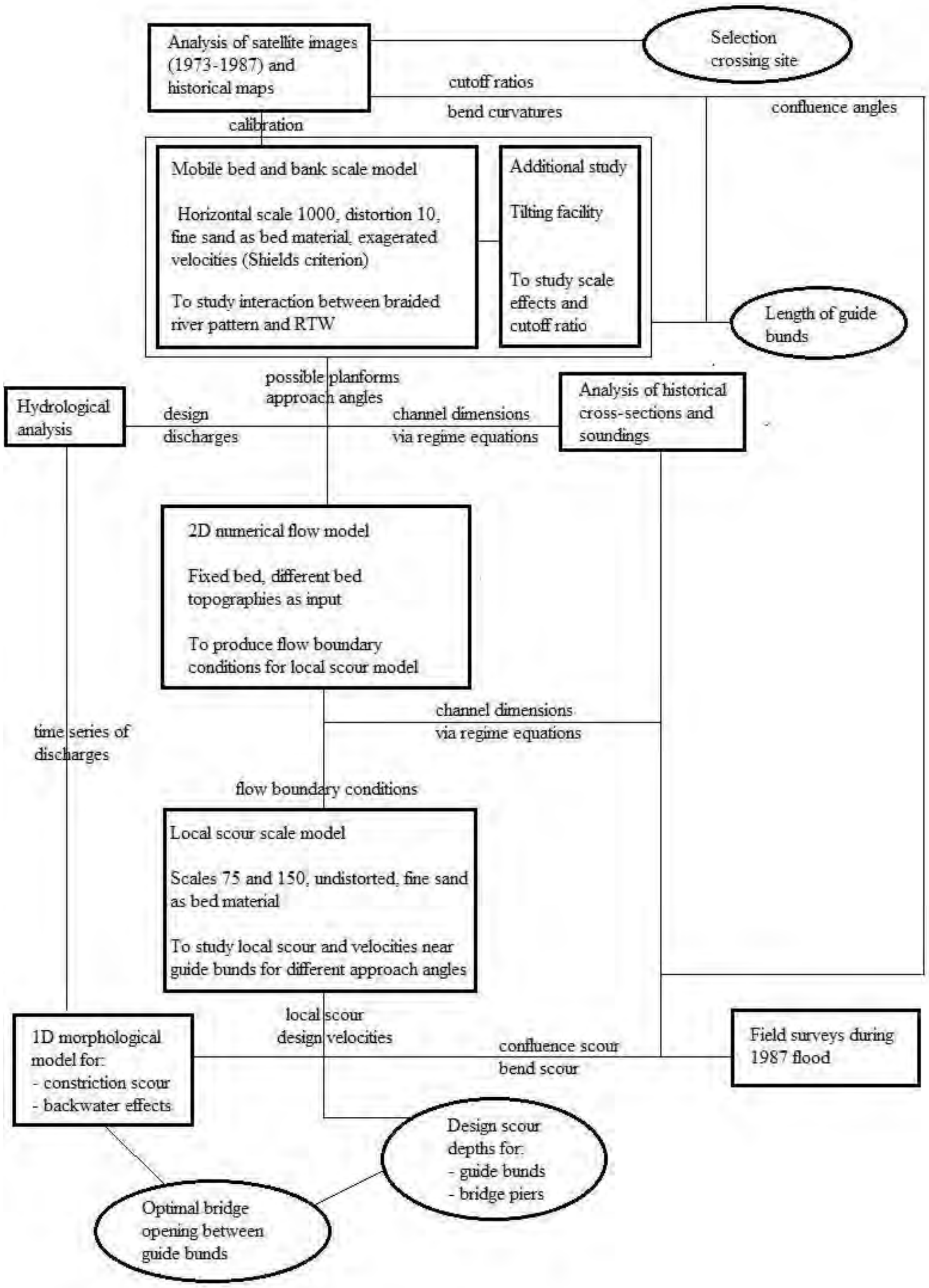



Figure 2: Position and layout Jamuna Bridge RTW 1990 design




*Supporting studies*

As already mentioned above a substantial number of supporting studies were carried out. Figure 3 provides an overview of these studies with an indication of which results were obtained and how these were used for the design of the RTW. In particular the extensive use of satellite imagery and the overall mobile bed and bank model were new. More information on the overall bank and bed model, its possibilities and its limitations and the results obtained are given in Klaassen (1990; 1992). For more details on all supporting studies see RPT et al (1990b). During the review described in this paper the results obtained while these studies were carried out were compared to the observations in the field.

Figure 3: Supporting studies for layout and design of slope protection of the Jamuna Bridge RTW (for more details see RPT et al, 1990b)

(Note that soil mechanical studies are not included in this overview)

*Changes of design prior to and during construction*

The construction of the RTW started only in 1993. At that time it was found that due to the delays in the start of the construction adaptations of the 1990 design was needed because of (i) erosion of river banks resulting in changed banklines and another site configuration; (ii) major shifting of main channels; (iii) increased flow through secondary channels other than the main fairway; (iv) discovery during construction of sub-soil conditions not known at the time of design. This meant that the design of both guide bunds and the Bhuapur hard point had to be adapted as regard location, plan form and cross-section. The need for changes, the changed design and the construction of the RTW is extensively described in Duivendijk (1997), Oostinga & Daemen (1997) and Tappin et al (1998) and hereafter only a summary is given. More detailed information is available in RPT et al (1996a through 1996d)

The change in layout of the RTW was on the one hand due to channel shifting and on the other to land acquisition constraints. In the period 1987-1992 the outer banks of the braided system at the proposed bridge axis moved 1 km to the east, and 1.5 km to the west respectively. This completely distorted the idea of having a 5km long bridge with guide bunds built in the flood plains at both sides. Because of the expected additional costs it was decided to construct one of the guide bunds on a mid-river char (local name for sand bank. The layout of the RTW as constructed is shown in Figure 4. It can be noticed that the guide bund system is not in the middle of the braid belt anymore and the guide bunds are not opposite each other but slightly shifted. The upstream length of the guide bunds was 2.2 km for the EGB (East Guide Bund) and 2.1 km for the WGB (West Guide Bund).

