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A CASE STUDY ON ONE OF THE SKEWED REINFORCED CEMENT CONCRETE BRIDGES IN INDIA

Vithal H. Jadhav

Civil Engineering Dept. Faculty, K.L.E Technological University, Hubballi, Karnataka, India
e-mail: vithaljadhavh@gmail.com

ABSTRACT: This paper pertains to the case study on a skewed Reinforced Cement Concrete deck Slab Bridge constructed at km 5.300 on Gotogali Goyar major district road in Karwar Taluka of Uttar Kannada District in India. The work pertains to the Karnataka Public Works, Ports and Inland Water Department. The author was working as an Assistant Engineer in the same department in design section. The author designed this bridge and had issued the design details along with working design drawings to the field engineers for execution. The location of the bridge is in a thick forest area under wildlife protection zone. In the design of a bridge, it is always preferred to fix the alignment of the bridge in such a way that it should make a right angle to the river flow which helps in easy flow of water and gives clear vision for traffic movement. But sometimes due to restrictions at site and other local obstructions, it may not be possible to fix the alignment of the bridge at right angles to the flow of the river. In such cases, the bridge has to be designed as a skewed bridge. The inclination of the centre line of traffic (road) to the normal to the centre line of the river in case of a river bridge or other corresponding obstruction is called the skew angle [Fig-1]. In the instant case as per the wildlife zonal regulations, there was an obstruction from the forest department to fix the alignment of the said bridge at right angles to the flow of the river by acquiring forest land. So the author designed the said bridge with 30o skew angle as per the detailed survey and the design details and drawings were issued to the field engineers for execution. Unfortunately, the bridge executed by the field engineers is in the opposite direction of the actual skew. This was observed by the concerned higher authorities during their inspection and brought to the notice of the designer (author) who was retired, asking him for the remedial measures for the safety of the bridge. By the time designer inspected the bridge the major portion of the bridge was already executed and the only choice left with the designer was to suggest the remedial and protection measures against the hydrodynamic effects of eddies on substructures and foundation. The designer suggested some protection measures after inspecting and studying technical aspects. So the author has taken it as an opportunity to

have a case study on the said skewed bridge to throw light on the protection measures taken up and to analyse the performance of the bridge.

KEYWORDS: skewed Bridge, bridge alignment, cavity formation, eddy currents, bridge protection, scour depth,

1 INTRODUCTION

The bridges are mainly classified as Culverts, Small bridges and Major bridges Clause 101.1 [1]

Culverts: These are the having linear waterway less than 6.0m
Minor bridges: these are the cross drainage structures having linear waterway above 6.0m and up to 60.0m.
Major bridges: These are the cross drainage structures having a total length above 60.0m.
Based on the alignment of the bridges they are classified as straight (right) angled bridges and skewed bridges.
Before going for the detailed design of the bridge, it emphasized on the comparison between straight angled bridge v/s skewed bridges.

Table 1. Comparison of straight angled bridges v/s skewed bridges

Straight angled bridge	Skewed bridge
1. Economical	Uneconomical
2.Ease of construction	Require more skill and knowledge
3. No eddy currents formation	Eddies may form and affect the substructures and foundation.
3. No eddy currents formation	Eddies may form and affect the substructures and foundation.
4. Design is simple	Design involves complications..
5. Vision is clear for traffic	Comparatively, vision is not clear for traffic
6. Stream/river training may not be required	Stream/ river training may be required to make the water to flow smoothly.
7. Protection of bunds generally not required.	Protection of bund may be required to safeguard the buds from eddies.
8. Span length, deck area, length of Abutments and piers will economic..	Span length, deck area, length of Abutments and piers will increase in proportion to cosec (θ) where θ is the skew angle.
9.The stress slab will be normal.	The stresses in the skew slab will differ significantly from those of straight slab.
10. There will be no uplift of corners in the straight slab.	The reactions at the obtuse-angled end of the slab supports are larger than the other ends. The bearing reactions tend to change uplift in the acute angle corners with an increase in skew angle. Hence require additional reinforcement at these corners to counteract the uplift.

