FAILURE OF 2–METER DIA AND 54 METERS LONG PILES NO. 3/1, 3/2, 3/3,4/3,6/2, CRACKS IN TRANSOM NO. 6 AND OTHER PROBLEMS OF WEST CHANNAL BRIDGE OVER RIVER CHENAB NEAR CHINIOT

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INTRODUCTION

1. The Romantic River Chenab of Punjab passes through two Hilly gorges near Chiniot City. One is called east Channel (Chiniot Side) and the other is named as West Channel (Sargodha Side). Maximum discharge over hundred years flood cycle is estimated to be about 1.4 millions cusecs down stream of Qadirabad Barrage which is roughly 50 miles upstream of Chiniot. The capacity of the valley between Qadirabad Barrage and Chiniot is nearly 0.4 millions cusecs. It means that approximately 1.0 millions cusecs of water is likely to pass through the two gorges in a dream flood. A table given below would give an idea of maximum floods of over 0.6 millions cusecs (the capacity of two gorges) which have passed through Qadirabad Barrage since 1971.

YEAR	MONTHS		
		August	September
1973	-	854341	-
1975	669810	-	-
1976	-	628741	-
1977	982871	-	-
1988	-	-	914000
1991	-	-	950000
1995	640577	-	-
1996	-	850000	-

2. During floods of "1991 scouring was noticed around piers of Railway bridges constructed on East and West Channels. To avoid damages to the well foundations, lot of stones were dumped around effected piers in an effort to stabilize them.

3. C&W Department Punjab initiated a scheme in eighties to construct a 4-lane Bridge about 9000 feet down stream of Existing Railway Bridges. The length of the Bridge was 3100 feet consisting of 20 spans of 150 feet each. The Bridge was to be designed for a maximum discharge of 936000 cusecs which constituted discharges through two gorges over which

Railway Bridges are located and the discharge through breaching section and spill through low levels near Rabwah.

4. Hydraulic studies which were carried out at Hydraulic Research Station Nandi Pur indicated that the capacity of the gorges was 6,00,000 cusecs only.

5. Towards the beginning of 1991, NHA Islamabad started a project to construct a bridge over river Chenab near Chiniot. Since the location proposed by C&W Department Punjab required extensive land acquisition, training works and long approach roads, it was decided that two new Bridges be constructed about 100 meters downstream parallel to the existing railway bridges. As per Designers report, the Bridges were to be designed on West and East Channel for discharges of 4,50/000 and 3,50/000 cusecs respectively as per direction of the Client . The Designer and the Supervisory Consultants were one party.

6. The Project was allotted to the Contractor after competitive tendering at a cost of Rs. 310.0 millions. It consisted of two Bridges on East and West Channels, a Rail Road Overpass near Rabwah and about 2.50 K.M. of Approach Roads and training works. The work was taken in hand on 01-01-1993. The details of the different components of the Project are: -

S.No.	Description	Length of Span	No. of Span	Total Length
1.	West Channel Bridges	40 Meter	7	280 Meters
2.	East Channel Bridge	40 Meter	6	240 Meters
3.	Rail Road Overpass	Different Lengths	25	588 Meters
4.	Approach Road	2.5 K.M		
5.	Training Works			

During the floods of August 1996, one of the under construction piles of West Channel Bridge of 2 meter dia i.e. Pile No. 3 of Pier No. 4 (4/3) was washed away, while the central pile of Pier No. 6 showed sign of settlement in the form of cracks in the transom as per initial report of the Supervisory Consultants of the Project. Excessive fluttering of recently constructed piles of Pier No. 3 (3/1, 3/2, 3/3) of the same Bridge was also noticed. The discharge down stream of Qadirabad Barrage was 8,50,000 cusecs during this flood. The dates of completion of these elements are:

In March	Name of Structure	Date of Completion
1.	Pier Pile No. 4/3	08–06–1994
2.	Pier Pile No. 6/2	14–03–1993
3.	Transom No. 6	20–02–1994
4.	Pier Piles No. 3/1, 3/2, 3/3	03–05–1996 05–06–1996 27–05–1996

CORING OF PILES

After the above mentioned mishaps it was advised by the Consultants that to check the integrity of the piles, ultra sonic testing of all the piles constructed so far be carried out. M/s Associated Constructors Ltd. Karachi were recommended for this job. After lot of discussions and personal contacts they were of the view that Sonic Integrity of piles over which shafts have been constructed would be of no use due to construction joints at cut off level of the piles.