The required change in cross section was due to difficulties encountered during construction of the west guide bund. During and after dredging the soil was found to be very sensitive to slight dynamic disturbances. This implied that a slope dredged to a gradient of 1V in 3.5H (as designed) could easily be subjected to a flow slide due to liquefaction. As the time to construct one whole guide bund was limited (6.5 months) and repeated flow slides adjacent to sections already completed would seriously jeopardise the ultimate quality of the slope protection works, it was decided (RPT et al, 1996a through 1996d), to apply a more gentle gradient. Down to PWD-4m the gradient would be 1V in 5H, while at deeper levels the gradient would be 1V in 6H.

More gentle slopes decrease the depth of the local scour. This led to the adoption of a higher level of the falling apron (PWD -15m instead of PWD -18m). In the bridge corridor the PWD -18m level was maintained. Some details on the adapted cross-section and on the levels of the different types of slope protection and the falling apron are presented in Figure 5 and Table 1.

Figure 4: Revised layout of Jamuna Bridge guide bunds as built

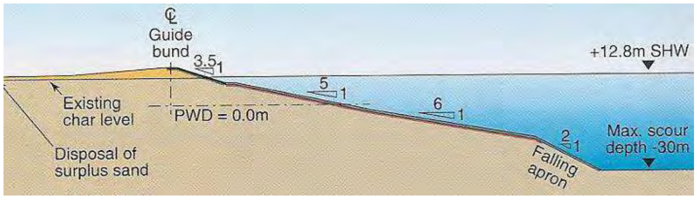



Figure 5: Typical cross-section Jamuna Bridge RTW (after launching of falling apron)

Table 1: Characteristics of slope protection of guide bunds

| Structural element | Aspect | Levels & structural parts | Characteristics |
|---|---|---|---|
| Open stone asphalt | Levels & slope | Upper level | PWD + 15.70 m |
| | | Lower level | PWD + 10 m |
| | | Slope | 1V to 3.5 H |
| | | Thickness | 0.15 m |
| | | Underlying filter | Geotextile |
| Rock slope protection | Levels & slope | Upper level | PWD + 10 m |
| | | Lower level | PWD - 15.00 m |
| | | Slope | 1V to 5H ( > PWD – 4m); 1V to 6 H (lower) |
| | Composition | Cover layer | Broken rock |
| | | Stone size | 0.25 – 0.30 m |
| | | Thickness cover layer | Minimum 0.5 m In practice 0.65m |
| | | Underlying filter | Geotextile with bamboo frame |
| | | In-between layer | Boulders |
| | | Stone size | 0.05– 0.10 m |
| | | Thickness | min 0.10 m |
| Falling apron | Levels, dimensions & composition | Apron setting level | PWD – 15.00 m |
| | | Width | 15 m |
| | | Cover layer | Broken rock |
| | | Stone size | 0.23 – 0.37 m |
| | | Quantity | 35 $m^3$/ m |

## 3 TESTING OF DESIGN BOUNDARY CONDITIONS

For the design of the different structures of the RTW assumptions were made as to the design boundary conditions. Usually typical boundary conditions for bank protection structures are maximum water levels, maximum scour levels, flow velocities and soil data. In this particular case of guide bunds and hard points as part of the RTW for JMB some additional assumptions were made to arrive at the design of these structures, notably maximum outflanking upstream and the type of attack which the structures might experience. This is important in this case because the upstream conditions were not fully controlled and the river still has a considerable degree of freedom left.

In this Chapter the focus will be on the conditions as experienced during the post-construction period (1997 - 2009) in relation to the assumed boundary conditions as used for the design (1990) and the redesign (1994 - 1996). In this Chapter only the most important design boundary conditions are considered, notably (i) predicted and observed planforms, (ii) attack on RTW and bridge piles; and (iii) scour depths. For the other design boundary conditions see MAUNSELL & AECOM (2010). A complicating factor is the fact that the design was done using probabilistic design methods, hence most values for design boundary conditions were given with an associated probability of occurrence, making it more difficult to decide whether in a particular case the assumed design boundary conditions are conform with the observations or whether there is a significant difference which cannot be attributed to the stochastic nature of the phenomenon. The comparison between assumed boundary condition and the boundary conditions which have occurred since 1997 is preceded by a short description of the "attack" of the river on the RTW structures, loosely called "the river regime".

*River regime since construction*

In a very dynamic river like the braided Jamuna River the "attack" of the river is determined by at least two factors, notably (i) the fluctuating (flood) discharge and (ii) the continuously changing planform of the river. A serious attack does not necessarily occur during extreme flood conditions, but may be a combination of an unfavourable river planform combined with a not too high flood. This aspect has been duly incorporated in the design of the JMB by applying probabilistic design methods.

So when discussing the river regime which is synonymous for the attack by the river on the RTW, these two parameters have to be assessed. Water levels at the bridge site corresponding to characteristics floods are:

- Average flood 65,000 $m^3/s$ PWD + 13.66 m
- 1:10 year flood 76,000 $m^3/s$ PWD + 14.29 m
- 1:100 year flood 91,000 $m^3/s$ PWD + 15.08 m

A summary of the hydrological information in the period after construction of Jamuna Bridge is provided in Figure 5, which gives maximum and minimum stages at the EGB for each hydrological year (which in Bangladesh runs from 1 April through 31 March). Except for the 1998 flood, in the period since 1997 the maximum flood level was always below the 1:10 year flood and in many years even the average flood conditions were not reached.

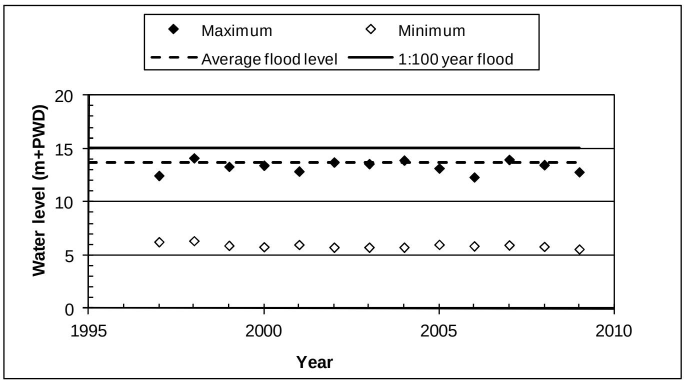



Figure 6: Yearly maximum and minimum stages at EGB in period after construction of Jamuna Bridge

Also the river planform near the bridge in the period 1997 – 2009 was studied. Satellite images were analyzed as to the attack on the guide bunds. In particular outflanking was studied, because outflanking was responsible for the deepest scour. Whether outflanking is important and could cause a threat to the RTW is probably determined by two aspects:

- the importance of the outflanking channel expressed in its relative contribution to the total discharge of the Jamuna River during flood;
- the extent of outflanking defined as the distance between the maximum bank erosion and line corresponding to the direction of the guide bund.