Straight angled bridge	Skewed bridge
11. No extra edge bars are required as the main bars are having the bearing at supports.	Bottom reinforcement placed perpendicular and parallel to the supports cut the free edge at an angle is ineffective in resisting the B.M at the centre of the free edge. So extra steel rods are provided at edges for anchorage.

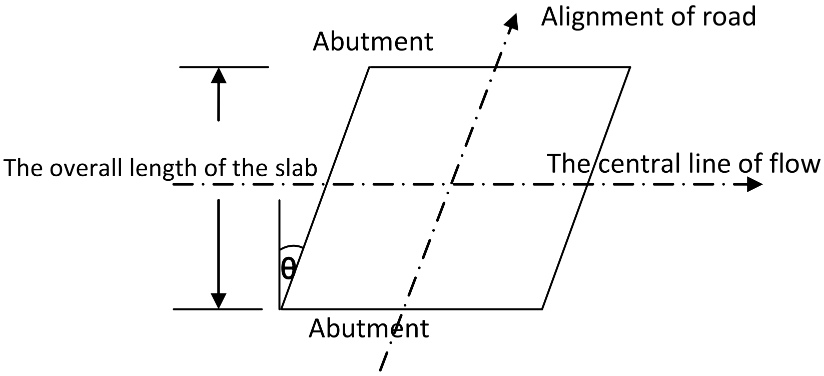

Figure 1. Skew angle θ is shown

There was a need of the local public to construct a bridge at km 5.30 on Gotogali Goyar major district road in Karwar Taluka of Uttar Kannada District in India across the local stream. The location of the bridge site and the catchment area on the upstream (U/s) of the bridge is coming under a thick wildlife forest zone. A preliminary and detailed survey of the bridge site was done. The details of the survey are tabulated below in [Table-2], [Table-3] & [Table-4].

Table 2. Field investigation details of the bridge site

Name of the stream	Local stream.
The surface of the natural stream	Clear, Skew bank, no rift or deep pools
Any obstructions in the stream	No obstructions are seen.
Banks	Undefined skew banks
The angle of repose ϕ =	30^o
Liability of the site for any earthquake disturbance.	Earthquake disturbance is predicted as it comes under the coastal zone.
Class of road	Major District Road. (M.D.R)
Traffic interruption on road	More than 2 hours during flood.
Traffic interruption on road	More than 2 hours during flood.
If the scour depth has been observed the description or any other special causes responsible for the same.	No deep scour depth is observed at the site. But calculated by the rational formula.

Table 3. Soil investigation details of the bridge site

Unit weight of soil under saturated condition	18 k N/m^3
S.B.C soil at trial pits	200N/m^2
Soil exploration as per trial pit	Up to 2.0m from average G.L Sand with medium clay. (Taken at banks due to water stagnation in the stream)
Lacy's silt factor (K_{sf}) For Standard silt [1]	1.00
Width of the gorge portion	70.00m
Any bridge in the near vicinity for the same stream either on U/s or D/s	No bridge in the near vicinity.
Nature of catchment area	Thick forest under wildlife protection zone. (afforestation)
Probability of large trees or rolling debris floating down the stream.	Expected about 6 to 7.0m (21 feet) large trees or debris.
Need for large scale stream training	Predicted as the bridge is to be designed as a skewed bridge.
Approaches	Both approaches are with hard soil.

Table 4. Hydraulic survey details of the bridge

Catchment area	35.000 km^2 (Computed from topography sheets)
O.H.F.L (Observed High Flood Level)	98.500 R.L m (on local enquiry) based on the floods in the last 5 decades. Surveyed with the arbitrary benchmark.
L.W.L (Low Water Level)	96.000 R.L m (noted during summer)
Skew angle	30^o (As per survey)
Bed slope	1:280 (based on the longitudinal section)
Tidal effect	No tidal effects.
Maximum flood depth at HFL	2.0 m

2 DESIGN OF BRIDGE

2.1 Hydraulic design of the bridge

a) Discharge by empirical Ryve's formula $Q=CM^{2/3}$ applicable for south Indian region. Where Q- Discharge in m^3/sec.
C- Ryve's constant, M-Catchment area in km^2.

