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SOME ASPECTS OF HYDRAULICS OF BRIDGE WATER WAYS

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1.0 INTRODUCTION

1.1 Rail or road bridge is a facility for safe crossing of rivers, bays and tidel inlets for the transport of passengers and freight.

Each agency having responsibility for the design or maintenance of scour-susceptible bridges crossing the alluvial water ways is concerned with the adequacy of the structure foundation in respect of maximum likely scour depth, minimum grip length required, base pressures and stresses in the sub-structure.

1.2 The cross section of river within the Khadir (river valley between high bluffs) consists of main channel and over bank flood plains on the two flanks of the main channel or only one flank of the main channel (Fig.1). The percentage of the flood flows over the over bank sections depends on the type of river, river stage, the river reach (which may be degrading or aggrading), the composition of soil, vegetation etc.

1.3 It is seldom economically feasible or necessary to bridge the entire width of a stream as it occurs at the maximum probable flood. Where conditions permit the approach embankments to the bridge are extended out into the flood plain to reduce costs, recognising that, in doing so the embankments projecting into wide flood plains, will constrict the flow of the river during flood stages. Too long a bridge may cost far more in capital investment than can be justified by the benefits obtained. Alexendra bridge over river Chenab, when originally constructed in 1870 (without guide banks) had 64 bays each of 132' span but 47 bays were closed by 1919 and it was left with 17 bays but with Bell's guide banks on the two flanks of the bridge. The Empress bridge on River Sutlej when originally constructed (1874-78) was 4000 ft. long and subsequently in 1929-1930 this length was reduced

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to 2000 feet. The railway bridge on River Ravi at Shahdara had originally 33 spans of 97.5 feet each but was cut short to 15 spans in 1909.

Again the confining of the river flow-way unduly may cause excessive back water with resultant flood hazards, or may cause excessive scour endangering the safety of the structure or costly maintenance of the approach embankments to the bridge or even contribute to the complete loss of the bridge or the approach embankments. The flood and traffic hazards at Alexendra bridge during 1957, 1959, 1973 and 1975 floods were unprecedented. For the same reason the restriction at Shahdara bridge resulted in inundation of wast areas during 1955, 1973, 1975 and 1976 floods.

1.4 The scour is erosive action of running water in excavating and carrying away material from the bed and banks of the fluvial channel. Local scour is caused by local disturbance of the flow and sediment transport field. A local increase in velocity and/or turbulance intensity gives an increase in local transport capacity.

1.5 Embankments projecting into wide flood plains create extreme concentrations of flow at the upstream corners of the embankment and constrict the water way opening of the bridge with a corresponding increase in flow concentration and resulting scour potential at the piers (Fig. 1). The borrow areas immediately upstream from an embankment markedly increase scour potential at the toe of the approach embankment due to concentration of flow parallel to the embankment, at the abutments and the piers adjacent to the abutment (due to redirection of flow caused by the embankments).

Constriction of a stream produces back-water (afflux) at flood flows resulting in higher velocities in the constriction and due to increase in local water slope also increase in tractive-force and stream power with consequent increase in transport capacity of the flow. The greater capacity of the flow for transportation results in scour of the bed in the vicinity of the constriction; the removed material is usually carried short distance downstream and dropped as the velocity decreases due to expanding and diverging out flow (Fig. 1). As the scouring action proceeds and the flood persists for a sufficient period of time the degradation of bed may reach a state of equilibrium i.e. the rate of bed material transported out of scour hole is essentially reduced to the rate of supply to the hole from upstream. At the state of equilibrium the water way under the bridge has enlarged, the back water has all but disappeared and the farmer regime is restored.

1.6 Guide banks projecting upstream from abutments afford protection to the embankments and the adjacent piers. The guide banks redirect the flow pattern to parallel the desired channel alignment, to even out the flow, to utilize the full bridge width and to minimise the scour at the abutments and the adjacent piers. The effectiveness of the guide banks is a function of the geometry and shape of guide bank, the quantum of flow on the flood plain, the river approach to the guide banks, the size of bridge opening,the placement of the road way embankments and the lay out of marginal embankments limiting flood plains. The installation of guide bank does not eliminate the localized scour at the piers, it decreases the degree of scour and its extent at the meeting point of forward flow and lateral flow parallel to the approach embankments, protects the toe of embankments against parallel flow as a dead water pocket forms behind the guide bank which bolsters the active flow away from the embankments. The guide bank if not suitably aligned for the worst river approach gets captured at its upstream nose and when fully captured a deep embayment forms behind the captured nose which sweeps off the main current away from the captured guide bank and towards the opposite guide bank where it digs a deep channel along a solid wall with the net result that the bays along the captured guide bank get masked by a sandy bela (especially when covered with vegetation) on the inside of river bend in between the guide banks whereas the bays along the opposite guide banks are overtaxed being opposite the deep channel (Fig.2) entailing river training higher up.

1.7 The flow approaching and dividing around the nose of the blunt pier in addition to being curved in plan, acquires a downward diving componant and as it straightens out by reversing plan curvature along the sides of pier it acquires a rising componant after leaving the wake of the pier. The diving flow at the nose causes scour and the rising flow plus spiral roller inside the scour hole throw out the scoured material from the scour hole.

The pressure field induced by the blunt pier if sufficiently strong, it causes a three dismensional separation of the boundary layer which in turn, rolls up ahead of pier to form horse-shoe vortex system(Fig.3).The pressure gradient is in the downward direction and so the secondary current is also in the direction of decreasing pressure. Mr.Hung (1968) made a detailed velocity and pressure distribution near a circular cylinder in an open channel. The pressure coefficient defined below is plotted in Fig. 4 which shows that the pressure gradient at the front face is in downward direction

$$C_p = \frac{p - p_y}{\frac{1}{2} \rho U_y^2}$$

where p is local measured pressure

p_y is the upstream undisturbed pressure at

y (measured from bed), ρ is the fluid density

U_y is the undisturbed flow velocity at level y.

The horse-shoe vortex system is not steady for all flow conditions and are shed periodically. The shedding is observable during scour as slugs of sediment being pulsed around the pier.

Some pier shapes, such as wedge or lenticular may be either blunt nosed or sharp nosed depending on the wedge angle and the angle of attack of the undisturbed approach flow. In case of sharp nosed pier the horse-shoe vortex system is very weak and wave-vortex system is strong. The wake vortex system is formed by the rolling up of the unstable shear layers generated at the surface of the pier (Fig. 3). The wake vortices are shed alternately from the pier and are convected downstream at Reynolds Numbers of practical interest. The strength of wake vortices is a function of the pier shape and the fluid velocity. The wake vortex system acts somewhat like a vaccum cleaner in removing the bed material. The secondary flow and the vortices in the wake of the pier cause scour depth near the downstream end of the pier whereas the strong horseshoe vortex system and downward secondary flow at the nose of the blunt nosed piers cause scour at the pier nose.

The influence of an angle of attack has been studied by Laurson and Toch who observed that the maximum scour at the pier is influenced by the angle of attack. A pier consisting of two or more circular piles is an attractive one as in case of an appreciable angle of attack, scour is not influenced by the angle of attack for centre line spacing of two circular piles longer than three times the diameter of the piles.

1.8 Bridges should be designed to withstand the scour at pier and abutment foundation that could occur during the maximum probable flood. To accomplish this the designer must first estimate the magnitude of the maximum probable flood, then predict the most adverse flow pattern at the bridge for that flow and finally predict the scour that will occur at the piers and the abutments. By designing for the maximum probable flood, the risk of failure is reduced to that entailed by the error in estimating the flood, the error in assessing the worst flow pattern and the error in predicting the scour. The risk must be almost nil because the cost of failure is many times the cost of insuring against failure at the construction stage.