Figure 7 compares for the WGB the conditions in the river since the construction of the RTW with the results of the overall mobile bed and bank model.

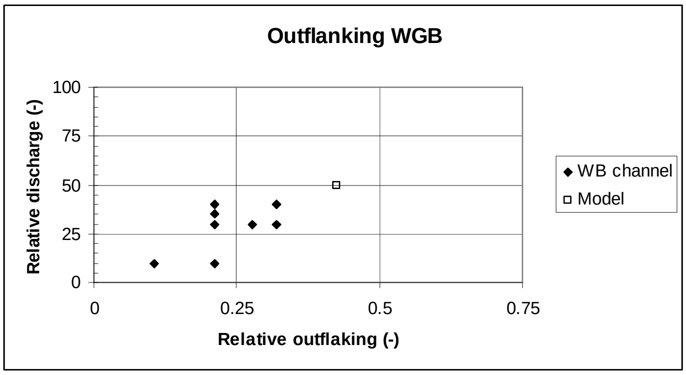



Figure 7: Observed relative discharge and relative outflanking in period 1997-2009 for the WGB compared to observations in the overall mobile bed and bank model

For the WGB until now the prototype conditions have always been less severe than in the overall model. A similar conclusion applies for the EGB.

Considering now both the yearly flood discharge and the yearly river planform in conjunction, it appears that in the period since the completion of the RTW in 1997 no serious combination of an unfavourable planform and an extreme flood has occurred. Also the outflanking has not been excessive. So the RTW have not been put seriously at test until now. In the future more serious river attack might occur.

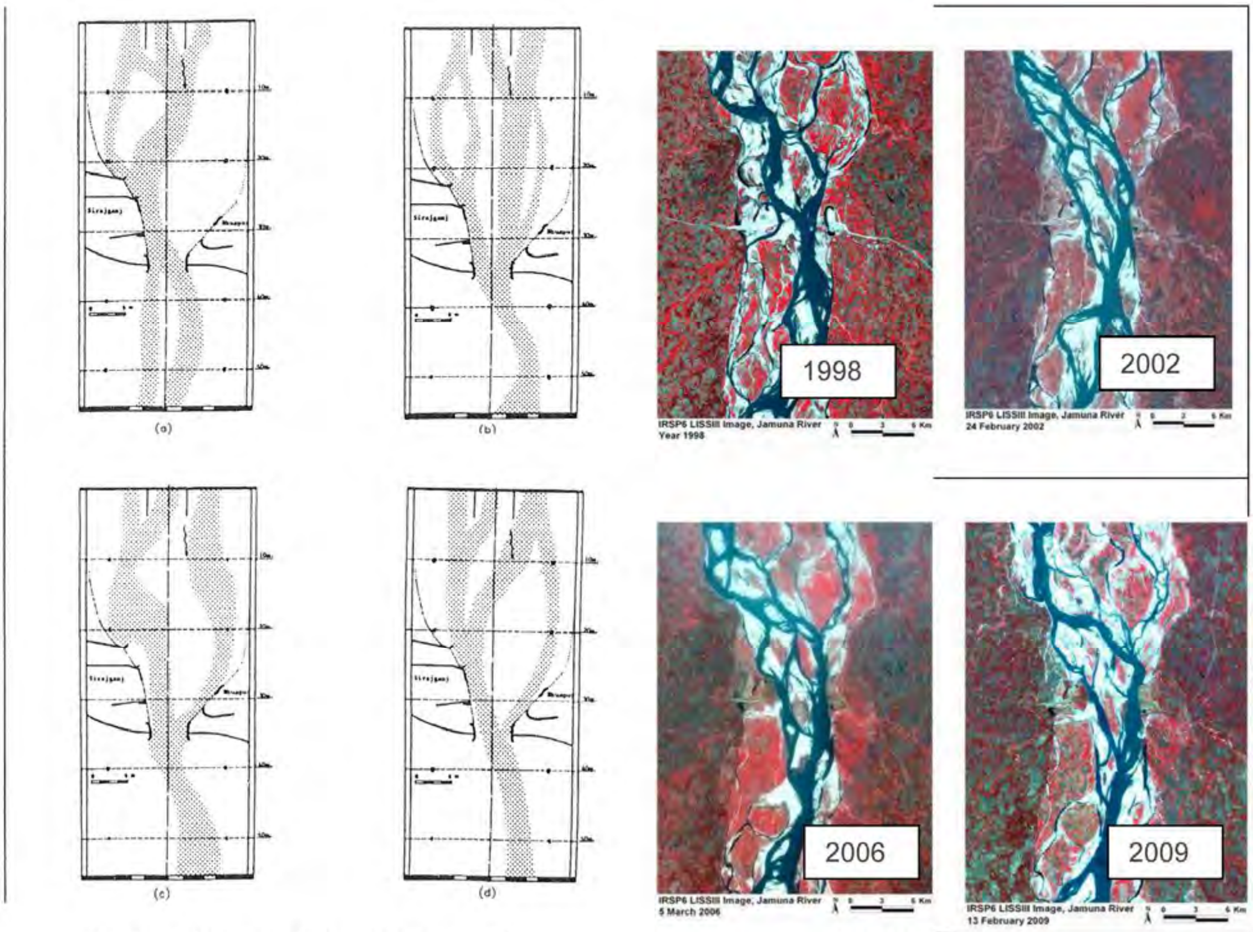
*Comparison predicted and observed planforms*

The main purpose of the overall mobile bed and bank model was to study the interaction between the braided Jamuna River and the RTW. The scale model was only a simplified representation of the real river, because building a scale model at reduced scale is always subject to scale effects.