There after, Consultants requested the Client to arrange coring of all the piles through a specialist Sub Contractor, as the main contractor did not posse required equipment. Three holes on each pile were desired to be drilled upto its tip level. The size of the bores was about 2". Coring was done on piles of Piers No. 5 and 6 only.

The comments of the Consultants in this connection were " in order to check the quality of concrete in the piles the Client decided to have the core drilling carried out in the pier piles of the West Channel Bridge. The Client appointed a Contractor to do this work. From the core samples it is concluded that there is no discontinuity or voids in the body of piles drilled so far. It is advised that the piles do not appear to have failed structurally. The damages to the piles Pier No. 3/2 and 3/3 have amply proved that the piles have failed due to flutter".

REMEDIAL MEASURES

No problems were faced by East Channel Bridge during floods of 1996 or 1998 and was completed without any mishap. The Project has been completed in all respect with following changes in the design of Piles and Transoms (Remedial Works) of West Channel Bridge.

1. Pier No. 3

4 new piles have been constructed in replacement of 3 old piles, one on either side of piles No. 3/1 and 3/3 and topped by 3 transom beams as shown on the attached plan. The old piles have been abandoned.

2. Pier No. 4

Piles No. 4/1 and 4/2 had not been constructed before the floods of August 1996. On both sides of disappeared Pile No. 4/3 one new piles has been provided. In place of unconstructed Pile No. 4/1 two new piles have been constructed one on either sides of its location. Transom beams were provided on all four new beams as in case of pier No. 3. Pile No. 4/2 was also not constructed.

3. Pier No. 6 : Details of CRACKS are given on Pages 342 to 343.

Concrete of part of transom of Pier No. 6 on the top of Pile No. 6/2 which contained cracks was removed by chiseling in a length of about 3m upto its soffit without damages and disturbance to the steel reinforcement. Two new piles were constructed one on either side of Pile 6/2 with transom beam over them which also passed on the top of Pile No. 6/2 in place of removed portion.

4. The length of replacement piles in Piers No. 3, 4 and 6 was increased from 55 m to about 60 meters. The steel reinforcement was also enhanced from 33 to 43 tons in each pile with different configuration/arrangements from original piles.

5. On the orders of the then Prime Minister of Pakistan an inquiry was conducted in February 1998 by Pakistan Engineering Council regarding damages to Pier Pile 4/3, 6/2 and Transom No. 6. Upto that time no damages to Pier Piles No. 3/1, 3/2 and 3/3 had occurred and therefore their impact on the inquiry was not possible.

6. Railway Bridges

Details of Railway Bridges constructed on East and West Channels up Stream of Road Bridges are given on pages 344 to 346.

SUMMARY OF REPORTS/COMMENTS OF THE

DESIGNER/CONSULTANTS ON FAILURE OF PILES NO. 4/3 AND 6/2 OF WEST CHANNEL

The Bridges on West and East Channels were designed for discharges of 4,50,000 cusecs and 3,50,000 cusecs respectively and the construction work proceeded accordingly.

The Bridges passed 8,50,000 cusecs whereas the gorges had a capacity of 6,00,000 cusecs only as established by Hydraulic Model Studies at Nandi Pur Research Station. The structure of West Channel Bridge would have been safe in all respects upto 5,25,000 cusecs. Excessive scouring around piles clearly indicates that a very large discharge passed through West Channel resulting in distress to sub structure elements.

Details of Pier Pile 4/3 can be see in Pages 336 to 341.

In a review of the Design the Consultant/Designer has given following calculations for computation of Depth of Scour. On the basis for 950,000 cfs.

	West Channel	East Channel	
-	Average Scour	21.668 m	20.181 m
-	Local Scour	43.336 m	40.362 m
-	Local Scour at Abutments	27.518 m	35.630 m

Highest flood level of 950,000 cfs from IRI report No. 924/HYD/1989 is at an EL 599 ft (182.575 m) and EL 600 ft (182.850 m) at East Channel and West Channel respectively.

Based on the above water levels, the corresponding scour levels calculated by them are:

		West Channel	**East Channel**
-	HFL	182.850 m	182.575 m
-	Average Scour	161.212 m	162.394 m
-	Local Scour at Piers	139.544 m	142.213 m
-	Local Scour at Abutments	155.362 m	156.945 m

For both the bridges, minimum scour level at piers has been adopted by them as 139.0.