1.9 The scour that may occur at a bridge water way can be categorised as follows:

a. The scour that would occur in the stream with or without a bridge crossing. Degradation may occur due to downward migration of a meander, the shift of thalweg line of a braided stream or a natural or a man made cut-off and consequent straightening of river course, the result of augmentation of the flow or the reduction of sediment supplied to a reach (downstream of a barrage or a dam). The stream bed may lower considerably due to degradation, therefore, any likelihood of this type of action should be assessed in the Hydraulic Design of a bridge. The aggradation of the bed would increase, the flood plain flow at the cost of main channel and hence greater concentration of flow at the abutment or guide bank head. The aggradation would mean the raising of the water surface and consequently inundation of more areas, lesser free board in the flood embankments and hang up of trash on the lower members of the bridge.

b. The scour that may occur generally at the bridge water-way because the flow is contracted by the bridge crossing. Some designers consider that a comparatively tight water-way is hydraulically superior in performance than a generous water way in which case the river tends to form a deep channel in part of the wider water-way and sandy bela upstream of water way which in turn tends to mask the bridge opening especially when covered with vegetation. The scour is a function of depth of flow and discharge per foot run as will be seen in subsequent paras. The scour is a function of $q^{2/3}$ where q is the localized discharge per foot run and so greater the local q the greater is the localized scour.

c. The local scour that occurs because of the distoration of flow pattern in the immediate vicinity of the bridge piers.

The local scour pattern is a result of flow pattern (which is a function of river approach, ratio of over bank discharge to main channel discharge, approach depth, geometry of constriction) and the mode of sediment movement and the variation in sediment transport capacity from point to point in the stream.

1.10 The role of model as a tool into investigation of several features of the water-way problem such as location, orientation, length and vertical clearance of bridge, scour at the abutments and piers, shape and geometry of guide banks cannot be over-emphasised.

The new bridges on Ravi at Shahdara, Mari Pattan, and Sidhnai, new bridge on river Chenab at Wazirabad and proposed bridge at Talib-wala, new road bridge on Jhelum river near Jhelum and the proposed bridges on Indus at D.I.Khan and D.G.Khan and at Moro-Dadu, and new bridge at Thatta-Sajjawal, new bridge on Sutlej near Adamwahan were model-tested at Hydraulic Field Research Station of Irrigation Research Institute.

Basic study on optimum water-way of bridge on fluvial channels and the optimum shape and geometry of the guide banks was taken up under Irrigation, Drainage and Flood Control Research Council (I.R.C.). The information and the material given here is based on Laboratory investigation on specific bridges and the basic study funded by I.R.C.

2.0 MINIMUM WATER-WAY OF A BRIDGE FROM HYDRAULIC CONSIDERATIONS

The clear water-way width of a bridge is determined from considerations of design flood discharges overbank flow, afflux, scour at abutment and piers and economy but it should not be less than the minimum required.

The water ways of bridges in Pakistan on main alluvial sediment carrying rivers are listed in Table I.

The ratio of actual water-way of a bridge and the Lacey's regime width for the design flood known as looseness factor is also given in Table -I. The looseness factor is as low as .28 for a bridge located in a rocky gorge. The looseness factor of bridge in alluvial reach of river Ravi located at Abdul Hakeem is as low as .44. The maximum value of looseness factor is as high as 1.71. About 50% of bridges in the alluvial reaches of the river have a looseness factor near about one.

The actual water way of the bridges with looseness factor one is less than given in the Table-I as the guide banks are encroaching on the side spans. No wonder if breaches in approach embankments or flood embankments of these bridges had occured. All the 5 bridges on Ravi were found inadequate during rare or even major floods in the years 1955, 1973, 1975 and 1976 floods. Out of 4 existing bridges on river Chenab three were found inadequate and natural breaches had occurred in the road and or flood embankments of those bridges during 1957, 1959,1973 and 1975 floods. Shershah bridge was found adequate as it had a looseness factor of 1.47.

To avoid traffic hazards and to lower flood heights upstream of bridges it is very vital that the water way should not be less than the minimum required for safe passage of anticipated floods.

There are cases where waterway is adequate but flow section within guide bank and the approaches to the guide banks are not adequate.

The experiments conducted on the model simulating river Ravi at Balloki and the barrage plus its ancilliary works on 1/150 horizontal and 1/25 vertical scale indicated that:

a. The sarkanda within the broken guide bank on both flanks of river formed by the spur 10, 4, 3, 6 and 5 on the right side and spurs 1,2,8 and 9 on the left side accretes the water levels above broken guide bank by 1.8 feet at a design flood of 2,25,000 cfs.

b) The straightening of river by cut off across two bends above spur No.6 & 5 (Fig. 5) results in lowering of flood heights by 1.8' at the design flood.

The flood levels generated during last two wet cycles in a period of 35 years have further indicated that:-

a) The afflux may not be due to tight bridges or barrages but due to channel deterioration, excessive vegetation and increased roughness of bed.

b) During dry cycles the flow section of a river gets restricted consequential to aggradation of river beds, due to maximum possible silt free withdrawals at barrages, and inadequate releases downstream of the barrages for transport of silt to the sea and growth of vegetation over the belas within the guide banks of bridges and barrages and in the Khadir of river beyond the control structures.

c) There is excessive accretion of water levels in the first rare or super flood after the dry spell (see Figs. 6.1 to 6.5 relating to control points on the river Chenab). There is substantial retrogression of water level in the next major flood if it recurs in a short span of one to two years.

2.1 The optimum water-way of new Road bridge on River Jhelum near Jhelum Town

The existing rail-cum-road bridge constructed in 1872 near Jhelum town consists of 50 bays of 90.0' span and the distance between the two abutments of the bridge is 4962 feet. The present road way is 14' wide and has no foot paths and it is severely conjested.It was structurally inadequate for modern heavy traffic and cannot be widened and strengthened economically therefore a new road bridge on vital Pakistan Highway was an immediate necessity.

To meet this demand a 3230 feet long bridge with super structure of prestressed concrete 4000 feet downstream of old rail-cum-road bridge (Fig-7.1) was opened to traffic in 1968. The bridge has been designed for a discharge of 900,000 cfs and no account of Mangla Dam's flood absorption capacity was taken as chances of a major flood when the reservoir is full cannot be ruled out.

The site is underlain by stratified layers of sand gravel and boulders in different proportions varying erretically. At a depth of about 40-50 feet the samples contained upto 40% of cobbles and boulders upto 8" size.

Irrigation Research Institute carried out test runs for determining the length of water way required to pass a discharge of 900,000 cfs without causing undue afflux and orientation of axis of new bridge in relation to the axis of old bridge on a model that simultated the prototype on 1/200 horizontal and 1/36 vertical scales and representing a river reach 7 miles upstream and 3 miles downstream of the existing bridge. The model was distorted to increase, hydraulic radius, turbulance and velocity of flow on the model.

It was noted that in the absence of new bridge as a consequential affect of right bank spurs above existing bridge constructed to protect the Jhelum town, the flow at bank full stage is deflected left of the axis of the Railway bridge and at higher stages the main currents shifts towards right bank.

To determine the minimum water way required for the new bridge (4000 feet downstream of existing bridge) tests were conducted with new bridge of 24,23,22,21 and 20 bays of 150 ft. span (most economical span length). To start with a bridge comprising 24 bays with its axis parallel but 150' left of the axis of the existing bridge was model tested. The number of bays were successively reduced by one bay along the right abutment by shifting the right abutment towards the axis of the new bridge. The water levels observed upstream of the existing bridge for peak discharges of 50,200,300,400,500,600,700,800 and 900 thousand cusecs plotted and compared with those in the base test without the new bridge in Fig.-7.2, indicate:

a) With 24 bay bridge the rise of water levels at the old bridge is insignificant and the corresponding rise for 23 bay bridge is .2 to .3'.

b) With 21 bay bridge the increase in flood heights is quite significant the rise being one foot for discharges above 5 lacs and the corresponding rise is 1.5 foot for 20 bay bridge. The rise of water level for 20 and 21 bay bridge is more at discharges below 500,000 cfs than at discharges above 500,000 cfs.

c) With 22 bay bridge (looseness factor of 1.22) the rise is of the order of .4 to .5 as compared to water levels in the base test. In view of some expected retrogression below Mangla Dam 22 bay bridge was recommended.