"C" value for 35.00 km^2 catchment area is = 11.25

$$\text{Therefore } Q = 120.37\text{m}^3/\text{sec} \quad (1)$$

b) Discharge by Area Velocity method.
The cross-sectional wetted area of the stream
A = 119.475m^2.
Bed slope (hydraulic gradient) (S) = 1: 280.
Wetted perimeter (P) = 110.451
Coefficient of Rugosity (n) = 0.035 Ref [2] Table 2.1
Hydraulic mean depth R = A/P = 1.082 (2)

Velocity by Manning's formula $v = (1/n) * (R)^{2/3}*(S)^{1/2}$

Therefore $V = 1.80$m/sec (3)

Discharge $Q = A*V = 214.966\ m^3/sec$ (4)

Design discharge: The design discharge is taken as the maximum value of the above equations (1) and (4). If the value so obtained exceeds the next high value by more than 50%, then the maximum discharge is limited to 1.50 times the lower estimate [5].

So the design discharge in the instant case is

$$Q_{dgn}= 120.37 * 1.50 = 180.56\ m^3/sec \quad (5)$$

Linear waterway W = A/D where A is Cross-sectional area of stream and D is maximum flood depth at H.F.L.

Using Lacey's equation for effective linear waterway for undefined banks.

$$W=C\sqrt{Q} \quad (6)$$

Where W = Effective linear waterway in m.

Q = Maximum designed discharge in m^3/sec.

C = A constant usually taken as 4.8 for regime channels, but may vary from 4.50 to 6.30 according to local condition [3].

Using equation (6) $W = 4.50*\sqrt{180.56} = 60.468$ m

Considering the economic span of 8.0m, no of spans = 60.468/8= 7.558 Say 7 spans of 8.0 clear (excluding the obstructions).

So it was proposed to adopt 7 spans of 8.0 m clear with the skew angle of 30^o with two lanes i.e., 7.50 m carriageway for M.D.R [2].

Scour depth calculations: As the linear waterway is less than 60.00m,

The mean scour depth $d_{sm} = 0.473*(Q_{dgn}/k_{sf})^{1/3}$ (7)

Using equation (7) $D_{sm} = 2.67$m.

Maximum scour depth near abutment = $1.27*d_{sm} = 3.40$m. This is below H.F.L

Maximum scour depth in the vicinity of pier = $2.0 *d_{sm} = 6.70$ m. This is below H.F.L.

Founding level of abutment = H.F.L – 3.40 = 95.10 R.L

Founding level of pier = H.F.L – 6.70 = 91.71 R.L

The depth of foundation for abutment below L.W.L = 96.00 - 95.10 = 0.90m

The depth of foundation for pier below L.W.L = 96.00 - 91.71 = 4.29m.

But it is to note that the soil investigation was done at bund sides and the properties of hard soil were considered in the hydraulic design. But in due course, it was asked the field engineers to investigate soil properties all along the bridge alignment. Then the following results were obtained.

It was confirmed at the site that for an average depth from natural bed level up to 0.30m to 0.60m hard soil, 0.60m up to 1.00m exposed ordinary rock and below 1.0m hard rock was met. According to IRC, the foundation levels were to be fixed to 1.50m below the L.W.L. in ordinary rock and 0.60m in hard rock.

2.2 Design of substructure and foundation

2.2.1 Design of abutment

Safe Bearing. Capacity considered as per test result

196 kN/m^2 The abutment has been designed for the following loads and forces. Class AA loading and 70R load.

a. Vertical load = Dead load + Live load

b. Horizontal load due to frictional resistance of braking force at the bearing.

c. Breaking force due to L.L (20%).

d. Load due to differential bearing pressure.

e. Surcharge due to L.L on backfill is negligible as R.C.C approach slab is provided.

f. Passive earth pressure at waterside up to L.W.L

g. Active earth pressure at the backfill of the abutment.

h. The abutment is designed with slab load and without slab load conditions and the results are as below.

P_{max} and P_{min} are within the S.B.C hence no negative pressure is developed.

The structure is safe against sliding and overturning.