Following reasons have been advanced by the Designer for failure of Piles: -

Continuous flutter due to Hydraulic dynamics of flowing water has been the main of cause distress to the Piles. This distress to piles has taken place in their free standing condition due to their natural frequency coinciding with the frequency of the vortex shedding in their wake in the temporary stage during construction. The computations so far indicate that with scour at E!.139 and water level at 182, the 43m free standing single pile has a natural period of vibration in submerged condition of about 2.45 sec. The vortex shedding lock-in velocity appears to be between 4m per second to 3.2m per seconds; that is, in this velocity range an isolated pile of submerged length of 43m in water tends to vibrate in resonance with the vortex shedding frequency. Piles at pier 3 were observed to move to and from laterally at right angle to the flow in the river in the flood of 1996 and also during the subsequent floods. This flutter like movement has been well documented in all structures such as chimneys during high winds. A similar phenomenon was taking place in case of isolated single piles during the flood.

It is clear from the above that the disappearance of pile P-4/3 in 1996 and displacement P-3/2 and P-3/3 in 1998 is due to the fact that these piles were left in isolation devoid of the restraint of the superstructure. The flutter caused large moment reversal causing loosening of the piles in their grip lengths in soil and possible structural damage to it in places where bentonite may have been left in the body of concrete.

The proof of the above is the fact that the East Channel Bridge has remained in tact as its superstructure was in place before floods. It has stood 1996, 1997 and 1998 floods without any problem, although the model tests indicate that flow concentration is severe in this channel.

Failure of pile 4/3 can also be attributed to deficiencies in construction operations particularly in view of the fact that a serious discontinuity in concreting operation in this pile took place and earlier boring bucket of drilling rig was lost and remained in the incomplete bore hole for almost a month.

It was noted that a left spur constructed in the 70's combined with a Mosque and Dargah in between the gorges had completely changed the Hydraulic characteristics of the River. Due to Hydraulic conditions, completely different from those envisaged at the time of designing it was proposed to the Client that a Hydraulic Expert be employed to look into the problem. He might suggest some training works or modification of the spur constructed on the left bank. In the mean time piles of Pier 3 and 4 be designed in such a way that even if discharge in the West Channel reaches 7,50,000 cusecs they should be safe.

Pier – 6 Cracks in Transom (Details of Pages 342 to 343)

After the floods of 1996, the transom of this pier was closely inspected as a prelude to placing the girders on the first left span when two cracks of less than 0.1 mm width were observed on the Sargodha side face, on the face of the middle pile covering a substantial depth of the transom upto the top projection. The level of bottom of the transom was checked and no deflection at the middle pile was observed. Moreover the cracks on the sides did not join the soffit of the transom.

In the subsequent period, the summer flood of 1997 passed. It was observed that the two main cracks joined at the soffit along the periphery of the pile and in addition two more smaller cracks appeared at a distance of 0.5m on the sides. These additional cracks did not fully extend to the soffit. There is no load on the transom except its self weight. A conservative check was made for crack width due to the maximum effective weight (with buoyancy at low water at E1.170) of the central pile imposed on the transom assuming a span of 2 x 6 = 12 m supported on the two exterior piles. These calculations indicated a crack width of 0.225 mm at the soffit and crack on the sides would disappear at a height of 0.33m. This crack must be accompanied by the corresponding deflection also computed to be 5.33mm. No deflection of this order has taken place to this day. This clearly shows that the hypotheses that the middle pile was hanging from the transom is not tenable.

Since the above computation did not confirm the presence of the cracks under vertical load without deflection, the side cracks would only be explained if the transom were to undergo first mode (bow-like)deformation in plan.

Preliminary calculation of natural period of vibration of a beam (Transom) of 12m span (assuming that the transom is supported on two exterior piles) of the cross section of this transom indicate that its natural period of vibration (in air) is 0.027 seconds. Therefore the transom and pile vibration coupling is out of question as the period of the 43m long submerged pile is almost 90 times longer and consequently, dynamic coupling cannot take place. There is a possibility of out of phase vortex shedding of the three piles, the scenario when the outer piles flutter in phase but out of phase with the center pile.

Taking a lateral load of 227KN acting on the length of pile, the transverse reaction on the transom would be 106KN, assuming an angle of attack of 45 degrees to the axis of the bridge.

The moment at the center of the pile would be 318 KnM. The reinforcement on each side is 4422mm2. The crack width is calculated to be 0.05mm. This corresponds to the observed cracks as they were visible in 1996. Subsequently in the following flood seasons due to repeated application of lateral loads in its free unrestrained state the cracks have become prominent, more so as the affected area has been painted subsequently to enable the cracks to become prominent to be easily seen from distance.

The cracking of transom without deflection is confirmed by analysis of lateral reaction of the piles.