The difference in bed levels along the right and left guide banks was 20, 12.5, 9.0, 7.5 and 5.5 feet respectively with 24,23,22,21 and 20 bay bridges as recorded after running one hydrograph peaking at 500,000 cfs after fresh moulding of model river for each test run.

The discharge passing through different bays was measured at a river stage of 500,000 cfs (See Table II) which shows that with 24 bay bridge the concentration was in extreme left bays but shifts in wards by two bays with 22 bay bridge.

2.2 The minimum water-way of old Railway Bridge on River Chenab near Wazirabad

The Alexandra bridge situated 9 miles upstream of Khanki H/W when originally constructed comprised 64 spans of 132.0 feet. The existing water way has 17 bays and due to encroachment of side bays by guide banks the effective water way is in between 15 and 16 bays.

The flood levels recorded at Alexandra Bridge corresponding to the peak flood discharges during the last 75 years at Khanki Headworks, situated 9 miles downstream, were as follows:-

Year	Peak Flood discharge Lakh cs.	Flood Level	No.of bays
1903	7.03	752.2	28
1917	4.24	753.1	28
1950	10.10	753.6	17
1954	8.07	752.8	17
1957	10.96	755.3	17
1959	10.21	753.6	17
1966	6.30	751.0	17
1973	10.00	756.0	17
1975	6.66	753.6	17
1976	6.15	753.2	17

The crest level of Khanki Headworks consisting of 8 spans of 500 ft each was raised by 2 ft in bays 4,5,6 in 1910-11, and in bays 1,2,3,7,8 in 1916-17.

It was raised further by 2 ft. in 1919-20 to the existing crest level.

Bays 4&8 were depressed by 12 ft, and split up into 18 spans of 20 ft. each and gated in 1934.

The designed capacity after 1934 remodelling becomes 800,000 cfs.

The rise in the flood levels in 1917 above those in 1903 was due to the back-water effect caused by the raising of the crest level of Khanki Headworks in 1910-11 and 1916-17.

The fall in the flood levels in 1959 and 1975 below those in 1957 and 1973 was due to the degradation of the river bed during the high floods in 1957 and 1973.

The generation of the higher flood levels in 1957 and 1973 was due to the constriction and obstruction of the flow section as a consequence of the aggradation of the river bed and growth on balas during the period of dry spell preceeding the high floods.

The river bed on the river side of Haripur Bund has silted up whereas the bed levels in the pocket lying in between the Haripur Bund, railway line and Wazirabad-Sambrial road are low, and the flow of Palkhu and Aik nullahs passes through this low-lying area. When the Haripur bund gives way the excessive flow from Chenab river into this low pocket results in the overtopping and breaching of the railway and road embankments. The total discharge through Alexandra Bridge and Spill and Palkhu bridges and the breaches in 1973 was assessed by Pakistan Railways to be of the order of 7.64 lakhs cfs out of which 4.40 cfs passed through Alexandra Bridge, and 3.24 lakhs cfs passed in between the Haripur bund and Wazirabad town. The breached length of the Haripur bund and the railway and road embankments was 3,540 ft and 1,850 ft respectively.

It is evident that the strengthening and extension of the Haripur bund to Sohdra village will cut off substantial component of the spill flow from Chenab River and Palkhu nullah, and the Spill and Palkhu bridges will have adequate waterway to take care of the flow from Aik nullah and any spill flow from Chenab river upstream of Sohdra village.

2.2.1 The findings (Fig. 8.1 to 8.5 and 9.1 to 9.3) of the hydraulic model study are summarised below:-

a) the discharging capacity of Alexandra Bridge has been assumed as 10.0 lakh cs on the basis of the designed capacity of Marala Barrage.

b) the waterway has been adopted as 3,204 ft, i.e. 1.2 x Lacey's width.

c) the 1919 designed H F L 754.00 is attained at the designed discharge 7.11 lakh cs with 9 additional bays.

d) the highest recorded H F L 756.00 is attained at 10.0 lakh cs with 8 additional bays.

e) the 1957 flood level 755.30 is attained at a discharge of 10.0 lakh cs with 12 additional bays.

f) the maximum recorded flood discharge at Khanki Headworks minus Bhimber nullah discharge is 9.0 lakh cs, and at this the discharge flood level gets lowered to the 1957 flood level 755.30 with 8 additional bays.

g) the rate of fall of the flood level at 10.0 lakh cs decreases beyond the addition of 8 bays.

h) the still water levels behind the Left and Right Guide Banks are higher than those generated at the railway bridge.

The water levels in the left pocket are higher than those in the right pocket which indicates that the main stream shifts towards the Left Guide Bank during the floods under the existing approach conditions.

j) at 10.0 lakh cs the water level in the left pocket is higher by about 3.5 feet than that at the railway bridge (pier 3) even with the addition of bays as follows:

additional bays	ft
0	3.8
4	3.6
8	3.4
12	3.5

This head is utilized in generating higher velocities in the long contraction within the guide banks.

k) the water levels at the farthest end of Haripur bund (extended) show that the pre-extension level 762.00 at 10.0 lakh cs is attained with 14 additional bays.

l) the dispersion of flow with 8 additional bays is better than with 12 additional bays as the discharging capacity of the Left and right hand bays decreases with the addition of 12 bays, and decreases still further with the addition of 14 bays.

m) the velocities generated along the Haripur bund in the basic test, and with additional bays, show that the velocity gets boosted up by providing more waterway on the left flank and as a consequence a J-spur and an L-Spur are necessary for the protection of the Haripur bund.

n) besides checking the velocity of flow along the Haripur bund, these spurs also improve the entry conditions at the upstream end of the Guide Banks as any embayment behind the Left Guide Bank gets eliminated and the main stream is pushed towards the axis of the railway bridge.

2.2.2 Recommendations:

a) 8 additional bays need to be opened up on the left flank to render the Alexandra Railway Bridge capable of discharging 10.0 lakh cs, and to lower the flood levels upstream.

b) Left Guide Bank has to be re-aligned at the end of the 8 additional bays.

c) Right Guide Bank has to be re-designed to conform to the Left Guide Bank.

d) Haripur Bund has to be strengthened and extended to Sohdara village as a full-fledged flood embankment and protected against parallel flow by means of a J-spur and an L-spur.

e) The new Chenab road bridge downstream, as well as the old one, shall need to be extended on the left flank simultaneously with the Alexandra Railway Bridge, and the Left Guide Bank re-aligned.

2.3 Model tests for determination of optimum waterway for Thatta Sujjawal Bridge across Indus River.

Laboratory experiments for the existing straight approach and angular approach forming heavy embayment behind right guide bank for 1956 actual hydrograph with a superposed peak of 11,00,000 cfs conducted for three waterways 4,000, 3,600 and 3,200' (Looseness factor of 1.43, 1.287 and 1.144) with guide banks of the same shape but the lengths of straight portion of guide bank cut short by 400 & 800 feet for waterways of 3,600 and 3,200 feet respectively, indicated:-

a) The afflux caused by bridge width of 4000' 3,600' and 3,200' for straight channel at 11,00,000 cfs is .71, .86 and 3.12 feet respectively against 1.0', 2.0' and 2.4' for oblique approach.

b) For straight and oblique approaches the concentration of discharges is opposite the deeper section of the channel even at 655,000 cusecs (the peak of actual hydrograph). The maximum concentration of flow for oblique approach over and above the average theoretical discharges is 229, 260 and 270 percent for waterway of 4,000, 3,600 and 3,200 feet respectively and the corresponding concentration of flow for straight approach is 172.5, 157 and 154.5 percent . The concentration of flow in the bay facing the bela is 18.3, 18.7 and 45.2 percent of average flow for the waterway of

4000, 3,600 and 3,200 feet and for the oblique approach the corresponding values of concentration are 50.2, 48.3 and 39.2 percent.

c) The maximum scour for the oblique approach for the three waterways of 4,000, 3,600 and 3,200 was 63.55, 68.5 and 79.5 feet and the corresponding values for straight approach are 49.2, 54.8 and 57.8'.

d) It is evident that in case of deep embayment behind one of the guide banks, the flow dispersion at the bridge is controlled more by the embayment behind the guide bank nose and lesser by the water way. In case of straight approach the flow dispersion is controlled by the waterway.

e) The minimum waterway for Thatta Sajjawal bridge is 3,600 feet i.e. or the minimum looseness factor is 1.28' from consideration of scour and discharge distribution.

