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HISTORICAL REVIEW OF DEVELOPMENT IN DESIGN, OPERATION & MAINTENANCE OF BARRAGES IN RELATION TO SURFACE FLOW

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By

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1.1.0 INTRODUCTION

The design problems of a hydraulic structure such as a barrage involve the following two broad specializations :

(i) the hydraulic and
(ii) the structural

In this paper it is proposed to deal only with the hydraulic aspects of the design of such structures. The scope is further confined to structures such as barrages built in rivers having permeable and erodible foundation. The design problems resulting from the seepage flow underneath the structure have been dealt with exhaustively in a book written by the author. In this paper it is proposed to deal with problems arising from the flow over a barrage. The subject of flow of water over a barrage built on an erodible soil present a host of problems. Some relate to the safety and stability of the structure itself and the others to adequate fulfillment of the functions for which the structure has been built. Each hydraulic structure, to be economically feasible, has to provide an efficient and trouble free service for a reasonably long period. The failure or damage to the structure due to hydraulic causes or an inefficient hydraulic performance of any of the appurtenances can be due to incorrect sitting of the work, incorrect fixing of levels and dimensions of the structure, which may be due to the lack of adequate hydrological data resulting in a misjudgment in estimation of discharge and levels. Inadequate provision in design for long term changes in the river behaviour in respect of accretion or retrogression of river bed and possible changes in the river course and subsequent neglect in the construction of adequate river training works at the most appropriate position and time, as it can also be responsible for the malfunctioning of the structure. Some of the omissions at times may result in heavy damage or even failure of the structure.

The diversion dams or barrages, even bridges, canal regulators and canal falls, built on permeable erodible foundation are a class of hydraulic structures in which the Civil Engineers associated with the Irrigation and Highway Bridge Projects are generally interested. Indus, Ganges and their tributaries in Indo-Pakistan Sub-continent and many other rivers of the class all over the world run through the alluvial plains made up of geological deposits consisting of recent stream deposits or

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glacial fluvial material associated with melting of glaciers. Such alluvial deposits in the Indus valley are as deep as 100 to 1000 feet and structures built on these rivers or channels in riverine area have been experiencing most serious localised scour problems. Scours as deep as 100 to 150 feet have been noticed at the pier foundations of Harding Bridge at Ganges. Deep localised scours at weirs, barrages and canal falls is a very common experience.

The irrigation system consisting of some 20 barrages in alluvial meandering rivers of Indus and its tributaries with nearly 3,500 miles of man-made canals, and thousands of canal falls to control the channel grade and hundreds of regulators to distribute the supply into the distributaries and minor channels, have mainly provided the data and information on the design and performance of the structures to be presented in this paper. The problems and the evolutions faced in the design, construction, operation and maintenance practices will be discussed with special reference to weirs and barrages.

After briefly discussing the main functions of the weirs and barrages and their principal effects on the flow in a channel, the historical development of the design features will be presented briefly, highlighting the successes and failures of some of important structures in the Indo-Pakistan Sub-continent. The lessons learnt from each, and consequent development of the science and engineering practice for the design and maintenance will be presented.

1.2.0 MAIN FUNCTIONS OF THE HYDRAULIC STRUCTURES

Hydraulic structures constructed on a permeable erodible foundation across a natural river or a man made canal are usually intended to fulfill one or more of the following functions. The structures are named and classified according to the principal function to be fulfilled by each.

(1) **To impound or Store Water**

Such structures are classified as large dams. These are preferably built in narrow hilly gorges to store as large volumes of water as possible during floods thereby serving as flood control measure. In addition, the stored water is used for power generation and irrigation during low river flow periods. For the exigency of passing a flood peak when the reservoir is full, one or more spillways to pass the flood are invariably provided. This is however not within the scope of this paper.

(2) The structures for this purpose when built across rivers, are named Diversion Weirs or Barrages, Water storage not being the main function in diversion works, real objective is to raise the Water level above the structure to a level just high enough to pass designed discharge of the main stream flow into the offtaking channels. In the design of the diversion weirs or barrages excessive efflux during floods

is not allowed. Generally in flat plains the maximum water level is not raised more than 2 to 6 feet above the highest flood level. The only barrage which provides for some storage of water in the ponded reservoir is Chashma Barrage.

(3) **To Control and Maintain Regime Slope in a Canal**

An artificial canal transporting sediment, when built in a region where the country slope exceeds the required regime slope, control structures such as Canal Falls have to be built.