Pier 6 Piles have to be investigated by core drilling before commencement of work.

COMMENTS/OBSERVATIONS OF THE CONTRACTOR ON THE FAILURE OF PILES NO. 4/3 AND 6/2

The soil investigation report on which design of piles was based was deficient in many ways. Presence of rock was not indicated in the bed of the river, whereas out of 30 piles in the West Channel, 9 piles were embedded in the rock formation. Similarly there is no mention of presence of drifted stones in the pile holes. Actually pitching stones were encountered at different depths ranging in thickness from 1/2 to 3M

The presence of rock in the first span of this bridge was not in the knowledge of the Designer. Its effects therefore on the flow pattern could not be visualised by him. Due to the presence of rocky bed an obstruction was formed which hundred the scouring in the first span and diverted part of the flow to the adjacent spans. Concentration of flow in these spans was one of the contributing factor for excessive fluttering of piles of pier No. 3 and washing away of Pile 4/3. Such a situation is covered by provision No. 703.2.2 of Code of Practice of Indian Road Congress and it is apparent that the important guide lines provided therein were not followed in the Design of Piles.

Piles provided for Chiniot Bridge are long piles, the ratio of length and diameter being more than 25. In such piles the cumulative passive resistance at the lower part is very high. As these piles cannot rotate failure many occur by fracture of the pile at the point of maximum bending moment. In the case of a long pile restrained at the head (Pier Pile No.6/2) the high bending stresses occur at the point of restraint where fracture may take place.

As such there is every likelihood that failure/fracture of piles 3/2,3/3 and 4/3 might have occurred due to excessive deflextion not taken care of in the design.

(Reference Page 430 of Foundation Design and Construction by M.J.Tomlinson Fourth Edition)

The left spur was constructed much before the Design of this Bridge was taken in hand. Why the effects of this spur, Mosque and that of Dargah not taken into account while designing the pile foundations. The best course would have been to run another model study to ascertain the behavior and pattern of flow in River during highest flood in both channels to establish design parameters.

It is true that while drilling pile 4/3, the bucket of the drilling rig got disconnected from Kelly at a depth of 36 meters from working platform. This was an accident/mishap inherent in pilling operations. It was taken out after great efforts. However it is wrong to say that there were deficiencies in concreting operations particularly in view of the fact that a serious discontinuity in concreting occurred. Concreting of pile is possible only after an admixture is added to the concrete. It increases workability or flow of concrete and also retards its setting time upto 3 hours sufficient to take care of minor delays in concreting as in the case of aforementioned pile.

The piles failed due insufficient depth with less provision of steel reinforcement. This assertion is confirmed as in the replacement piles length has been increased by 10% and the steel reinforcement by about 33% with different configuration by the same designer. In one of the recommendations of the Consultant/Designer, it is stated that replacement piles be designed in such a way that even if the discharge in West Channel reaches 7,50,000 cusecs, they should be safe. It clearly means that the design discharge was under estimated in West Channel and the failure of piles occurred due to deficient design.

HYDRAULIC EXPERT REPORT

Deterioration of River Channel, and Rising Flood Levels and Intensities

In 1998 the Client appointed a hydraulic expert to look into Hydraulic Design of this Major Bridge. He states: “We support the recommendation of the Project Consultant that it is important to under take further study for River Training on the up stream as well as other measures to ensure safety of the Client's Road Bridges and more so of the up stream Railway Bridges. The Railway Bridges having shallower foundation are under a great danger by fast flood flows, oblique currents swirls and scour with increasingly uneven flaws through the two gorges”.

River regimes in the alluvial plains of Pakistan have been significantly disturbed due to human interventions. Canal withdrawals have constantly increased to support irrigated agriculture to meet the rising food and fiber needs of the growing population. As a consequence of the Indus Waters Treaty 1960, substantial river supplies have been diverted to feed the canals of the Ravi, Beas and Sutlej rivers. Historic flows through the rivers in different reaches have fallen below the historic pattern in changing patterns.

In case of the Chenab river, generally no supplies are allowed to pass below Qadirabad for long periods from September to April, and the Jhelum and the Chenab flows together hardly meet the need of the canals. Even in early and late summer, supplies through the Chenab get significantly reduced. This has influenced the formative discharges in the river, and caused deterioration of Main River Channel**.** Silt exclusion at barrages has raised silt load in the river downstream, and has resulted in the river bed becoming higher and higher. The pressure of urbanization has led to construction of more and stronger flood protection bunds along the river to check spills. The relief available due to dispersal of high floods in the river valleys has got greatly reduced, and has resulted in higher and sharper flood peaks in the rivers and at the bridges across them.