2.4 Basic Study for determination of minimum waterway of a bridge on a fluvial channel

2.4.1 Experimental Set-Up

The study was taken up in a new experimental tray 50' x 25' (excluding the feeding and stilling tanks) fully equipped with facility of moving bridges carrying instruments, tail level control and head box for accurately feeding measured supply of discharge. The tray was filled with clean Ravi sand of d_{50} = .19 mm, the specific gravity of 2.67 and angle of repose = 33^{o} and 15'. The experimental tray was provided with bed-filter allowing the tray to be dried up quickly after each test.

The instruments mounted on the tray trollies, consisted of sensitive pointer gauges (a water level sensing probe) fitted with electric water level indicators, pittot tubes for velocity measurements fitted with manometers, bed and surface electronic recorders. The straight edges supporting the wheels of moving bridges carrying instruments were checked from time to time by a sensitive survey level for ensuring correct measurement of water level.

2.4.2 Alignment of Model rivers

Characteristics of the fluvial channels used for model experiments are described as under:

The shape of the channel was sine curve with equation:

$$Y = A \sin \frac{2 \pi X}{M_L} \quad \ldots\ldots\ldots\ldots\ldots \quad (21)$$

where X is the distance along the channel axis (i.e.Khadir axis), Y is the distance along transverse direction. M_L is the meander length adopted according to Inglis equation derived from Indo-Pak Sub Continent prototype river data as:

$$M_L = 25 \; Q^{.5} \quad \ldots\ldots\ldots\ldots\ldots$$

Meander length was kept fixed at 22.4 feet to correspond to a discharge of about 0.8 cusecs. The sine curve used can give channels of various sinuousities by varying the amplitude (A), such that meander belt $M_B = 2A$, The fluvial channel cross section(moulded in sand of d_{50} = .19 mm) had a flow depth of .25 feet, bed width of 1 foot with side slopes of 1:1 and bed slope of 1/500 designed for bankful stage of 0.25 cusecs (Fig-10.1). The flood plain had a width of 6 times the bed width on each flank of the main channel.

Characteristics of six model channels

Channel No.	A	M_B	M_L	M_B/M_L	
1.	Straight. Approach.		-	-	-
2.	0.7 ft.	1.4 ft.	22.4 ft.	1/16	1.004
3.	1.4 ft.	2.8 ft.	22.4 ft.	1/8	1.071
4.	2.8 ft.	5.6 ft.	22.4 ft.	1/4	1.16
5.	5.6 ft.	11.2 ft.	22.4 ft.	1/2	1.45
6.	Forming extreme embayment behind one of the guide bank so that the channel encroaches on approach embankment.				

$$= \frac{\text{The distance along the channel between the two points}}{\text{The air-line distance between the same points.}}$$

The central lines of the six channels as aligned on the model are shown in Fig-10.2

2.4.3 Hydrograph

A hydrograph Fig.10.3 of discharge varying between 0.15 and 0.8 cusecs with bankful stage discharge of 0.25 cusecs and flood peak of 0.8 cusecs, was adopted for the study.

2.4.4 Bridge water ways tried on the model

The bridge line was selected at the middle of the 3rd cross-over of two 'S' channel. The geometry of the guide bank tried on the model is shown in Fig. 10.4. The actual dimensions of guide banks varied with waterway being tested. The projected length of the guide bank on the axis of the bridge is only 60% of L (the total length of bridge). The noses are diverged out such that the distance between outer tips of guide bank is 1.5 L.

The looseness factor K, of the bridge is defined as:

$$K = \frac{\text{The actual waterway between the guide banks}}{\text{Lacey's P-Value.}}$$

The value of P as calculated from the equation $P = 2.67\sqrt{Q}$ for maximum discharge Q = .8 cusecs is 2.39 feet. Eight values of looseness factor (K) tried were K = 0.60, 0.8, 1.0, 1.2, 1.4, 1.6, 1.8 and 2.0 and the corresponding waterways are 1.43, 1.92, 2.39, 2.87, 3.34, 3.84, 4.30 and 4.78 feet respectively.

Two approach embankments were placed to block the Khadir width not occupied by the bridge.

For the above values of K, the ratio of bridge length and Khadir width varied as 0.86, .12, .16, .18, .20, .22, .24, and .26 respectively.

The bridge piers were eliminated, as local scour at the bridge piers would have complicated the interpretation of results and to know the scour constant for different contractions. (The local scour at bridge piers will be discussed in paragraph-4.0

2.4.5 Tests & Observations

In Series 1-6 (besides initial trial tests) in all 48 tests were conducted under six series of tests (8 tests for each channel of a particular sinuosity). There were 6 different channel sinuosities as described earlier.

The observations recorded consisted of the velocity profiles at the bridge line and at the apex and cross over of bends, water surface slopes, bed configuration after the running of one complete hydrograph, the scour at the guide bank nose and the bed profiles. The maximum scour at the bridge at peak discharge was recorded with bed electronic senser. The head-loss across the bridges, the afflux, caused by the bridge, the depth of maximum scour, velocity head, mean velocity, the variation in discharge per foot width from abutment to abutment, scour constant K_1, the parameter $\alpha = \frac{h}{V^2/2g}$ and other parameters were computed.

2.4.6 The channels alignments tried on the model can be split up into categories.

Category I channel approaches of zero or low sinuosities. Category II channels approaches of sinuosities forming deep embayment at one of the guide banks (left) when the flow is fully captured at one of the guide banks and it does not kick off the flow completely on to the other guide bank. The flow splits up into two currents after leaving the captured nose of the guide bank (See Figs.10.5 and 10.6). The one current follows all along the side of captured guide bank facing the axis of bridge and the strength of this current increases with stage and sinuosity. With increase in sinuosity and river stage the discharge in the channel decreases and the flood plain discharge increases and the flood plain discharge enters the guide bank right of the axis of bridge and keeps off the deflected current from left guide bank nose or meets it right along right guide bank resulting in scour hole at and below the meeting point.

The strength of the deflected current from the captured guide bank nose is again a function of river stage and embayment behind the captured guide bank which sweeps off the captured flow to impinge on the opposite guide bank (the severity of impingement decreases with increase in the value of K). The flood plain over-bank flow also forms some embayment at high stages at the nose of right guide bank. The channel No. 5 and 6 with sinuosity of $^{M}L/2$ and the channel forming extreme embayment behind the guide bank belong to this category(III).

Channel No.1 is the ideal channel alignment difficult to achieve and maintain on the prototype. So the channel alignment No.2,3 and 4 are more probable alignments for conventional diverging out guide banks. For these approaches the channel discharge does increase proportional to the total flow and the embayment behind the guide banks remains effective in kicking off the main current on to the opposite bank. The behaviour of channels 2,3 and 4 becomes like that '1' described above for alignments 5&6 for shape of guide banks recommended by Irrigation Research Institute.