(4) **To Measure Water**

Meter flumes and outlet flow metering structures are generally built to measure the rate of flow in a canal, a distributary or a minor. In fact all hydraulic structures such as Spillways, Weirs, Canal Falls, Regulators and Outlets can also be used to measure the flow after some sort of calibration.

(5) **To Provide Crossing Over a Natural or Man-Made Stream**

Highway or Railway Bridges are the well known hydraulic structures that fulfill the above objective.

(6) **To Cross a Stream by Another Stream**

Structures which allow one stream to pass underneath or over another are called Inverted Syphons or Aqueducts respectively. The crossing may even be at level with the other stream. Such a crossing is generally known as Level Crossing.

1.3.0 THE EFFECTS OF HYDRAULIC STRUCTURES ON FLOW

All the above hydraulic structures cause some sort of obstruction to flow. The obstruction may be caused either by reducing the flow section by creating a barrier in the bed or from the sides by introducing lateral contraction. A hydraulic structure may simultaneously impose both lateral as well as vertical obstruction to flow as in the case of spillways, flumed canal falls or weirs where the length of the structure is small compared to river Khadir width. A Highway or Railway Bridge in an alluvial meandering river also provide an example of lateral contraction of flow without imposing vertical constriction. A barrage, a meter flume or a canal fall generally have both vertical and lateral constriction. A syphon spillway, a crossing in the form of inverted syphon or an aqueduct are examples of an extreme case of lateral contraction. It is not possible to historically review the developments in design, operation and maintenance of all the above types of Hydraulic structures in one paper. For each type of structure, the problems are apparently similar in nature but differ on effective parameters involved in each case.

1.3.1 HYDRAULIC PHENOMENON AND PROBLEMS

The vertical or lateral restriction of flow section in different hydraulic structures classified above, generally cause some change in flow regime, resulting in problems which have to be taken care of in the design. One problem relates to the efflux caused upstream of the structure with consequent drop, resulting in the creation of high velocity flow having excessive kinetic energy to be dissipated on the downstream side. If not properly taken care of in designing, it can cause heavy scour downstream. Over a long period a general degradation followed by agradation of the stream bed is also experienced. The changing course of river upstream and downstream of a bridge or a barrage can also create problems which can only be solved by providing training works. In case of barrages and canal regulators equitable distribution of water sediment complex for the off taking channel is another problem which cannot be neglected in the interest of efficient functioning of the structure. Moreover, it has to be kept in mind that the structure does not destroy the aquatic life of the stream or hinder the water born communications. A neglect in foreseeing any of the above problem in the design may result in malfunctioning of the structure. The effects of hydraulic structure constructed in an alluvial channel on the flow and channel regime with accompanied problems and dangers to the safety of the structure are briefly listed below.

1.3.2 EFFLUX

A structure constructed across a stream causing any flow restriction invariably creates an efflux or an increase in depth of flow upstream of the structure compared to the normal depth of the stream. A very large drop in water level across the structure is created by dams where the main objective is to store large quantities of water. This efflux can be made very high if the dam and the spillways are founded on rock. Dams to a height of more than 900 feet above the river bed level have been built. On permeable foundations, there are serious limitations as regards the maximum permissible drop.

1.3.3 ENERGY DISSIPATION AND SCOUR

In Spillways, weirs, barrages and canal falls, as the water flows over the structure, the potential energy is changed into kinetic energy in the high velocity jet at the toe of the structure. This energy, if not properly dissipated within the confines of the structures, can cause dangerous scour when the bed material of the stream on which the structure is built is erodible. For a low dam or a weir in case of rapid draw-down or existence of oblique flow causing swirls and eddies, serious upstream scour can also result. Greater the discharge intensity and the drop in level at the structure, greater will be the energy to be dissipated. These conditions impose serious limitation on the permissible drop in water levels for hydraulic structure built on erodible river beds.

1.3.4 AGRADATION AND DEGRADATION

For a raised obstruction such as a spillway, or a weir constructed in the bed of an alluvial stream carrying appreciable bed load, the channel up-stream of the structure is likely to be silted up with time causing general accretion of the bed upstream and corresponding retrogression or degradation of bed downstream. Proper care in designing such structures for passing down the sediment with least obstruction specially in the period when the river carries heavy sediment load, is essential in the interest of satisfactory function and long life.