Figure 8 presents planform observations in the model together with some satellite images from a few years. The following observations can be made:

- After some initial years with one central channel, the conditions in nature are characterised by two channels flowing next to the guide bunds; in nature the guide bunds seem to attract the channels. In the model this was less pronounced.
- In the model it was often observed that the two main channels formed a major confluence within the reach between the guide bunds where the bridge piers were present. In nature such a major confluence of the two major channels flowing along the guide bunds happens only more downstream of the guide bunds.
- In nature an oblique channel attacking the more downstream part of the guide bund occurs only occasionally. In the later years, such an attack was always due to a small channel and often combined with a confluence with the channel flowing parallel to the guide bund with minor confluence scour.
- In the model substantial outflanking did occur. In nature such extreme outflanking has not been observed, apart from the small channel upstream of the EGB.

Summarizing it can be stated that, although there is some similarity between the conditions found in the model and in nature, in particular the occurrence of two channels parallel to the guide bunds seem to be more pronounced than in the model. Hence also their confluence is not near the bridge piere but more downstream, which alleviates the conditions for the bridge piers in the middle of the bridge opening.




(a) Overall mobile bed and bank model (b) Satellite images

Figure 8: Comparison of planforms from model and as observed in nature

*Attack on guide bunds, approach roads and bridge piers*

The results of the overall mobile bed and bank model were used to develop scenario's as to possible attack of the river on the RTW, the approach roads and the bridge piles. In view of the possible scale effects also satellite images were used to study bends and curvatures in the prototype.

From observations in the overall model three types of attack on the guide bunds were identified (Klaassen, 1992), notably (i) *oblique attack*, for which it was supposed that it could happen anywhere along the total length of the guide bund, (ii) *frontal attack*; and (iii) *attack by an outflanking channel*.

The following observations can be made based on the observed attacks on the basis of satellite images in the period since 1997:

- oblique attack occurred in particular in the first years; later it became less pronounced;
- frontal attack is quite common over the last 10 years or so;
- attack by an outflanking channel has occurred during some years but both the extent of outflanking and the relative importance of the outflanking channel has been smaller than anticipated during the design.

The possible attack on the approach roads has been an important criterion for the determination of the upstream length of the guide bunds. From the overall model investigation and the additional study carried out, a cutoff criterion was identified (cutoff ratio $\lambda$ never larger than 1.7) which allowed to develop a design methodology for the minimum upstream length of the guide bund. Because outflanking has been minor since 1997, also an attack on the bridge approach roads has been absent. Hence it is not possible to check whether this cutoff criterion is appropriate.

For the design of the pier length an estimate was made of the maximum scour depths based on a combination of constriction scour, confluence scour and local scour around the piers. The inclusion of confluence scour in the scour assessment, was based on observations in the overall model.

During the review it was attempted to compare the model results with satellite imagery and field observations. Before already some remarks were made on the occurrence (or rather absence) of a confluence near the bridge piers between the guide bunds. Apart from the satellite images more detailed information is available from three bathymetric low-flow surveys carried out by IWM (Institute for Water Modelling). Inspection of these bathymetric surveys supported the earlier statement that the overall picture is two channels running parallel along the guide bunds. The conclusion is that near the bridge a minor confluence and associated scour might be present but probably with less deep scour than anticipated during the design of the bridge piers.

*Design scour depths*

Extreme scour is a typical feature of the Jamuna: river branches flow in thick, loosely packed, alluvial depositions of, predominantly silty sand. The ultimate scour is the result of the combination of the different types of scour (see Breusers & Raudkivi, 1991). Local scour seems to be dominant over all other forms of scour; however it should be realised that the local scour is a function of the "initial" depth, which in turn is a function of scour in an outer bend and constriction scour.

The scour to be expected along a guide bund was considered to be of the order of 40 m, relative to the level of the flood plain (PWD + 12 m). But, in order to limit construction costs probabilistic design methods were used to determine maximum permissible scour depths for which the design would be made. The result is shown in Table 2 for a slope of 1V:3.5H. The first two columns are copied from the 1990 design report (RPT et al, 1990a). The next two columns have been added using a method to determine the probability of exceedance in a certain period (see Jansen et al, 1979 or TAC, 2001).

Table 2: Design scour depth guide bunds for 1V:3.5H slope

| Scour level (m + PWD) | Probability of exceedance per year | | |
|---|---|---|---|
| | Per year (-) | During life time of 100 years (%) | In 10 years (%) |
| - 28 | 0.050 | 99 | 40 |
| - 30 | 0.023 | 90 | 21 |
| - 32 | 0.008 | 55 | 8 |
| - 34 | 0.002 | 18 | 2 |

The actual slope under which the guide bunds were constructed is however 1V:5H and 1V:6H. Hence it was attempted to estimate this reduction in local scour. Although the results are not conclusive a first estimate of the reduction in local scour is in the order of 5 m. In that case scour levels of about PWD - 29 m would probably have the same probability of exceedance as assigned in Table 2 for a scour depth of PWD – 34 m.

In the period 1997-2009 regular soundings of the scour depths were done with echosounding equipment and proper positioning methods. When needed the soundings were done very frequently (every few days) and detailed (every 20 m). These soundings can be used to evaluate the actual scour depths observed. Some results are presented in the Figures 9 and 10. See also Marga-Net (2006).