2.4.7 The head across the bridge, the velocity head and the afflux caused by the bridge shown in Fig. 10.7, 10.8 and 10.9 indicate:-

a) The head across, velocity head and the afflux are maximum at the bridge at K value of.8.

b) The sinuosity exercises great influence on the head across, velocity head of afflux and so the training of river upstream of guide bank is called for attaining a straight and head on approach to the bridge.

c) The rate of fall of afflux with increase in the value of K is rapid upto the value of K = 1.2 and almost linear for K values in between 1.2 to 2.0. The rate of fall of afflux is different for six alignments indicating different flow pattern for different approach alignments.

d) The fall of head across the bridge is almost linear for the approach channel 2-6.

e) The rate of fall of velocity head with the K value is linear for alignment No.2 and 5 and for alignment 1, 3 and 4 the curves are parallel with uniform rate of fall upto K value = 1.4 and lesser for K values 1.4 to 2.0.

2.4.8 The value of K in the equation $D_s = K_1 q^{2/3}$ at the bridge for maximum scour observed electronically during the run of hydrograph for different K values and plotted in Fig.10.10 shows that the maximum value of K is 2.0' increases with sinuosity of channel and decreases with increase in the value of K and for approach channel 2, 3 and 6 is not influenced by K beyond K value of 1.4. Fig. 10.10 will be used to estimate the value of K_1 for determination of D_s at the bridges.

2.4.9 The maximum localized discharge intensity at bridge line plotted against K in Fig. 10.11 indicates that the maximum concentration q_{max} is opposite the main channel and is a function of flow pattern while approaching the bridge and is more a function of conditions of flow at the entry into the guide banks. So long as the embayment behind the captured guide bank is effective in kicking off the current the concentration of flow is independent of the length of waterway, similarly the q_{min} Fig. 10.12 is not dependent on the length of waterway and is governed by the bed configuration within the guide banks.

2.4.10 The bed contours at the end of the test run in the six series of tests are plotted in Fig. 11.1 to 11.6 indicate:-

a) The unevenness of the bed contours is minimum or the flow is more central at value of K = 1.2 for all sinuosities.

b) The scour at the left guide bank head is a function of the degree of convergence of currents and depends more on the disposition of captured guide banks in relation to embayment behind it and is not a function of K value.

c) The maximum scour at the captured guide bank decreases with increase in the length of waterway for channel No. 1 & 2 whereas for other approaches the maximum scour at the nose of captured guide bank increases with length of waterway as with fixed axis of the bridge and alignment of the channels the concentration at the captured guide bank increases by shifting the guide banks away from the axis of the bridge towards the apex of bend behind the captured guide bank.

d) The scour at the bridge line is significiently less than scour at the captured guide bank nose and occurs at the point where either the discharge per foot run is maximum or where the degree of convergence of currents is maximum.

2.4.11 The experiments show that the channel capacity decreases at higher sinuosities at higher stages.

In case the embayment behind the captured guide bank is very deep the river should be trained higher up to minimise the action at the guide bank and to protect the approach embankment.

2.4.12 The minimum value of K is 1.2 from Hydraulic considerations.

3.0 LENGTH, SHAPE AND GEOMETRY OF GUIDE BANKS OF BRIDGE

3.1 Mr.Bell recommended that the guide banks should be brought closer together at their upstream ends than at bridge site as he thought that the Vena Contracta as a consequent of its much larger effective area of cross section than that at the bridge is likely to centre the flow and make it fan out equally in all the spans, thus ensuring better dispersion of the flood flows between the abutments.

In case of weirs and barrages the shape of the guide banks has a bearing in carving out a suitable approach to the canal pocket thereby affecting the conditions of silt laden water filaments entering off-taking channels.

The bottle neck guide bund system constructed at Sulemanki Headworks were incompatible to good sediment exclusion as large islands have always formed upstream of canal pockets.

Model experiments in Irrigation Research Institute for Kalabagh Headworks indicated a little superiority of slightly diverging type of guide banks over other types.

The approach channel to pockets is primarily governed by the river approach upstream of the guide banks and the interface and area of contact between the moving water and the solid walls of the guide bank, and the large islands within the guide banks are the ancilliary factors controlling the entry of silt in the off-taking channels.

Sir Claude Inglis probably being scared by the fact that Belly shaped guide bund system alone would not help in forming suitable approach channels along the two pockets for all the river approaches has suggested alternate canal heads at the heads of the guide bunds so that if flow is captured at the head of left or right guide bank, the left or the right off-take canals will be drawing from outside of the curve at the head of the guide bank while the right or left off-taking canals will be drawing at the outside of bend at the pocket regulators.

3.2 The diverging out and Belly shaped guide banks for Guddu Barrage Fig. 12.1 were tested in depth at Irrigation Research Institute for different approaches (Fig. 12.2). Capturing the left or right guide bank nose or straight and perpendicular to the barrage. The findings of the tests with still pond system and offtaking channels drawing their full supply discharge are:-

a) For similar approaches embaying at or behind the guide bank nose the shoal formation along the front face of the captured guide bank is higher for Belly-shaped guide bank then the diverging out guide bank resulting in more masking of bays of bridge or barrage along the captured guide bank for Belly shaped than for diverging guide bank.For the same reason the silt entry in canal offtaking on the side of captured guide bank side is more for the Belly shaped guide bank than the diverging out guide bank.

b) For similar approaches embaying at or behind the captured guide bank nose the scour along the opposite bank is more in depth and extent if the opposite bank is diverging out type than the Belly shaped guide bank as due to its protrusion it receives severe beatings of the impinging currents approaching it at an angle and the severity of beatings increases with angle of attack. The concentration of flow in bays facing the scour along the guide bank will be more for diverging out guide bank than for the Belly shaped guide banks. For the same reason the silt entry in canals on the side of guide bank subjected to the severity of attacking flow from the nose of the opposite bank will be less for diverging out guide bank than for the Belly shaped guide bank.

Due to reflections of currents from the Belly shaped guide bank acting as concave bank, the dispersion of flow within the guide bank (excepting the bays facing the shoal) is better for belly shaped guide bank than for the diverging out guide bank and so from the stand point of localised scour at the piers it will be less for the belly shaped guide banks than for the diverging out guide bank as the localized degree of concentration will be lesser for belly shaped guide banks.

c) At higher stages of river the flow will drift away from the embayment behind the guide bank earlier for deeper embayment than for a milder embayment.

d) The most suitable position for the canal off-taking regulator is downstream-ward of the point where the diving flow along the concave bank after reaching the bed of channel is deflected towards the convex bank.

e) The ratio of velocities along the two flanks of divide wall at the nose of the divide wall determines the silt going into the pocket. The greater the tilt in the velocities on the river side of the divide wall the lesser the silt entering the pocket and hence the regulator.

f) The heads of the two guide banks should be as apart as possible so that in case one of the guide banks is captured the embayment behind the guide bank, however, deeper should not deflect the currents to impinge on the opposite bank.

g) The body of the guide bank or any portion of it should not obstruct the path of flow currents within the guide banks.

h) The shape and the length of the guide banks should be such that at no stage shoal should form in the vicinity of pocket of the barrage or approaches to the side bays of the bridge.

3.3 Length & Shape of guide bank for Thatta Sujjawal road bridge.

Though the waterway recommended by Irrigation Research Institute was 3,600 feet a width of only 3,200 feet was adopted. The reasons for reducing the bridge width as given in the design report are enumerated below:-

a) The main current of the river which had been attacking the right bank for several years upto 1961, has shifted away from right bank and no significant erosion has been observed since that year.

b) The frequency of occurrence of the design flood of 1,100,000 cusecs is likely to be reduced as a consequence of various control structures on the river Indus further upstream.

c) The right bank is stabilised and the main channel approaches the bridge in a straight course, normal to the bridge axis.

As the clear waterway of the bridge has been reduced by 400' some care is required in the determination of the shape and length of the guide banks, to achieve uniform approach flow to the bridge and allow for embayment behind these guide banks without affecting uniform flow to the bridge.

The tests were conducted on a model representing 10 miles of the river Indus (5 miles upstream and 5 miles downstream of the bridge) on 1/300 horizontal and 1/40 vertical scales to simulate the latest survey with a straight approach to the bridge.