2.2.2 Design of pier

Pier has been designed for the following loads

i) Dead load
ii) Live load
iii) Buoyancy effect
iv) Longitudinal force due to breaking force
v) Differential bearing pressure
vi) Wind Loads
vii) Water Current with skew = 30^o
viii) Earthquake forces. (Both vertical and horizontal)

Stresses P_{max} and P_{min} computed are beyond the limit stresses of 2.00T/m^2 for tension and 150.00T/m^2 for compression. So the pier section is designed as R.C.C pier for S.B.C 196kN/m^2, considering concrete grade M20 for the moderate condition of exposure for reinforced cement concrete for bridges [4] and Steel Fe415.

Effective depth (width) of pier required is 741.54MM

However, overall depth provided is 850MM

Stress at the bottom is less than S.B.C hence footing is safe.

For footing depth, the effective depth required is 0.542m but overall depth provided is 0.60m hence ok.

2.3 Design of superstructure

2.3.1 Design pier and abutment caps

The pier cap width is fixed as 1000MM considering the bearing width of 0.37m and 0.02m expansion joint. The abutment cap width is fixed as 0.80m accommodating 0.37m bearing and 0.02m expansion joint.

Both pier and abutment caps are provided with steel considering 1.5 % of the cross-sectional area equally distributed on all the sides.

2.3.2 Design of dirt wall

The section of dirt wall is provided as 0.865m x 0.30m considering the depth of deck [3]. This section is designed for breaking force and backfills earth pressure.

2.3.3 Design of deck slab

Since deck slab designs are readily available, the details have been adopted for a skewed bridge with skew angle 30^o for a clear span of 8.0m with two-lane carriageway of 75.0m [2].

2.3.4 Design of bearing

Since span is less than 10.0m with deck (solid) no specific bearing is required hence Tar paper is adopted.

2.3.5 Levels and dimension of bridge components fixed as per the design

Table 5. Bridge details as per design

Components	Level / Dimensions
High flood level	98.500
Low water level	96.000
Minimum vertical clearance as per clause 106.2.1[1]	0.90m
Top of bridge deck including wearing the coat of 0.075m thick.	100.775 R.L.
Thickness of deck	0.700m
Top of bridge i.e., the bottom of the deck	100.00 R.L.
Width of steam of R.C.C pier	0.850m
The thickness of abutment and pier cap	0.30m
Abutment cap section	0.80m x 0.30m
Pier cap section	1.00m x 0.30m
Founding level of the abutment	94.000
Founding level of the pier	91.700
Carriageway	7.50m
Overall width (length of abutment)	8.40m
Height of handrail	1.00m.
Length of returns on both sides	5.00m
Ease water on U/s triangular with height equal to½ the width of steam of pier	0.85/2 =0.425m
Ease water on D/s semi circle with radius equal to ½ of the width of steam of pier	0.85/2 = 0.425m

3 EXECUTION OF THE BRIDGE

After the design was finalised by the author it was handed over to the field engineers with the detailed design drawings for the execution of the bridge. The field engineers executed the bridge. The commencement of the work was not brought to the notice of the designer. The major work was carried out and the higher authority planned of inspecting the work. During execution, the higher authority at Chief Engineer level had inspected the work and was shocked to know about the bridge was executed opposite to the actual skew mentioned in the design drawing. Then the matter was brought to the notice of the designer (author) and asked him to suggest remedial and protective measures for the safety of the bridge. By the time designer inspected the bridge execution of the major portion of the bridge i.e., 5 spans out of 7 were executed opposite to the actual skew and the remaining 2 spans were ready for deck concreting in the same direction. There was no chance left to the designer for suggesting any remedial measures except the protection measures.

3.1 Technical remarks made by the designer during the inspection of the work before monsoon, are as below

The foundation levels for abutments were suggested to fix at 0.90 m below the L.W.L but the founding levels are executed up to 2.0m below with dowel bars of 25 mm diameter M.S rods anchoring 1.50 inside the hard rock and 1.50 lengths inside the body of abutments. The safe foundation was adopted.