1.3.5 CHANGE IN RIVER COURSE AND NECESSITY OF TRAINING WORKS

In a wide meandering alluvial river a lateral contraction is invariably provided in the interest of economy by reducing the length of the bridge or the barrage in relation to the 'Khadir' width. Bridges or barrages 3000 to 4000 feet in length have been constructed in river Khadirs as wide as 5 to 10 miles. In such cases a system of guide banks with training works to guide and pass the river flow through the bridge or the barrage as uniformly as possible is generally an un-avoidable necessity.

1.3.6 EQUITABLE DISTRIBUTION OF SEDIMENT WATER COMPLEX AT HEAD-WORKS AND REGULATORS.

In case of barrages constructed in meandering alluvial rivers, it is essential to correct the river approach to the barrage in the interest of correct proportional distribution of sediment water complex into offtaking channels. Any sediment in excess of the sediment carrying capacity of the channel can cause the head reach to silt up in a few years to the extent that the canal will refuse to take its authorised designed discharge. This problem being of very great importance, river training works upstream of barrages have some times to be built in meandering rivers to achieve the objective of creating and maintaining a favourable river approach for correct sediment entry into the offtaking channels. Other measures such as silt excluders and ejectors have also been devised.

1.3.7 PROTECTION OF AQUATIC LIFE OF THE STREAM

Hydraulic structures across a river or a natural stream must have a fish ladder as a bye-pass to allow the fish an easy passage to the upstream pond specially when during certain parts of the year the entire incoming flow in a river is likely to be diverted into the off taking canals. The fish in the depleted river downstream of the barrage is likely to die due to drying of the river unless it can travel up the fish ladder into the pond upstream of the barrage.

1.3.8 MAINTENANCE OF WATER BORN COMMUNICATIONS

If a barrage is built across a river where water born communications already exist and the facility has to be maintained in the post construction period also, a lock has to be constructed to allow the barges and boats to negotiate the difference in level upstream and downstream of a barrage.

1.4.0 EVOLUTION OF BARRAGE DESIGN IN INTO-PAKISTAN SUB-CONTINENT

The behaviour of hydraulic structures built on erodible permeable beds of meandering rivers and the associated problem as summarised above were not all clearly understood by the early designers. The early engineers in Indo-Pakistan Sub-continent designed the weirs more by intuition and engineering skill than by any theory or established practices. In fact the successes and failures of the early works in Indo-Pakistan Sub-continent make the history of the evolution of design of such structures ultimately leading us to present day state of knowledge on the design and operation of a modern barrage. The historical evolution of design of barrages in Indo-Pakistan is briefly discussed to highlight the development in type designs. The problems encountered, the solutions and the remedial measures evolved and found successful as a result of long experience will be described.

1.4.2 SOME WEIRS BUILT PRIOR TO 1870
Grand Anicut at Tanjore

In 1830, when the British Engineers were investigating the problems of irrigation in the region they discovered the Grand Anicut at Tanjore which was buried due to silt deposition on the upstream side to its top level and over which water was flowing unimpeded into the Colerun. The huge stones were still intact and sound. The British Engineers obtaining additional granite from a nearby source, raised the height of the weir once again to divert the low flood water into Tanjore Canal. Many more modifications have been made ever since and the work still stands on a foundation which is perhaps a thousand year old.

1.4.3 UPPER ANICUT ON COLERUN

Encouraged by the experience of Grand anicut at Tanjore, Captain Arthur Cotton in 1830, set himself on a most difficult and daring task without previous precedence of founding a structure not on rock but on pure sand, by building an Upper Anicut across Colerun by the same technique. In 1836, a weir consisting or rectangular bar of masonry across the river was constructed with double line of wells on the upstream side sunk deep into the sandy river bed and with a single line of similar wells on the downstream side. the cross section of the weir is shown in Fig. 1.1

1.4.4 JOBRA WEIR ON MAHANADI RIVER

Soon after demonstrating that the feat of Chola Kings could be reproduced by engineering skill, weirs on Mahanadi, Kistna and Godavari were constructed prior to 1855. Jobra weir was constructed across Mahanadi river having maximum discharge of 900,000 cfs. The river bed at the weir site consisted of deep sand; in some cases stiff clay and in others rock underlying the sand was rising to the surface. The main object of the weir was to raise the water level in dry season to feed the canal. Three feet high folding shutters were provided on the crest. The plan and cross section of the Jobra weir on Mahanadi river are shown in Fig.1.2 (a,b). It had two undersluices, one in the centre having 10 bays each of 45 feet and the other at the southern end consisting of 38 vents, 5 feet each. The weir glacis had a slope of 1 in 12. The central undersluices failed in 1886 probably due to undermining and sour.