These factors have and will continue to influence and tax the safe flood capacity of the bridges, flood levels and the meandering character of river. Due to river channel deterioration, flood levels are steadily raising**.** Further, if a super flood occurs after a dry cycle, HFL will be significantly higher than when such a flood follows a smaller flood or occurs in the middle of a wet cycle.

The 100 year flood peak at Chiniot is estimated as 9,40,000 cfs. exclusive of spill flow of 50,000 cfs on right flank of the River.

The local drainage channels flowing into the Chenab during major wide-spread rain storm in plains may synchronies with peak discharge in the Chenab, thereby influencing the percentage reduction due to valley storage in Qadirabad Chiniot reach.

There is a continuous flood embankment on left flank of river in Qadirabad-Chiniot reach whereas the flood embankment on the right flank of river is not continuous and is not strong enough and gets breached. If at any stage the right flank embankment is strengthened in Qadirabad to Chiniot reach, the 100 years flood at Chiniot may be around one million cusecs.

Reasonable design discharge of the Chenab river approaching the new bridge at Chiniot consisting of East and West Sections may be taken as 9,50,000 cfs. The distribution between the East and West Bridges is influenced by a number of factors.

The Road Bridge located only 90m below the railway bridge across the gorges inherits all the adverse flow characteristics of the gorges. Greater waterway at East/West Bridges will not be of much assistance due to interference and masking of end spans by the gorges and the upstream Railway Bridge guide banks. Flow through the road bridge spans would remain disturbed and uneven. Side spans of the bridges may remain ineffective or not fully effective. It has been experienced in 1996 flood that almost right one third of West Bridge and the end spans of the East Bridge reportedly had reverse rollers, leading to high concentration in other bays, disturbed flow conditions and vortex formation. These factors would lead to development of deep scours. A better location would have been fairly downstream, as recommended in IRI Report No. 924/Hyd/1989, where the river discharge gets mixed, resulting in better flow conditions. This opportunity is no longer available.

River Approach

The perusal of river surveys and field reconnaissance reveals that there may be various scenarios of river approach to influence flow distribution and current directions within the gorge

Case 1:

The river upstream is deflected by the left upstream spur and swings to the right to hug the right bank and forms an embayment. This embayment is checked and the current is deflected by the stone armoured foundations of 500 KV electric tower on the right bank of river. The main current upstream hugging the outside of the bend is deflected to the left flank of the gorge.

Case 2:

The river may approach the right gorge after deflection of the current from the nose of the left spur. It is then deflected at the stone armoured front of the embankment protecting the mosque on the left flank of the west gorge, and by the left guide bank nose of the railway bridge. The main current adheres to the right flank of the gorge

Case 3:

The river may approach the west gorge on axis line of the gorge, but at an angle skew to the axis line of the railway bridge.

Case 4:

The river may approach the gorge from both flanks of the river, causing dispersion of flow in the gorge, and more even flow distribution within the gorge.

Case 5: Worst Case

The worst scenario is that the river approaches the gorge from the right in an extreme bend, leading to concentration of flow in certain areas and intensification of scour by uneven skew currents, boils and swirls.

Analysis by Lacey Approach:

(For brevity only one analyses has been discussed in this paper)

It is proposed to analyze and estimate probable scour levels for the road bridge by using Lacey approach. Various scenarios of flow concentration, and corresponding appropriate multiplier factor would be considered.

Basic Data:

Based on the Design Report, for the road bridges, clear waterway is 268m (879.25 ft) for West Bridge, and 231.273m (758.76 ft) for East bridge.

At 950,000 cfs upstream river discharge, west channel share at 59% comes to 560,000 cfs., and East channel share at 41% is 389,500 cfs. The respective average q values are:

q_{av} (West channel)	:	560,500/879.25	= 637.5 cfs/ft
q_{av} (East channel)	:	389,500/758.76	= 513.3 cfs/ft
H.F.L	:	RL 601.00 ft or 183.175m	
R	=	0.9 (q2/f)1/3	
With f	=	0.9	
R (West)	=	69.01 ft.	
R (East)	=	59.51 ft.	

Scour:

This is to estimate scour with reference to average q cfts/frt. With a multiplier of 2.5 (guide bank nose condition), respective scour depths would be 172.5 ft (52.575m) and 148.78 ft. (45.345). with reference to HFL of 183.17, the probable scour levels are 130.61 and 137.83m for West and East bridge piers.