The foundation levels for the pier were suggested to fix at 4.29m below low water level as per design for hard soil strata. But as the ordinary rock was met with for the piers' foundation the depth was to be restricted to 1.50m below low water level. But the foundation levels were fixed up to 4.0m depth even removing hard rock beyond 2.50m by blasting. Foundation provided for piers in rock is much safer for piers.

Due to the skew provided opposite to the actual skew the water hits on the right side bund on the upstream side (U/s) at the entry of bridge and erodes the bund and deviates to enter the bridge vents. This may drastically affect the foundation by scouring and cavity formation in piers and abutments due to abrasion of flowing stones more drastically for 3 spans at the entry side. After the water enters the bridge vents it will create an impact on left side bund on the downstream side (D/s) and then follows flow of the natural stream. These predictions are shown in figure [2].

The 30^o skew angle adopted is not rallying with the actual skew as mentioned in the design drawing. Predominantly eddies are expected to form on the right side bund on the upstream side of the bridge and also on the left side bund on downstream of the bridge, Figure [2]

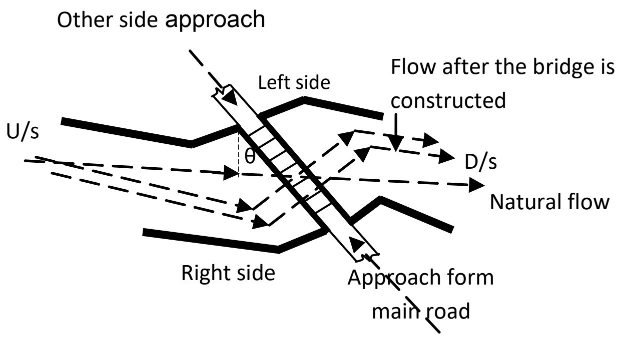

Figure 2. The skew and bridge against skew

4 STUDY ON TECHNICAL ASPECTS IN THE INSTANT CASE

A detailed study in the instant case has been made [5]. Based on the study it is arrived at the following technical remarks.

Erosion of foundation and substructures will be due to abrasion of suspended particles dragged by the velocity of the water. The rate of abrasion is dependent upon the quantity of sand and gravel in water, shape, size and hardness of particles and velocity of the current [5]. In the instant case, the exposed rock is ordinary and hard rock boulders. So it expected that abrasion will take place more predominantly at L.W.L and below.

There is a relationship between the size of particles drawn and velocity of water in the stream.

$$\text{i.e., } (d)= 36.15 \times Vb^2 \tag{8}$$

Where d-diameter of the particle in mm to be dragged by the flow of water & Vb is the velocity of water in m/sec in the stream.

Using equation (8) d=36.15x1.92 = 130.50mm.

The size of major part of the boulders found at site is less than 130.50mm hence it expected that the abrasion may take by the boulders lesser than 130.50 mm dia.

From durability consideration of the concrete for piers and abutments, the grade of concrete to be considered is M15 for the velocity of water up to 3m/sec. whereas the velocity of water in the stream under study is 1.9m/sec and the grade of concrete considered is M20 hence safe.

However, the protection needs to be given for piers and abutments at L.W.L and below, to avoid abrasion of substructures.

Discharge Q =A*V So V = Q/A. By increasing the cross-sectional area, the velocity can be reduced. If the low water level is reduced by 0.50m the cross sectional area gets increased to 121.975 m2 instead of 119.475m2.

V= 180.56/121.975 = 1.82m/sec. But is no such reduction in velocity could be found. However, by increasing the height of the vent by lowering the L.W.L by 0.50m by excavation the waterway will be increased.

Both U/s and D/s bunds on the left and right side banks are to be protected by constructing retaining walls. The retaining walls are to be constructed using individual stones weighing not less than 40kg [7]. It is predicted that the eddies may hit bunds badly on the right side on U/s and left side on D/s.

For the cost-effective solution is to construct the retaining walls using the naturally available boulders in the stream duly breaking them into pieces with sharp angled edges to have better interlocking.

As huge amount of stones are required to construct the retaining wall, the stones available weighing 40kg are very less in quantity, it was to be suggested to form Gabions of size 2.00m x 1.00 x 1.00m with 3mm gauge aluminium wire mesh and to lay them in three layers on below the ground and two above the ground on all the four sides of bund as shown in figure [2]. The ends of the retaining walls have to be protected with toe walls.