1.4.5 BESWADA ANICUT ON KISTNA RIVER

The cross sections of Beswada Anicut on Kistna river, built in 1854-55, shown in Fig.1.3 consisted of wells on top of which 3 feet thick ashlar masonry was laid above which 13.5 feet high wall of rubble masonry was built. This formed the crest block. On the downstream side massive stones of all sizes upto 5 to 6 tons were deposited to give it flat downward slope of 1:12. Stone was added year after year.

1.5.0 TYPICAL WEIRS BUILT BETWEEN 1870 AND 1900 FT.

In Northern India Western Jamna Canal was the earliest perennial canal built in the 14th century. It was restored after disuse for a period during Moghal Empire in the 15th century. It was again restored by the British in 1820. Throughout these centuries, the construction and reconstruction consisted of a simple dam of shingle and brushwood to head up the flow and divert the same into the canal. The temporary dam across the river wad washed away each year during flood and was rebuilt during October, for getting water for winter crops. The supplies were erratic and uncertain. Consequently in 1870 after about 50 years of experience, with the temporary diversion, it was decided to construct permanent masonry work across Jamna river. To overcome the peculiar difficulty of having to keep the perennial flow of the river always in contact with the two canal heads situated on the opposite banks of the river, without any past precedence or experience to guide, the designer Col. Crofton built the weir slanting to the flow and located the head of the Eastern Jamna Canal higher up and that of Western Jamna lower down (Fig.1.4a) so that perennial flow may first pass by the head of the Eastern Canal. At each head of the canal one set of undersluices were built. The idea, though well conceived, had defects and drawbacks. In actual practice it became necessary to construct series of river training works at considerable cost to keep the perennial stream to the planned course. Cross-sections of the weir are shown in Fig.1.4(b).

1.5.1 NARORA WEIR AT GANGES

The narora weir was built in 1878 across Ganges river having fine micaceous sand bed. The Lower Ganges Canal takes off from this weir. The weir is constructed in about one half of the waterway of the river and the efflux caused by it is about 2 feet.

The weir consists of a wall 8 feet thick at the base and 10 feet high, founded on aline of blocks or wells 10 feet square, sunk 7 to 30 feet below the river bed. Downstream of the wall is a horizontal floor 40 feet wide and 5 feet thick as shown in Fig.1.5(a).

The undersluices consist of 42 openings 7 feet 3 inches each. The undersluice floor level is 572, which is the same as weir floor, the level of the weir crest is 582 and the top of the weir crest shutters is 585. The highest flood level was 590. The floor level of head regulator is 575, i.e. 3 feet higher than that of undersluice. The higher level of the canal head regulator floor compared to the undersluice level was purposely designed to get an advantage of minimising silt entry into the canal.

To protect the weir in high floods marginal bunds and training works (spurs) had to be constructed on the left bank upstream of the weir to protect the low land from inundation. The river could have otherwise cut into low lands on the left bank with a possibility of out flanking of the weir. It wad considered essential to force the main stream away from the left bank and compel it to flow in a direct course from the Railway bridge to the weir. (see Fig.1.5-b).

It has to be noted that only short flank walls exist at the two ends. the concept of the guide banks had as yet not been evolved. The undersluices were, in the original design, not separated form the main weir by any divide wall.

The study of the performance of Narora weir and its failure stand out prominently in providing some object lessons. In 1895, the engineer incharge attempted to clear a large sand island formed upstream of the weir by manipulating the gates during the floods. Groynes at C an F projecting from the weir were also built to avoid parallel flow (see Fig.1.5-C). In 1895 scour of about 35 feet was developed only 40 feet from the weir crest and in 1896 similar scour was noted 150 feet from the crest. During the floods of 1895-96 and 1897 the shutter were kept erect in a length of about 800 feet on the left flank with view to scouring away the island on right centre but in place of that island a new one was formed on the left flank in front of the obstruction. Thus before the failure in 1898 the upstream clay puddle apron had disappeared. On the downstream side also the bed had scoured by 8 feet below the floor. The weir collapsed on March 29,1898 under a head of 12 feet, with no flow over the weir. The crest wall of the weir subsided in a length of nearly 400 feet. The ashlar of the floor was lifted up to a height of 2.23 feet, tearing with it one course of brickwork beneath it. The failure of Narora weir has been discussed by the author (see Ref.1) in relation to the inadequacy of proper hydraulic gradient after the upstream and downstream scours had developed. The actual failure no doubt occurred when there was no over flow, but the mischief had been done earlier by the creation of very deep scour upstream during preceding flood when the gates on the left half of the weir were closed and all the flow had to turn round the Groyne C, causing flow concentration at X in (Fig.1.5-C).