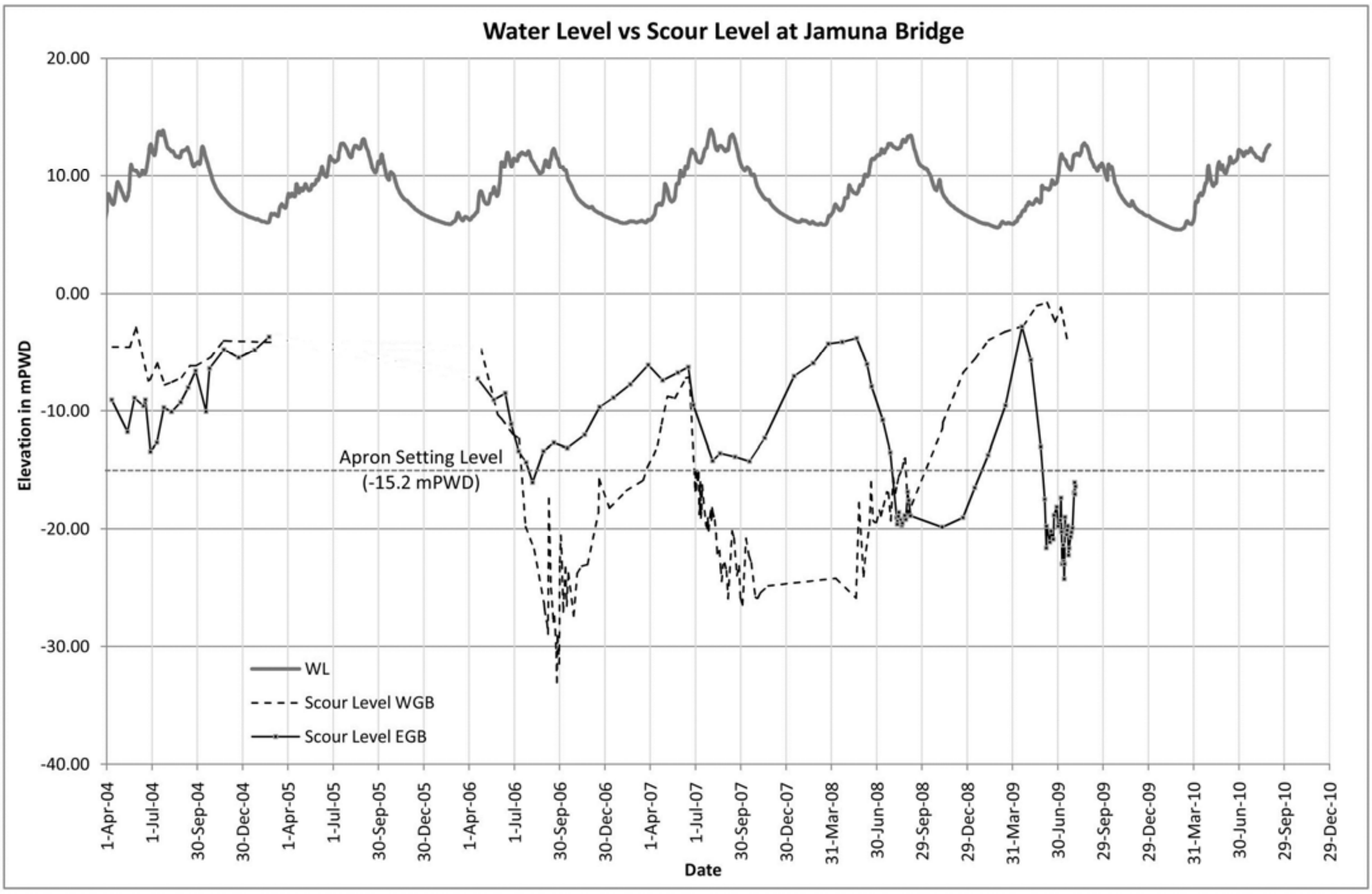
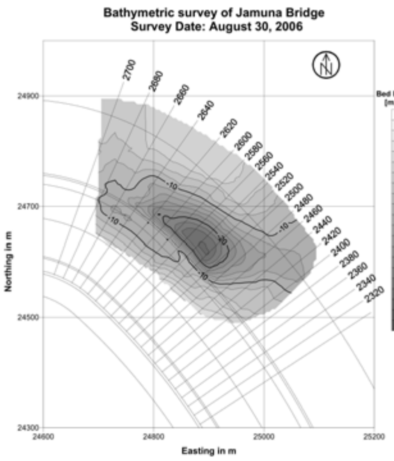
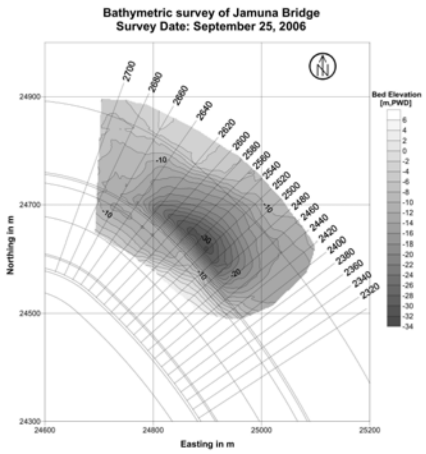


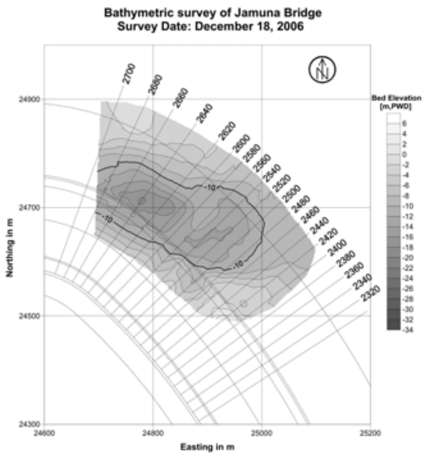

Figure 9: Maximum scour depths near WGB and EGB and water level as measured at the EGB in the period 2004-2009




(a) 30-8-2006 stage PWD + 10.82 m, deepest scour PWD -25m




(b) 25-9-2006 stage PWD + 11.53 m, deepest scour PWD -33 m




(c) 18-12-2006 stage PWD + 6.85 m, deepest scour PWD -18m

Figure 10: Development over time of scour hole near WGB in the year 2006

Figure 9 presents the maximum scour depth measured in the period 2004 through 2009. As can be noticed in particular in the year 2006 the observed scour near the WGB reached more than PWD -30 m. Figure 10 shows the extent and the time-wise development (via date and stage) of this very deep scour hole. It occurs in particular during the flood season and is located near the falling apron. During the subsequent recession the scour hole is filled in. Also it is important to notice that the length of the scour hole below PWD -15 m is only about 200 m, hence the scour is indeed very localized. The maximum scour depth corresponds toPWD -33 m. This is deeper than the predicted scour depth according to Table 2 when corrected for the more gentle slope under which the guide bund was constructed. Hence the design scour depth is exceeded in a year with below average flood, be it over a very short length. The year 2006 though is characterized by substantial outflanking. In the years 2008 and 2009 also deep scour holes below PWD -15 m were observed near the EGB (see Figure 9), and these again correspond to outflanking conditions.