To protect the substructure portion of pier and abutments at low water level and below, it is necessary to construct toe wall in un-coursed rubble masonry with reinforced cement concrete cap with M20 concrete all along the length of the bridge both on upstream and downstream abutting to piers and abutment.

In between piers and piers and abutments for all the 7 spans, it is necessary to construct permeable apron using un-coursed rubble stone masonry with proper vertical keystones to protect the bed from scouring.

5 TECHNICAL SUGGESTIONS GIVEN

These are given based on the technical study and it was suggested the following protection measures.

Both on right and left sides of bunds on upstream and downstream have to be protected with Gabion retaining walls with blocks of 2.0m x 1.0m x 1.0m in three layers one below the ground and two above the ground level. First preference in execution shall be given to the right side of U/s bund which is predicted for the worst hit Figure [3]

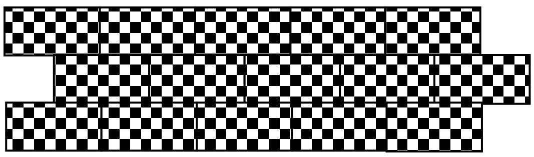

Figure 3. Gabion block of size 2.0m x 1.0m x1.0m

Construction of toe wall with un-coursed rubble masonry all along the length of the bridge both of upstream and downstream to be taken up to 2.0m depth from low water level as the strata met with is hard rock. Coping of 0.15m thick with reinforced cement concrete using M20 grade concrete is necessary for the entire length of the toe walls, Figure [4].

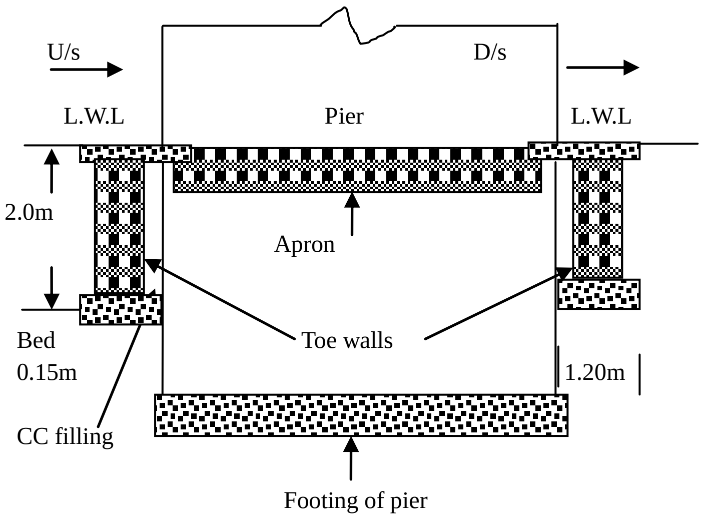

Figure 4. Toe walls and apron

Filling the open space at ease and cutwaters with P.C.C was suggested.

It was advised to wait for one or two monsoons to see if any abrasion or cavitations are noticed then protection of pier and abutment walls with epoxy mortar or jacketing with reinforced cement concrete would be thought off.

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6 OBSERVATION OF THE BRIDGE DURING THE MONSOON

It was informed by the field engineers to the designer that the bridge is being affected by the flood and suggested to inspect the bridge. During the inspection, it was noticed that monsoon all over Karnataka and Goa (India) was the worst hit during that year (2018). It was unfortunate to note that, the early monsoon had started before taking up protection works of the embankment with Gabion retaining walls. Flood level recorded was about 1.0m more than ever before during the last 5 decades.

The bridge was subjected to many forces in addition to hydrodynamic forces. There were many impacts on substructures due to the flow of debris. Big trees of length varying from 2.0m to 8.0m and girth of 1.0m to 1.20m were dragged by the flow of water in the stream and chocked the vents of the bridge. About 3 spans out of 7 were almost chocked. Thus the water was being catered about 42 to 43%, Photo [4]. Remaining discharge and due to eddy current, the water was diverted towards the right side on U/s bund. Some quantity of discharge was overflowed above the deck to a depth of 1.0m, Photo [3]. Due to the diversion of water towards the right bank of U/s bunt i.e. at the entry of the bridge the approach was completely washed away, Photo [1].