The conditions could be considered slightly less severe, with multiplier = 2.3. Respective scour depths are 158.72 ft. (48.38m) and 136.87 ft (41.72m). Scour levels are 134.79m and 1412.45m for West and East channel piles.

Consider a few concentration values of 30%

q (West) is 637.5 x 1.3 = 828.75 cfs

and R = 81.87 ft.

q (East is 513.3 x 1.3 = 667.34 cft.

and R = 70.88 ft

with a multiplier factor 2, as by COWI,

the 2 R values are 163.74 ft (49.91m) and 141.76 ft (43.21m).

Scour levels work out as 133.26m and 139.96m for West and East channel bridge piles.

Consider one fourth of the bridge waterway as masked (average) due to upstream skew approach and interference by railway bridge guide banks. This was reportedly the condition observed during 1996 flood. The concentration ratio comes to 33.3% above q (average). The respective R-values are 83.50 ft. and 72.01. With 2 as the multiplier, respective R-values are 167.0 ft. (50.90m) and 144.02 ft. Scour levels come to 132.27m and 139.27m, for West and East Channel Bridge Piles.

Consider one third of the bridge way as not effective on an average, in which case concentrations factor for q is 50%. This could be a situation with adverse upstream approach, and higher degree of skewness. As a result, R-values work out to be 90.40 ft. and 77.96 ft. In such a case, the multiplier may be taken as 1.75, giving probable scour values of 158.21 ft. (48.22m) and 136.43 ft. (41.58m). Scour levels come to 134.95m and 141.59m for West and East channels.

There may be another very adverse scenario. Consider that half of the bridge way is not effective, and for purposes of maximum q, only half of waterway is considered. The value of q doubles and would be 1275 cft (West) and 1094 cft (East). R-value comes to 109.4 ft and 94.3 ft. With such a high value of q, multiplier may be considered as 1.5. This would be probable scours as 164.2 ft. (50.06m) and 141.5 ft. (43.11m).

The scour levels work out as 133.11m and 140.06m for East and West channel bridge piles.

After examining the above data and giving different scenarios, the design scour level for bridge piles could be considered as 134.0m on the Lacey approach.

Recommended Scour Level

The East and West Channels have been analyzed With a variety of available empirical approaches and design formulae to estimate probable scour at the Bridge Piles with a peak design flood discharge of 950,000 cfs in the River through two gorges, HFL recommended for design at Road Bridge site is 183.17 m (RL601.0) and probable minimum scour level at the pile of 134 meter.

REMOVAL OF ROCK PROMONTORY ON LEFT OF EAST GORGE

It was recommended in IRI Report 730/1 lyd/1975, that the share of discharge of East gorge can be improved by about 5%, thereby giving some relief to West gorge, by providing greater waterway at the mouth of the East gorge. For this purpose a temple on the left was proposed to be dismantled and its base on the rock promontory was to be lowered, in a width of 65ft., upto low river water level in winter of RL 578.

The temple was removed, but upto an elevation of about RL 600, which is almost the HFL for 800,000 cfs river flood. It was of no consequence. It is recommended that the promontory may be lowered, in the recommended width, upto a level RL 578 (EL 176m). If site conditions permit, the width of rock removed may be wider and to a depth as much below RL 578 as possible. There are hardly any flows through the gorge in winter. This would provide relief to the West Road Bridge, and help in better dispersion of flow through the East Road Bridge. This step would be equally beneficial for the Railway Bridges.

Protection of Railway Bridge Piers and NHA Bridge Piles by Dolloses:

Dolloses are cast RCC six-legged pieces, weighing 6 to 8 tons each, which can be launched in the river bed. The Dolloses have interlocking capacity and have a great resistance to catapultation by high velocity currents upto 20 to 30 fps. They sink and settle at their assigned locations, are nearly immobile, and are dependable. They help in dissipating energy of the fast flow currents. They reduce bed scour when used for protection of bed, and site conditions.

CONCLUSIONS AND RECOMMENDATIONS BY THE AUTHOR

Selection of Site

It has been debated and argued that selection of the present site for construction of Bridges was inappropriate. Even the Hydraulic Expert has, commented in his report that "A better location would have been fairly down stream, as recommended in IRI Report NO. 924/Hyd/ 1989 where the river discharges get mixed resulting in better flow conditions. This opportunity no longer exists".