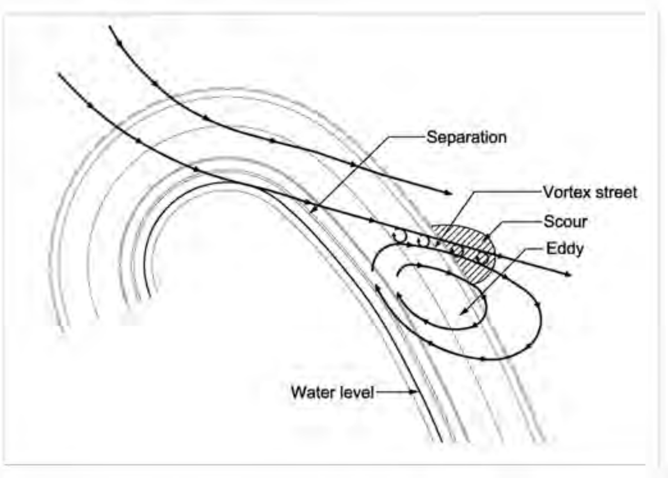



Figure 11 Possible flow pattern at head of guide bund during outflanking

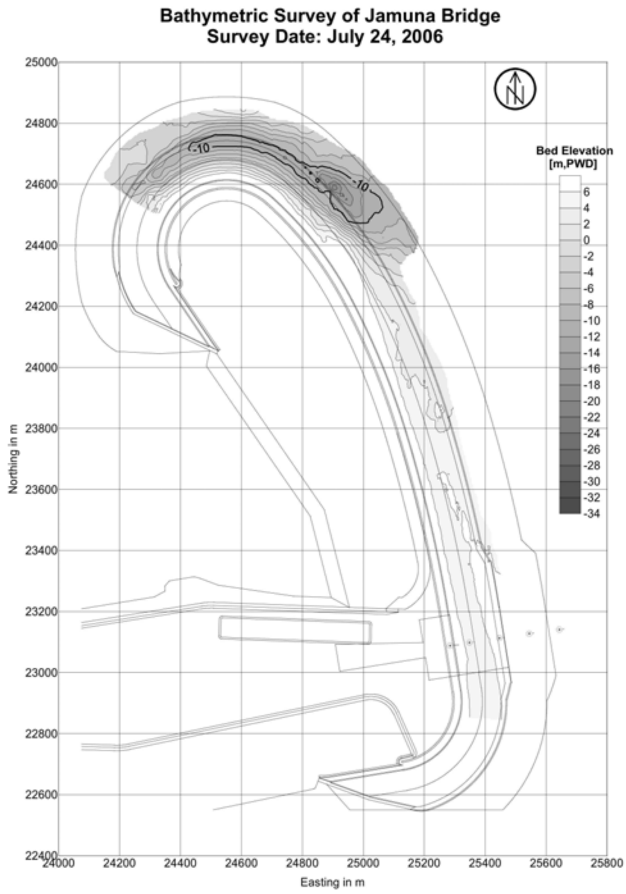



Figure 12 Sounding of scour and deposition along WGB on 24 June 2006

Different possible explanations for the deep scour hole observed in 2006 near the WGB were considered. Several witnesses which observed the flow pattern at the head of the guide bund during flood conditions reported a very strong eddy downstream of the head of the guide bund. Such a strong eddy would indicate that separation takes place. The eddy is driven by the main flow. At the interface between main flow and eddy a vortex street will develop (see Figure 11). Vortex streets generate high turbulence levels, this might have increased the vertical diffusion and this, in combination with uplift under these vortices, might have lifted the bed material into suspension. This could explain the very deep scour holes and the steep slopes. It also explains why the deep scour hole is observed near the head of the guide bund and why immediately downstream sand deposits occur: in the eddy zone sediment will deposit.

The implication would be that when during outflanking conditions a very serious flood would occur with even larger velocities and a stronger vortex street, at the head of the guide bunds the design scour depths might be exceeded even more

Until now only the scour at the head of the guide bund was discussed. During the design it was assumed that the design scours deeper than PWD -30 m could occur along the whole length of the guide bund. However, Figure 12 shows that deep scour occurs only at the head and more downstream the guide bunds are even covered by sand deposits. This continues to be the case during floods. Probably the more downstream parts of the guide bunds are overdesigned and a slope protection could have been shorter and/or the falling apron could have been placed at a higher level and contain less rock.

## 4 PERFORMANCE REVIEW OF GUIDE BUNDS

In this Chapter the performance of the river training works is reviewed. Because in the preceding Chapters already the overall layout and the planform of the guide bunds is discussed, the topic here is in particular the slope protection. Limitations of space allow only a brief discussion of the most salient findings. For more details MAUNSELL/AECOM (2010) should be consulted.

The slope protection of the guide bunds consists of three parts (see Table 1and Figure 4), notably (i) an open stone asphalt (OSA) part (above PWD + 10m), (ii) the rock slope protection (SP) (between PWD +10 and -15 m), (iii) falling apron (FA) placed at PWD -15 m and supposed to take care of the deepest scour). Near the bridge proper the levels are slightly different (-18 m instead of -15 m for the falling apron setting level). During the review period the OSA experienced some minor damage which was subsequently repaired and which is not discussed here.

With regard to the SP from 2006 onwards two different problems have occurred which may or may not be linked. It concerns the shifting of rock in downward direction exposing at some places the geotextile filter and the formation of one localized hole in the lower part of the slope protection. The shifting of the rock on the guide bunds slope is shown in the two photographs in Figure 13. The cause of the shifting was studied on the basis of the limited information available. The most plausible explanation is the occurrence of flow slides of the sand deposits on the guide bunds during receding flows.

(a) Slope SP WGB below berm without damage

(b) Damaged section with shifted rock with exposed geotextile and some sand deposits

Figure 13: Original and damaged section of slope protection (SP) of WGB

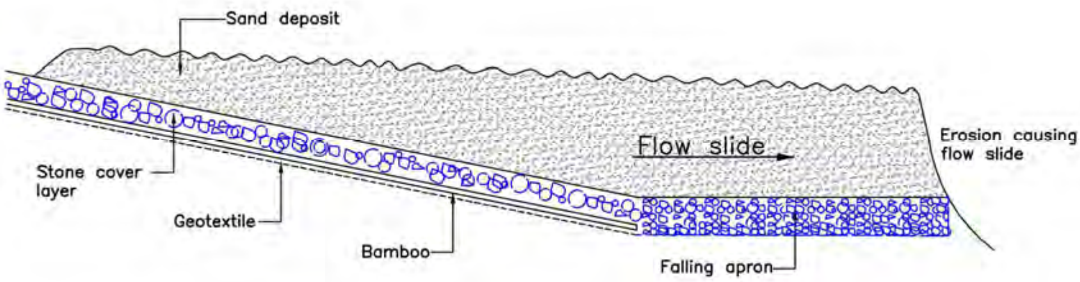



Figure 14 Flow slide of sand deposits on top of slope protection (SP) as possible cause of shifting of rock

Such a flow slide in the mass of sand and silt overlying the SP can occur due to the lowering of the water levels during the end of the flood season and/or the erosion of the sand deposit in a later stage. The geotextile sheet could function as a kind of sliding plane for the mass of rock, boulders and sand and silt, because (i) the relatively thin rock layer is embedded in silt and sand, (ii) the presence of a thin layer of boulders placed between angular rock and geotextile to prevent punctures in the latter during rock dumping, (iii) the bamboos on the geotextile by now have partly deteriorated and (iv) the friction between the rock and the geotextile is small (see Figure 14). Other explanations (wave action, theft) cannot be ruled out and more detailed studies are needed to better understand this shifting, which by the way did occur at a substantial scale.