Since the round-shaped boulder available in the stream were used without breaking them to sharp angular pieces to fill up the approach embankment there was no proper interlocking so approach washed out Photo [2].

Photo 1. The approach is washed out

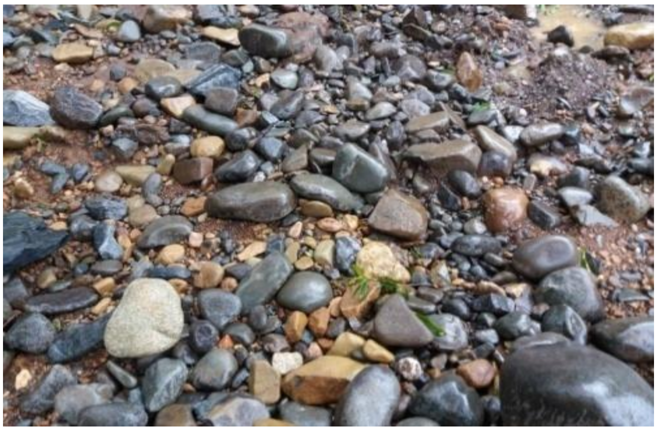

Photo 2. Round pebbles used for the approach embankment

Photo 3. Water flowing above the deck. Even the openings of the R.C.C handrails are choked up with debris

Photo 4. Vents chocked up due to floating debris (trees) on U/s of the bridge

Photo 5. Construction of U.C.R apron

7 OBSERVATION OF THE BRIDGE AFTER THE MONSOON

It was frightening to see the major effect of eddies. The approach at the entrance was washed out including tilting to falling of handrails. The handrails were not constructed on the filled up earthen embankment of approach instead of the top of returns which had the firm base.

The toe walls masonry on upstream for a length of 3 spans from the entrance where eddies were much predominant were partially washed out. Reinforced cement concrete coping was overhanging for this length. The deterioration of the submerged surfaces of masonry brought about by the abrasive action of solids in motion in the water is called erosion.

The permeable apron stones of size more than 0.45m up to 0.75m were also dragged to downstream by the current. As the velocity varies with the intensity of rainfall the velocity in the stream during 2018 monsoon must be much higher. So if velocity is calculated with the reverse process using equation (8). (d)= 36.15 x V_b^2 V_b^2 = 750/36.15 = 20.746 Therefore:

$$V_b = 4.55\text{m/sec} \tag{9}$$

which is more than 2 times the velocity considered in the design. Since the velocity is >3.0m/sec and <6.0m/sec forces due to velocity head, afflux and differential head and also the erosion of piers are the effects [5].

According to this when it was observed surface of piers small cavities of 10 to 15mm diameter were formed. Since the substructures were protected by toe walls no such cavitations were noticed. The depth of water at the entrance approach was 2.0m.

8 ADDITIONAL SUGGESTIONS AFTER THE MONSOON INSPECTION

Since due to eddies and chocking of bridge vents the entrance approach was washed out, it was felt necessary to provide additional relief vents in the entrance approach portion. Length of approach was 60.0m and the depth of the water during the flood in this portion was 1..0m cross-sectional area was calculated as 60x1.0 = 60m2 Using equation (9) additional discharge to be catered is Q ad = area x velocity Qad = 60.00 x 4.55 = 273 m3/sec allowing 50% through relief vents after clearing all the vents of the bridge the discharge to be catered is 136.50m3/sec. So adopting 1.20m diameter Np2 R.C.C Hume pipes, the discharge through one row = Cross-sectional area of one pipe x velocity. Area of one pipe = 3.142 x 1.2 x1.2 = 4.524m2 Discharge through one pipe = 4.524 x 4.55 = 20.586m3/sec. So, no of pipes required to cater 136.50 m3/sec No of rows = Discharge to be catered / discharge through one row of pipe. o of rows = 136.50/20.586 = 6.63 Say 7 rows

So it was suggested to provide 7 rows of 1.20m dia Np2 Hume pipes with M20 P.C.C headwall with the foundation depth equal to the foundation of returns. It was also suggested to provide arresters on the U/s of relief vents to arrest the entry of debris into the pipes. It is also suggested to construct one more span at the entrance approach for easy flow of water under floods in future. It is also suggested to provide arresters on the U/s of the main bridge to avoid the entry of the debris into the vents.