Though this view point has some weight but it is not a very fair assessment. The Bridges can be constructed any where even in a sea connecting two islands. Latest example is that of a proposed bridge between Sicily and the Italian Main Land. Railway Bridges upstream at a distance of about 90 m near the gorges also belie this stance as they are working without any hydraulic related mishap for over 70 years in spite of floods of very high intensities during this period. Other considerations favouring this location over that proposed by IRI or C&W Department Punjab are: -

a) Extensive land acquisition.

b) Long Approach Roads,

c) Long Training Works

What has gone wrong or is amiss is lack of assessment of the problems of the present site. These problems may be summarized as: -

a) the direction and the acceptance that the Bridges on West and East Channels be designed for discharges of 4,50,000 and 3,50,000 cusecs respectively was not logical as it did not take into consideration the changes in the bed of the river, water flow pattern and the construction of a spur, a mosque and Dargah on the left side and their effects.

b) There was absence of Expert’s advice or opinion about the effect of deterioration of the channels, formation of curved embayment, rising flood levels, and oblique flows/currents which will continue to influence and tax the safe flood capacity of these bridges and scours around piers.

c) Model Studies should have been carried out again to ascertain the flows between the two channels under the existing conditions for a dream flood of one million cusecs specially the concentration of oblique/skew flow through a portion of waterway, the presence of rocks in part of the river bed and constraints such as non-operation of Breaching Section in an emergency (Discussed later).

d) A study for river training on the up stream for appropriate flows, through the two gorges to ensure safety of the Road Bridges and more so of the up stream Railway Bridges with lower shallower foundations due to fast flood flows, oblique currents etc. is very much needed even now and would be useful for appropriate action in future.

e) From the Design Calculations provided by the Designer, the minimum scour level (Figure Page 336) is computed as 139.0 m whereas the Hydraulic Expert estimates it as 134.0 m, a difference of 5 meter. This level (134.0m) is also approximately the same as adopted in the replacement of damaged piles.

From above submissions it is quite clear that for mega Bridge Project, association of a Hydraulic Expert with the Designer/Consultants is very essential and may be made mandatory with appropriate clauses in Contract Documents and the model studies at IRI should also form an integral part of the Design.

DESIGN DISCHARGE

The normal practice for calculating discharge for mean depth of scour is that the total design discharge is divided by the effective linear waterway between abutments or guide banks. This method appears to be on the conservative side, as it does not take into consideration any concentration of flow through a portion of waterway assessed from the study of the cross section of the river, as has been done in the case of Chiniot Bridge.

It is therefore suggested that the discharge for mean depth of scour may be the maximum of the two condition mentioned above (concentration of flow in a portion of Bridge). This suggestion is in conformity with the guide lines provided in the IRC code of Practice 78 -1979 clause 70322.1

Risk Factors

A. A clear cut picture has not emerged regarding the reasons for failure of piles after the coring. According to the Designer/Consultants fluttering of piles and vortex shedding in free submerged state without over burden were the main causes of failure of piles. The other reason, which has been frequently discussed among concerned Engineers is that piles broke due excessive B.M and deflection under lateral loads. In this connection reference can be given of the design calculation provided by the Contractor.

B. Coring was confined to piles of Piers No. 5 and 6 only on the advice of the Consultants. These piles did not show any damage apparently. It is felt that the coring and detailed investigations of damaged/lost piles would have been more revealing. Once the lost pile had been located its profile should have been mapped to find as to weather it had been dislodged due to scour or it had been broken due to excessive deflection. Similarly tilted piles (3/2, 3/3) were also not investigated. These piles faced discharges of 0.85 millions cfs in 1996 and about 0.6 millions cfs subsequently without any visible damages, but tilted when the discharge was hardly 0.228 million cfs. WHY?

C. Construction of foundation of Bridges in Rivers of Punjab is a seasonal work. The working season starts approximately from middle or 3rd week of September every year. Some times in winter in March, April due to wet cycle sub structure construction comes to a stand still. It is always not possible to provide super structure to stabilize the completed piles. So a free standing submerged pile has a tendency to flutter in the flowing water. The

Designer/Consultant states that "*when the natural frequency of a submerged pile coincides with the frequency of Vortex shedding behind the pile then large oscillations are induced which result in loosening of the pile in the bed of the River. This is a condition peculiar to a free standing pile. The completed Bridge does not allow this to happen because of restraints provided by the super structure. Piles at pier 3 were observed to move to and fro at right angle to the flow in the River in floods of 1996 and also during the subsequent floods*".

Are such problems considered at the Design Stage? There is no mention of fluttering of piles, vortex shedding, bending moments, deflection and stiffness of piles in the design calculations for depth of scour provided by the Consultants. This leads one to think that such aspects of design were not considered at design stage. Vortex Shedding appears to be a new concept in pile design/ construction.