During the 2007 flood season it was found that 'holes' started to appear in the SP of WGB between chainages 2420 and 2580, i.e. over 160 m along the guide bund at the bottom end of the slope between the as-built rock levels PWD – 8 and PWD - 10 m. The initial dips, developing rapidly into gullies, ultimately became locally deeper than 5 metres while the overall length of SP affected (measured up the slope) could locally be as much as 35 m (out of 135 m). The local lowering of the SP is illustrated in Figure 15, showing bed level changes in 2007 compared to the as-built conditions. In MAUNSELL/AECOM (2010) a careful and detailed analysis of all available data is presented to arrive at an explanation of these holes. Possible explanations are linked to a geotechnical failure of the falling apron (as it concerns the same reach where very deep scour holes were observed, see Chapter 3) and/or construction problems related to the mattresses. During the

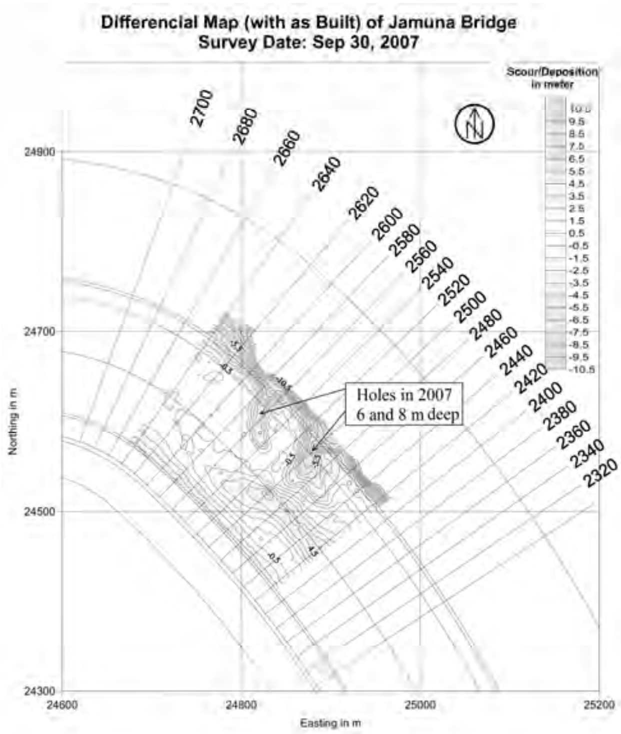



Figure 15: Holes in the slope protection in 2007 near reach where falling apron has vanished

deployment process the falling apron reached slope angles substantially steeper than 1V:2H, which is geotechnically problematic. These steep slopes were unexpected as commonly rock is believed to launch at angles of 1V:2H.