According to the suggestion given the relief vents of 7 rows of 12.0m diameter NP2 pipes are constructed and the bridge is inspected after 2019 monsoon and the following observations are noted down.

The relief vents provided are functioning well. M.S angle arresters are provided on the U/s of relief vents. No chocking of relief vents are witnessed in Photo [6]. No additional cavitations are witnessed on piers and abutment surfaces. No major debris has chocked the vents.

At present no arresters are provided on the U/s of the main bridge.

9 RECOMMENDATION AND CONCLUSIONS

The design of a bridge is a typical and complicated task for engineers as several variable parameters contributing to the design are not uniform and constant. All the field parameters differ from region to region on this earth. The engineer has to study the design aspects based on the standards laid down based on previous experience for several years. The effect of nature is a continual challenging task to engineers. So the design, execution and maintenance of bridges are never-ending processes with the new experience and challenges being faced during the day to day life. Among these, if the mistakes are done either during the design or during the execution is another challenging task for the engineers. It is very important to analyse the root cause for the mistake before taking up remedial or protective measures. In the instant case, study has been done on a skewed bridge constructed against the actual skew. The devil effect of eddy water current has been analysed and the protective measures have been taken up and also the performance of the protective work is studied and concluded that the bridge is safe with additional protective measures for the protection of the bunds. Considering this case study it recommended for necessary modification that can be had in empirical formulae for the computing design discharge for the areas with thick forest as below.

Conclusions of this study are as follows.

1. The discharges calculated by any empirical formula using catchment area may not reveal the correct discharge of the river/stream. This is because the contour maps prepared many years ago may not suit to the present topography of ground which might be changed for many years due to forestation, afforestation, scouring or depositions due natural disasters.
2. It is concluded that the precise way of computing the design discharge is to consider the discharge by area velocity in case of thick forest area as the entire quantity of water from the catchment area has to cater through bridge site.
3. A thorough and deep study is required to fix up the highest flood level during the past 5 to 10 decades. It is required as it is the base to arrive at precise design discharge. The exact marking of the highest flood level should be observed by thorough investigation.
4. To compute the design discharge, the Indian Road Congress clause, to consider 1.50 times the lower discharges arrived by any two methods needs modification for thick forest area. This clause may have to be modified to consider the maximum of the two discharges.
5. In the case of skew alignments, it is recommended to change the alignment to construct the bridge at the right angle to the river/stream crossing to avoid the hydrodynamic (eddy current) effects on the bridge structure.
6. In case if the bridge is constructed opposite to the actual skew for any reason, the protection works for piers, abutments substructures and

foundations should be taken up with concrete toe walls considering the grade of concrete depending upon the velocity stream.

7. In the case of skewed bridges, the major protection should be taken up for bunds where the effect of eddies is predominant. The size of the stone or gabion block should be following the velocity of water in the stream where the required size and weight of stones are not available.
8. If the impervious apron is provided with stone masonry it should be provided with the keystone without fail to sustain the water force on the apron.
9. The impervious apron between piers is advisable instead of the permeable apron using un-coursed rubble masonry with vents to release uplift pressure.
10. If major cavitation is noticed in substructure surface the surface should be plastered with suitable rich epoxy mortar or the jacketing to the pier and abutments should be taken up with rich concrete and Nito bond or any other such epoxy preferred to achieve adhesion between old and new concrete based the velocity of stream or river.
11. If the foundation is the open shallow foundation in hard rock if is recommended to provide dowel bars of 25mm M.S 1.50m within the rock and 15.0m within the body of pier and abutment at 12.0m centre to centre in X and Y directions in a zig-zag way or as per the direction of the engineer in charge.

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