There is a lesson to be learnt from above. The construction of piles in river beds be considered a stage construction. There should be a built in Risk Factor in the design for fluttering of free standing submerged piles during floods. Therefore in addition to ensuring that there is an adequate safety factor against failure due to scour, the deflection of the piles under the lateral loads should be calculated to ensure that it is not excessive and that stiffness of the pile is safe specially when bridges are constructed on treacherous sites without detailed hydraulical studies as in the case of Chiniot Bridge.

The length of replacement piles of West Channel Bridge has been increased by about 10% with enhanced steel reinforcement. This may be one of the solutions of the above mentioned problems as it appears that this change in design has been done by taking into account the increased concentration of flow in effected spans. To emphasis this point the comments/observations of Pakistan Engineering Council in its Enquiry Report in this connection are reproduced below:

"The fluttering of piles of a Bridge where transoms have not been constructed is not an unexpected phenomenon. However there was certainly a need for the design of the piles for the construction stage also".

Increase in the Capacity of East Channel

There are recommendations in IRI Report 730/Hyd/1976 that share of discharge of East Gorge can be improved by about 5%, thereby giving some relief to West Gorge by providing greater waterway at the mouth of East Gorge. For this purpose a temple on the left was dismantled but its base on the rock promontory which was to be lowered in a width of 65 feet upto low water level in winter of RL 578 as per recommendations was not touched. This may be done now.

CRACKS on the Faces of Transom No. 6

The Consultants/Designer has given a detailed explanation for cracks on the faces of the transom No. 6, which may be read with interest on pages 342 to 343. However, the findings of Pakistan Engineering Council are quite different which are reproduced below:

"The large width of the cracks on the faces of the transom in the web is attributed to inadequate steel on the faces of the web as against code requirements. This is a design deficiency".

It is therefore for the Designers to take into consideration both explanations/reasons while designing the transoms over Piles.

Exploratory Drilling

The sub Soil Investigation Report available with the Contract Documents showed that piles were to be constructed in sandy alluvial soil. However during actual construction 9 out of 30 piles were constructed on rock, while in other piles pitching stones in different depths from 10 to 54 meters ranging in thickness from 1/2 to 3 meters were encountered and had to be removed for completion of piles. This resulted in delay as Reverse Rotary Rigs which were mobilized at site proved ineffective and special rigs were imported for this job.

On Exploratory Drilling design of the foundation is based. If it is incorrect the results would be disasters as lot of problems would be created and lead to invitation of claims and delay in completion of the projects as in the present case. It may be advisable for the Designers to attach their Engineers to supervise the Exploratory Drilling.

BREACHING SECTION (B.S):

There is a Breaching Section on Pindi Bhatian - Chiniot Protection Bund adjacent to the left spur as shown on the attached plan upstream of Chiniot city. It is about 200 feet long and is protected on all side by barbed wire. There is a gauge near by on which the danger level is painted in red colour. Instructions are written on a board when the breaching section is to be operated by the Army in the presence of Civil Authority and the representatives of Irrigation Department and Pakistan Railways. The city has been protected by another Bund, which crosses the Main Faisalabad Sargodha Road. The city has expanded towards the River Chenab. Residential and Commercial Buildings have been constructed in the area where discharge from breaching section is to flow. There could be very serious law and order problems, other repercussions and resistance from local people for operation of the B.S. This is quite evident from the very fact that no action had been taken by the concerned authorities to activate the breaching section when the water attained dangerous level in 1996.

There are many breaching sections on Rivers in Punjab, such as a breaching section on River Ravi near Shahdara and the other near Wazirabad on River Chenab. It has been experienced that whenever the breaching section near Shahdara is activated, the town is completely submerged under flowing water which plays havoc with normal daily life, destroying property, creating misery, death, hunger, more poverty and disrupting traffic on National Highway apart form spreading different type of diseases in the area. Can any body visualize the calamity, the disaster, the loss of human life and the horrors which resident of Rawalpindi suddenly face when Lye-Nallah over flows its banks during rainy season.

Breaching Section were useful and easy to operate in the past but now their operations are fraught with danger due to reasons mentioned above. This is a problem, which needs serious consideration. B.S were provided to save cities and expensive structures on Rivers worth Millions of rupees. By operation of B.S Private Properties worth millions of rupees are endangered now. Whether public or private, they are all Pakistani Properties. They are our valuable assets. We should therefore consider ways and means to safeguard interest of all the parties involved in the matter.

One of the solutions may be construction of permanent channels on B.S with gates at the entrance to discharge excess water down stream in the River.