The tests comprised two series. The first series of tests concerned with the optimum length of the guide bank and in this series of tests the length of the left guide bank was kept fixed at 5,000 feet and the length of the right guide bank was varied, e.g. 5,000, 4,000, 3,500, 3,000 and 2,200 feet measured along the right bank Fig. 13.1. The upstream head of the right guide bank was in all tests given a circular shape with R = 1000' and covering a 120° segment of a circle. The over-bank flow from the flood plain on the right side converged at the nose of right guide bank and the degree of convergence increased with shortening of the guide bank. The maximum scour observed at the right guide bank head after run of one hydrograph of 1956 with super imposed discharge of 1,100,000 cfs. was observed to be R.L.+23.0, 15.0, +13.5, +9.0, +2 and -8.0 with corresponding right guide bank length of 5,000, 4,000, 3,500,3,200 3,000 and 2200 respectively indicating that the scour increased 31 feet by shortening of right guide banks from 5,000 - 2,200 feet.

The discharge distribution in different bays of the bridge with different guide bank lengths indicated that with the longer guide bank there is a greater concentration of flow in the right hand bays. With shorter guide banks the embayment above the guide bank nose tends to deflect the main current away from the right abutment and thus the reduction in length makes the flow distribution more symmetrical through the bridge as the main channel becomes more central.

In test series No.2 the projected length of guide banks on the axis of the bridge was kept constant at 3,000 feet and the upstream nose of the guide bank was shifted back on a line perpendicular to the bridge axis 3,000 feet upstream of bridge line at intervals of 0, 100, 200,300,400,600,750,900,1050, and 1200' (Fig.13.2) The entire guide bank was realigned in each case to form a parabola with upstream nose as its apex. The maximum scour at the nose of right guide bank observed after run of one hydrograph was at El. +13.0, +12.0, +11.0, +10.5., +7.5, +5.0, +1.5, -2.0 and -6.5 for the shift of guide bank laterally by 1200, 1050, 900, 750, 600, 400, 300, 200, 100 and 0 feet respectively.

From the second series it follows that with the setting back of the guide bank nose the scour goes on reducing and is almost stabilized with the guide bank nose shifted back by about 1000 feet from the abutment line. No appreciable change in the bed cross sections at the bridge or discharge distribution across the bridge was observed on the model.

3.4 Length and shape of guide banks of road bridge on River Sutlej near Adamwahan

The railway bridge over Sutlej river near Bahawalpur known as Empress bridge is badly sited, as the river approach is oblique and heavy embayment has formed behind the right guide bank nose resulting in concentration of flow in the left spans of the bridge and choking of two end spans on the Multan side. The present right handed approach above the Empress bridge is persisting after the addition of Bell's guide banks and closing of 8 bays (250 feet span) out of 16 bays on the Multan side (right side) in 1930 because the axis of bridge is located left of the river axis. In 1953 floods the deepest part of the embayment behind the right guide bank came within 800 feet of Railway embankment. The embayment is held in position by three mole spurs (Fig. 2).

The new road bridge (replacing the boat bridge) an permanent crossing facility located 3,000 downstream of the Empress bridge was opened to traffic on 26-12-1968. The location of the bridge, its water way and the shape and length of the guide banks were decided as a result of detailed model study for a design discharge of 400,000 cusecs (whereas the Mailsi syphon higher up is designed for 350,000 cfs). The recommendations of Irrigation Research Institute as a result of model studies which were practically all incorporated were:-

a) To cater for both right handed and left handed approaches a 12 bay bridge (160' span) placed symmetrically to Empress bridge at a distance of 3,000-3,200 is quite adequate from hydraulic and economy considerations.

b) The optimum length of guide banks projected on the axis of bridge is 1,200.

c) The optimum shape of the guide banks is an ellipse with major to minor axis ratio of 4:1. The length of guide banks was actually kept 1,500' keeping in view the advice of Sir Thomas Foy who remarked "I am not prepared to accept the short guide bank of the order of 1,000 feet length under any circumstances as experience has shown them to cause trouble".

In view of benefits to be derived from the existing protection walls of Empress bridge the rules of design of guide banks from Bell, Spring onward are not applicable in the present case. It was apparent from the flow pattern and scour pattern that the embayment behind the guide banks of road bridge coupled with convergence of flow at the noses of guide banks decreases as the length of guide banks increases and with 1000 feet long guide bank the flow starts skiring along the noses of guide banks without any embayment and with 1200 long guide banks the degree of convergence of flow at guide bank noses is zero.

3.5 Dikes employed by the Mississippi State Highway Department

The Mississippi design requires a minimum length of 150 feet. Undoubtedly, individual sites can be adequately protected by shorter dikes but many may require more length. Of all the shapes tested by Mississippi State Highway Department, the elliptical shape with a 2.5 ratio of major to minor axis provides the best over all result.

3.6 Basic Study for determining the length & shape of guide banks

The facility of tray created for determining the optimum waterway was utilised for determining the length and shape of guide banks. For this series of tests the bridge water way had a looseness factor K=1 and the channel sinuosity $M_B / M_L = 1/2$.

3.7 Straight Guide Banks

Irrigation Research Institute conducted some experiments for exclusion of silt from Mughan Canal off-taking from Mill Mughan Barrage on river Aras a common river between Iran and Russia. The design of barrage Fig. 14 was prepared by U.S.S.R. Ministry of Power & Electrification. It is clear from the Fig. 14 that the length of the straight guide banks is 5 times the barrage width.

To start with the guide banks were kept straight but their length was changed in different tests. Guide bank lengths 1.0 L, 1.5 L, 2.5 L, 3.0 L, 3.5 L and 4.5 L Fig. 14.2 were tested. The maximum length approached one half of the meander length and was sufficiently long to cut off one river bend completely and force the flow from the cross over to take a straight run to the bridge.

The bed configuration within the guide banks is shown in Fig. 15.1 and 15.2. The head across the approach embankment is plotted in Fig. 15.3. The distribution of discharge at bridge is given in Fig. 15.4 . It is concluded from Fig. 15 that:-

a) The left guide bank is captured at all lengths of the guide banks but the scour at the nose was maximum when the guide bank length is $^{M}{}_{L}/9$ or equal to the width of waterway as in this case the embayment behind the guide bank is maximum. The scour at the left guide bank of length = ML/3 was again significant when the flow was still left handed and approaching head on to the nose.

   The scour at the left guide bank was minimum when guide bank length was $^{M}{}_{L}/2$ as when the length was $^{M}{}_{L}/2$ the main current was brushing past the guide bank all along the length of guide bank though dispersing out on the way. The scour was again minimum when left guide bank length was $^{M}{}_{L}/4$ as when the length of guide banks increased from $^{M}{}_{L}/4.5$ to $^{M}{}_{L}/4$ the angle of flow approaching the left guide bank head had decreased suddently by 90^{o} and so the sweep of flow to the opposite bank was quite mild.

b) The embayment behind left guide bank was deflecting the currents on to the opposite bank upto the guide bank length of $^{M}{}_{L}/4.5$ but this deflection was quite mild when left guide bank length was $^{M}{}_{L}/4$. For guide bānk length $^{M}{}_{L}/2.0$ the main current was hugging all along the left guide bank whereas when left guide bank had a length of $^{M}{}_{L}/2.5$ the flow separated out from the left guide bank at its mid length.

c) The bed at the bridge line becomes almost even when the guide bank length becomes equal to $^{M}{}_{L}/2.5$ and $^{M}{}_{L}/2.0$. This is also confirmed up by the discharge distribution at the bridge.

d) The afflux increases with the length of guide bank (and this is utilized in generating higher velocities within the guide banks) upto $^{M}{}_{L}/3$ and then decreases and is minimum at $^{M}{}_{L}/2$.