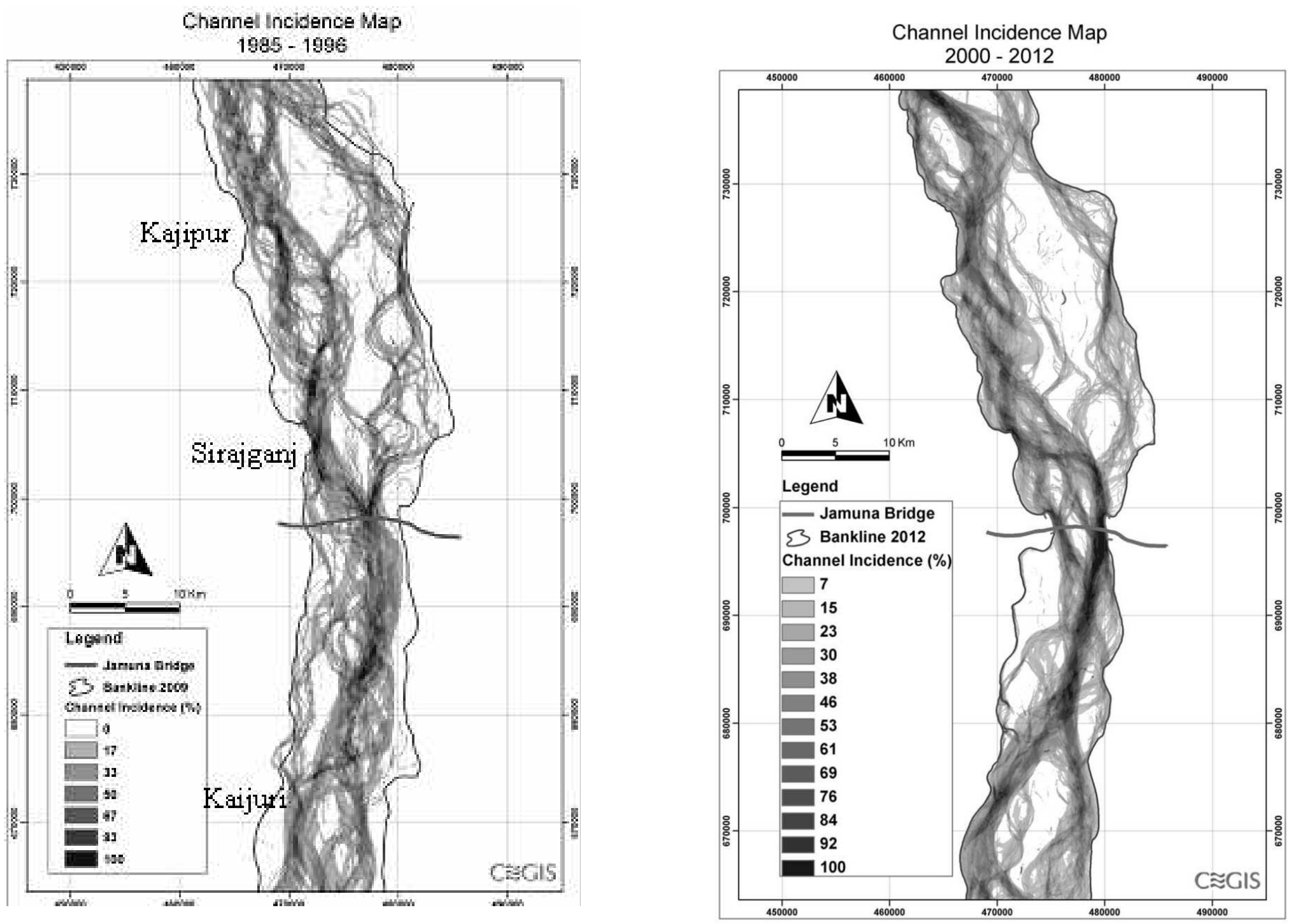
## 5 INTERACTION RTW AND RIVER AND DOWNSTREAM IMPACT

As discussed in Chapter 2, an overall mobile bed and bank model was used to study the interaction between the river training works and the braided river. One result from this study were the possible angles of attack of the flow on the guide bund heads (see Chapter 3). Another item of interest is the channel pattern and possible attack on guide bunds and piers in the reach where the river was flowing in between the guide bunds. Although not specifically designed to reproduce this aspect correctly, still the results were used as part of the probabilistic design.

Satellite images of the low flow season since the bridge was constructed allow to study the impact of the river training works on the braiding pattern and the downstream impact of the RTW. This was also studied during the review of the RTW (see MAUNSELL/AECOM, 2010); a summary of the impact on the downstream reach can be found in Sarker et al (2009). Figure 16 shows the incidence of channels (via the frequency that a particular part of the river a deep channel is present) before (a) and after (b) the construction of the bridge. Figure 16b is an updated version of Figure 3(c) of Sarker et al (2011), by including three more years (2010, 2011 and 2012) into the analysis.

The following observations are made on the basis of a comparison of the incidence maps and the results of the overall mobile bed and bank model

- The Jamuna Bridge RTW have stabilized the channel pattern slightly upstream in between and downstream of the guide bunds.
- During most years since construction two channels are present, one along the WGB and the other one along the EGB. These two channels confluence a few km downstream of the guide bunds.
- Downstream of the WGB a considerable area of former char land is not subject to erosion any more. Also more downstream over a distance of about 20 km  newly formed land along the right bank of he river seems to remain in a stable position, whereas more downstream a bifurcation has created a system with two anabranches. This system is still widening.




(a) Pre-bridge conditions (1985-1996)

(b) Post-bridge conditions (2000-2012)

Figure 16: Channel incidence, showing probability that a deep channel is present at a particular location in the channel beld

- The downstream impact of the Jamuna Bridge RTW was not extensively studied in the overall mobile bed and bank model, but Figure 8(a) suggests that in the model the stabilization was less than observed in nature until now.
- The observed stabilization might be partly due to the eccentric position of the guide bunds in the river. This seems to support the theory that the Jamuna River can be stabilized via a number of fixed hard points at strategically selected distances (see Klaassen, 2009 for a proposal to stabilize the Jamuna River over its full length in Bangladesh).

## 6 DISCUSSION

The present review has uncovered some intriguing findings which either were not known during the design of the Jamuna Bridge or where model studies carried out during the design were showing different phenomena than later observed in nature. The review findings have been quite useful for the design of the even larger Padma Bridge. In general it is recommendable to carry out such a review, in particular when the design and construction of the concerned structures can not be based on existing experience and requires "out-of-the-box" thinking.

The main conclusions from the present review are presented and discussed hereafter:

- Since their construction the Jamuna Bridge RTW have performed quite well; no major damage has occurred and on the average the costs of yearly maintenance amounted to about 0.13% of the initial costs only.
- In retrospect it appears that the hybrid modeling approach (comprising of a combination of scale and numerical models, the analysis of satellite images and the field surveys) has provided solid basic information for the design.
- The guide bunds appear to "attract" the Jamuna channels which do not join inside the reach between the guide bunds; hence confluence scour is not a dominant phenomena for the design of piers and guide bunds. The functioning of major bank protection works as "attractors"is already known from Sirjaganj and Chandpur and is also discussed in the literature (Mosselman & Sloff, 2002). The upstream heads of the guide bunds of bridges crossing major braided rivers are the parts which suffer the most serious attack and sufficient effort should be made to properly optimize their design.
- The occurrence of a very deep scour hole at the guide bund head next to the WGB during moderate attack in 2006 is worrying; its cause should be studied in more detail, using scale models, possibly advanced mathematical models and field measurements. Possible counter-measures (see e.g. FAP21/22, 1996 for some ideas) should be considered. If the occurrence of a vortex street is the responsible phenomenon for the deep scour holes observed, then the question rises why this deep scour hole was not observed in the local scour model, which was applied in modelling approach as described in Chapter 2. In the local scour model sand in size comparable to the sediment in the Jamuna River was used (uniform and $D_{50}$ of about 0.2 mm). As could be shown scale effects prevent that this material goes into suspension, and hence the sucking up of sediment did not occur in the local scour model. In retrospect it would have been better to use light weight material in this local scour model.
- Probably the more downstream parts of the guide bunds are less seriously attacked than assumed earlier and their design can be lighter. A similar phenomena was observed in the overall bed and bank material, but this was attributed to scale effects.
- Strengthening of falling aprons after launching, as required according to recent findings (Oberhagemann et al, 2006 & 2008), is difficult to carry out in the case of a constricted reach like between the guide bunds. During the flood season hydraulic conditions make these operations dangerous, whereas during low flow conditions the launched falling aprons are covered by sand deposits.
- The cause of the shifting of the protective layer on the guide bunds, leaving the geotextile filter exposed at some places, is not fully understood and requires further studies. For the time being only timely maintenance can prevent subsequent damage which ultimately might result in the failure of the guide bunds.
- The downstream impact of the Jamuna Bridge is a stabilization of the channel pattern, which might be more pronounced due to the asymmetric location of the RTW in the river.

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