3.8 Long guide banks but diverged at the nose

The straight guide bank, with lengths $^{M}{}_{L}/9$, $^{M}{}_{L}/4$ and $^{M}{}_{L}/2$ were diverged out at their noses in the form of a common compound curve of Radii R = 2.5L to form a 14^{O} segment of circle followed by a 23^{O} segment of Radius, R = 42 (Fig. 16.1). The bed contours after the usual run of one hydrograph are plotted and compared in Fig.16.2 and 16.3. The distribution of discharge at the bridge is given in Fig. 16.4, 16.5 and 16.6. The performance of the guide banks with diverged out noses improved a lot by diverging out their noses. The bed levels at the bridge line tend to become almost uniform for the three guide bank lengths by diverging out the noses of the guide banks. The evening out effect was more pronounced for guide banks of length equal to $^{M}{}_{L}/9$ = L. This is confirmed up by discharge distribution at the bridge line. The distribution of discharge is more symmertrical with diverged out guide banks with projected length (on the axes of bridge) equal to L. For the other two lengths of the guide banks the discharge distribution improves but the flow is not evenly distributed in the two compartments on the two sides of the bridge axis.

3.9 Optimum divergence angle for guide bank lengths = 1.0 L.

3.9.1 In an attempt to know the optimum angle of divergence of segment with Radius = 2.5L the segment of circle with radius L/2 was kept intact whereas the angle of major segment was varied from 14° in the earlier test under para 21.2 to 19° and 7°. The total length of the guide bank was kept equal to 'L' and the length was adjusted by varying the straight portion of guide bank along the abutment line Fig.17.1

The bed contours are compared in Fig. 17.2 whereas the distribution of discharge across the bridge is given in Fig. 17.3 to 17.6 . It is evident from Fig. 17 that the 14° divergence angle for guide bank length equal to the waterway length is the optimum as the flow distribution is more symmetrical and nearly even whereas in other cases the flow is unbalanced with higher concentration in the compartment along one abutment and lower concentration in the compartment along the other abutment.

3.9.2 Optimum divergence angle for guide bank length = 1.5 L

The length of guide bank was equal to 1.5 L and only straight length of guide bank was varied to accommodate divergence angle of major segment of compound curve at 7°, 18°, 25°, 30° - 31' (Fig. 18.1). The bed contours at the end of usual hydrograph for 4 divergence angles are given in Fig. 18.1. The distribution of discharge is plotted in Fig. 18.2.

It could be inferred from Fig. 18.1 and 18.2 that 18° divergence angle was nearly optimum for guide bank length of 1.5L as the uneveness of bed levels at bridge line was minimum though there was some deposition along the left guide bank and similarly the uneveness of distribution of discharge in the five compartments of the bridge was minimum though there was slightly more discharge in the side compartments than the middle two compartments but the difference between maximum discharge and minimum discharge in any compartment was only 13.5%.

3.9.3 The optimum divergence angle for guide bank length $\doteq$ 2L

The three divergence angles of the major component of compound curve of radius 2.5L tried were $12^o, 28^o$ and 36^o The bed configuration in the three tests of the series are given in Fig. 19.1 whereas the discharge distribution in five compartments is shown in Fig. 19.2 The test results show that 28^o divergence angle was better than the other angles tested.

3.10 The optimum length of guide bank is equal to the length of the waterway if it is diverged out to follow a compound curve of radii equal to 2.5L and .5L at a divergence angle of 14^o to 15^o and starting divergence of guide bank at a point .2L u/s of the abutment.

4.0 SCOUR AT BRIDGE WATERWAYS

4.1 Scour is defined as the displacement of bed and or bank material of fluvial channel by the erosive action of running water. It may occur naturally or be the result of channel constriction or changes in flow pattern causing local disturbances of the flow and sediment transport field. A local increase in mean velocity and/or turbulance intensity gives an increase in local transport velocity. From the equation of continuity.

$$\frac{\partial d}{\partial t} = \frac{\partial s}{\partial x} \qquad \begin{array}{l}(d = \text{depth} \quad)\\ (S = \text{Transport})\end{array}$$

The scouring continues until the local depth has increased so much that the velocities are reduced sufficiently to bring $\frac{\partial S}{\partial x}$ to zero and the result is dynamic equilibrium or in other words the scour hole enlarges to such an extent that the hydrodynamic forces have neither sufficient strength nor further contact with bed and sides of the scour hole.

Scour holes created by the forces exerted by flowing water over a bed of sediment around bridge piers are a major cause of uncovering of bridge pier foundations (well or friction piles) and subsequently there occurs reduction of friction between foundations and the surrounding soil and consequent settlement of foundations of bridge piers. These scour holes develope so long as the capacity of flow is sufficient to transport sediment out of scour hole at a rate greater than the rate at which sediment is supplied to the scour hole by the undisturbed flow approaching the piers. The greatest scour, except under unusual circumstances, occurs during the largest floods in areas containing alluvial material. The effects of local scour can be overcome by an increase in foundation depth or diminished by bottom protection. The scour that may occur at a bridge waterway can be categorised as follows:

a) The scour that would occur in the fluvial channel with or without a bridge crossing.

b) The scour that may occur generally at the bridge waterway because the flow is contracted by the bridge crossing.

c) The local scour that occurs because of the distortion of the flow pattern in the immediate vicinity of the bridge piers and abutments.

In the absence of evidence to the contrary, scour of these different categories must be treated as additive.

4.2 Degradation is usually a slow process and may be due to removal of sediment load by upstream damming, removal of control point downstream, development of cut-off across the neck of bend, channel improvement downstream or sympathetic fall in level at the meeting point of another river, migration of a meander and the result of augmentation of the stream flow. The stream bed may lower considerably due to degradation, therefore, the likehood of this type of action should be assessed in the hydraulic design of a new bridge or during in-depth periodic inspection of the existing bridges. If degradation occurs, it can change the depth of approach flow to a pier and the percentage of flow over the flood plain. Degradation is usually slow, because it involve a large area and tends to be continuous.

4.3 Scour due to Contractions

The embankment fills of the highway crossing may create severe contraction of the river in the flood. The over-bank flood plain flow then moves laterally to the bridge or returns to the channel above the abutment if it is provided with guide banks thereby augmenting the discharge of the channel very substantially in the floods, thereby increasing the capability of the flow to carry more sediment then the upstream river reach and thus there will be general scour over the entire waterway opening.

The following two methods can be adopted for assessing the magnitude of scour at the bridge waterway as a consequent of encroachment of flood plain and/or main channel by the approach embankment.

4.4 Laur-Sen Method

4.4.1 Analysis of the Long Contraction

The definition of the long contraction of a river in a flood is given in Fig. 21.1. The flow is assumed to be subcritical, the transition from wide to narrow section is ignored. The limit of scour in the contraction is the condition that results in continuity of flow and in a balance of sediment supply and sediment transport capacity. The Mannings formula was used to describe the flow. The sediment transport expression used by Laur-Sen is:

$$\bar{C} = (D/y)^{7/6} \left\{ \frac{v^2}{120 y^{1/3} D^{2/3}} - 1 \right\} K \left(\frac{\sqrt{g y s}}{w} \right)^a$$

in which

$\bar{C}$	=	Suspension plus bed load concentration by weight
D	=	Dia of the bed material.
y	=	Depth of flow in feet.
v	=	Velocity of flow in f.p.s= $\frac{1.486}{n} R^{2/3} S^{1/2}$
g	=	Acceleration due to gravity in feet per second.
s	=	Slope in feet per foot.
w	=	Fall velocity of the sediment in feet per second.

The exponent, a, and the coefficient k are dependent on the shear-fall velocity ratio. K drop,out of analysis and a varies approximately as follows:

$\sqrt{\frac{gys}{w}}$	a
< 1/2	1/4
1	1
> 2	9/4

The factor-1 in the term in brackets can be ignored in floods. By balancing S_1 total silt charge in the main channel to S_2 the stablized silt charge in contraction:

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} x \left(\frac{B_1}{B_2}\right)^{6(2+a)/7(3+a)} x \left(\frac{n_2}{n_1}\right)^{6(a)/7(3+a)}$$

The Mannings roughness ratio $\frac{n_2}{n_1}$ should be generally close to unity and so it can be neglected:

$$\frac{y_2}{y_1} = \left(\frac{Q_t}{Q_c}\right)^{6/7} x \left(\frac{B_1}{B_2}\right)^{6(2+a)/7(3+a)}$$

The first term $(Q_t/Q_c)^{6/7}$ expresses the effect of constriction of the overbank flood plain forcing the clear water over the flood plain into the channel.

The second term $\left(\frac{B_1}{B_2}\right)^{6(2+a)/7(3+a)}$ expresses the effect of the channel being restricted by the approach embankments. If both constructions are present the total effect is multiplicative. The two separate effects are shown in Fig. 21.2

4.4.2 Discharge Intensity Formula

Dr. Mushtaq Ahmad Formula is:

$$D_s = K_1 q^{2/3}$$

D_s = depth of scour from water surface.
q = average discharge intensity at the bridge and is equal to design flood discharge divided by clear waterway.

K_1 = a multiplying factor depending on the looseness factor and Characteristics of the approach channel and is given by Fig. 10.10

This formula was derived for bridges crossing alluvial rivers in deep sand fills in Pakistan and is based on the field experience and the model studies.The equation is derivable from Inglis Lacey relationship.

$$D_s = .946\ (Q/f)^{1/3}$$

in which Q is total river discharge and f is Lacey Silt factor = $1.76\sqrt{\text{Mean dia of sand in mm}}$.

4.5 Local Scour Hole Around a Bridge Pier

The local scour hole around a bridge pier or abutment is a direct consequence of the pier or abutment being an obstruction to the flood floe .The vortex system causing local scour was described in Chapter-1.0.

There are many parameters which may influence the scouring phenomenon around the pier.The variables characterising the fluid.

g = acceleration due to gravity.
ρ = density of the fluid
υ = kinematic viscosity of fluid.

Variables characterising the bed material.

ρ_s = density of the sediment
size distribution
grain form
cohesion of material.

Variables characterising the flow.

d_o = depth of approach flow

$\bar{u}$ = mean velocity of undisturbed flow.

η = the roughness of the approach flow.

Variables characterising the bridge pier.

its form.
its dimensions.
its surface condition.
any protection system.

The following restrictive conditions are called for:

a) bed material- the sediment is non-cohesive, and has a uniform size D.

b) flat bed, No. dunes, No. ripples and so η depends. On the diameter of sediment D.

Thus the maximum scour d_s is $=f(\rho, \upsilon, g, D, \Delta, do, U_r, b)$

where

and

Δ = relative submerged density of bed material.
U = shear velocity = $(gdos)^{1/2}$

The analysis of experimental data and dimensional analysis lead us to general relationship.

$$\frac{d_s}{b} = f\left(\frac{\bar{u}}{\bar{u}_c}, D/b, \frac{do}{b}\right)$$

NOTE: $\bar{u}_c$ is the velocity for initiation of movement of the undisturbed bed material.

Neglecting the influence of shape, Froude No. bed material density and gradation.

Thus the scour depth d_s will depend mainly on the ratio of mean velocity to mean Critical velocity and the relative values of grain size, flow depth and pile diameter.

(a) Influence of $\frac{\bar{u}}{\bar{u}_c}$ - various investigators in this field have distinguished the following intervals of $\frac{\bar{u}}{u_c}$

$\frac{\bar{u}}{u_c} \leqslant .5.$ No scour

$.5 \leqslant \frac{\bar{u}}{u_c} \leqslant 1.0$ Clear water scour. In this interval scour depth increases almost linearly with $\bar{u}$.

The limit scouring depth d_s, goes through a maximum d_{sm} for flow conditions corresponding to inicipient movement in the absence of any obstacle (i.e. $T = T_c$=Critical Tractive Force).

$$\frac{d_s}{d_{sm}} = 2\left(\frac{\bar{u}}{\bar{u}_c} - 1\right)$$

$\bar{u}/\bar{u}c \geqslant 1.0$ Scour with sediment in motion. Here scour does not increase further with velocity because the dynamic equilibrium between transport out of the scouring hole and the supply is not unfluenced by the magnitude of the transport rate. Scour depth fluctuates with movement of bed dunes. In a natural river the condition $\bar{u} / \bar{u}_c = 1.0$ is usually met with during floods almost with certainty and so even this parameter is ruled out for practical purposes.

(b) Influence of D/b:-

The perusal of available experimental data shows that:

(1) The influence of grain size is considered to be negligible for $D \leqslant .5$ mm.(shen et al. 1966a).

(2) Scour depth increases with grain size upto D = 2 mm for constant water depth do Fig.22. (NICOLLET 1971)

The coarser material roughens the bed thereby increasing the velocity gradient near the bed and the strength of vortex system.

(3) HANCO (1971) gave an increase of ds/b with $(D/b)^{.2}$ for D .5 to 5mm his relation can be converted into $d_{sm}/b = 3.3 \ (D/b)^{.2} \ (do/b)^{.13}$

(c) Influence of d_o/b

This factor has been studied extensively by almost all the researchers in the Field.

The following relationships can be distinguished:

(1) Relationship using Regime Depth as a parameter.

Blench: $ds/dr = 1.8\ (b/d_r)^{1/4} - 1$

where d_r = Regime depth.

Arunachalam: $ds/dr = 1.95\ (b/dr)^{1/6} - 1$

(2) Relationship using ds/b as function of do/b
Laursen and Toch (1956) and Neill (1965)
$ds/b = 1.5\ (do/b)^3$ for rectangular piers
Veiga da Cunha . 1970

$ds/b = 1.35\ (do/b)^3$ for circular piers.

Hanco 1971 $ds/b = 3.3\ (D/b)^{.2}\ (do/b)^{.13}$

(3) Relations giving scour as a function of b.

Larras: 1963

$ds = 1.05\ b^{.75}$ (m. units).

Shen : 1969

$ds = b^{.619}$ (for constant $\bar{u}$)

Breusers 1965

$ds = 1.4\ b$ (circular pier)

Barak 1975

$ds = .588\ b^{.586}$ (rectangular pier).

The experimental evidence is compared in Fig.23.1 for circular piers only. The results by Laursan were scaled with effective width (.6' instead of .2') as pier under an attack angle of 30° was used and corrected for pier shape.

Basak results were also reduced by a factor of 1.2 because he used rectangular piers.

It is evident from Fig.23 that for $d_o/b > 3$ the influence of this parameter becomes equal to that of $d_o/b = 3$.

The following relation gives upper enveloping curve for the data in Fig. 23.

$ds/b = 1.43\ (do/b)^{.3}$ for circular piers and is recommended to be used as it includes the effect of angle of attack.

The average curve of the data is given by: $ds/b = 1.5 \tanh do/b$.

4.6 Effect of Shape of Pier

Taking together all the available evidence on the subject it can be safely concluded that if circular or the round nose is taken as a reference, a reduction in the order of 25 per cent in scour depth can be obtained by stream lining (Fig. 23.2 and 23.3) the pier. This positive effect disappears for an angle of attack larger than 10-15°. On the other hand a rectangular pier gives 20-40% more scour than the reference circular pier.

A pier consisting of two or more circular piers seems to be an attractive one in case of an appreciable angle of attack as angle does not influence the scour. (Chabert & Engel Dinger) for centre line spacings larger than 3 b.

4.7 Influence of Angle of attack

The influence of an angle of attack has been studied by Laursen and Toch 1956. Fig. 23.4 which gives a good estimate of coefficient K_d. against an angle of attack. Some authors propose the use of the projected width in their relationships (BARAK 1974) but this gives an over-estimate in most cases.

In the field it is difficult to measure the angle of attack which usually varies with river stage and