1.5.2 OKHLA WEIR AT JAMNA

The Okhla weir consists of first two walls 30 feet apart from centre to centre and 4 feet thick; the upper one is 10 feet and the lower one is 9 feet high, the interval between them is filled with rubble stone and carefully packed with very large rubble on the top. There is third wall 4 feet 9 inches high 40 feet below the second one and the interval is packed in the same way. The cross section of Okhla weir is shown in Fig.1.6.

The weir was damaged by the first flood which passed over it after its completion in 1871. The velocity over the talus where the damage was done was estimated to be over 18 feet a second.

The undersluices of this weir consisted of only 16 vents of 6 feet each in width and 10 feet high. The crest level of the weir is 659.27, the floor level of the undersluices being 648.27, and that of the head regulator of the canal 652.27, or 4 feet above the undersluice floor. The fixation of the relative levels of weir, the undersluices and the canal regulator crest shows that the designers had full cognisance of regulation in the pocket for minimising sediment entry into the canal.

1.5.3 RUPAR HEAD WORKS AT SUTLEJ

The Sirhind Canal was supplied water from the Rupar Headworks built in 1882 on Sutlej river at Rupar. The bed of the river had a slope of about 2 feet a mile and was composed of sand with underlying shingle and boulders of moderate size.

The undersluices are at right angles to the stream but the Weir inclines upstream from the sluices at an angle of 15 degrees Fig.1.7.(a.b). The weir proper is 2400 feet long. The total length is 2663 feet between abutments. Its crest was originally built at EL.865 or 8 feet above the general level of river bed, but it has since been raised by 1.5 feet. Falling shutters have been provided. The weir consists of two walls, the upper one founded 7 feet and lower one 3 feet below the ordinary river bed. The weir section, showing the position of these walls is given in Fig.1.7-C.

The upstream slope of the weir is 1 in 3 and downstream slope is 1 in 15. The pitching below the floor is very heavy, each stone weighing as much as 30 cwt. At the toe of the stone pitching below the undersluice floor, masonry blocks 12 feet are built as shown in Fig.1.7(c).

The weir is designed for maximum flood discharge 315000 cusecs and corresponding upstream and downstream water level are 877.0 and 872.5 respectively, the efflux being of the order of 4.5 feet.

The Sirhind Canal was designed for 6,000 cfs with a slope of 1/5,000. As a general rule, it may be said that the river is completely directed into the canal for about two months (usually January and February) each year.

From 1893 to 1900 large quantities of sandy silt were deposited in the head reaches of the canal, especially in the first twelve miles. In 1894 as much as 19.6 million cubic feet in five Summer months of the year passed into the canal. A deposit of 4 million cubic feet in 10 days of flood were not unusual. The silt deposited in the canal bed in the first 20 miles from the year 1892 to 1900 as recorded by Kennedy are given in Table-1.1.

TABLE-1.1

Year	Maximum (about August)	Minimum (about Spring)	Difference deposited each Year
1892	284	180	104
1893	202	107	95
1894	187	100	87
1895	218	68	150
1896	211	79	132
1897	249	73	176
1898	192	67	125
1899	262	66	196
1900	208	70	138
1901	82	24	58
1902	56	24	32
1903	67	16	51
1904	30	14	16

A study of the prevailing conditions of regulation revealed that the trouble was caused by the coarser sand, and the sill of the regulator was consequently raised.

The remedies evolved for sediment exclusion and carried out from 1893 to 1904 are recorded below :

(I) The capacity of the escape was increased from 2000 to about 4000 cusecs.

(II) The canal was closed down during heavy floods. This was the beginning of the idea of still pond system of regulation which was adopted in 1901.

(III) The divide wall AB (see Fig 1.7-a) was constructed so as to form a pond or silt trap, which was occasionally scoured out through the undersluices.

(IV) the regulator sill was permanently raised by 7 feet above the floor and was provided with a movable sill which could be raised still higher when the level of the water in the river permitted.

It would be clear that as far as 1893 the engineers took steps to minimise silt entry into the canal which are even now considered important for sediment exclusion. Eminent engineers of the time like Kennedy(2) and others, were beginning to visualise such measures as bifurcation of flow higher up, canal closures during floods, creation of a pond in a pocket with a divide wall, initiating sluicing operation after silt deposition in the pocket and raising the regulator crest level.

1.5.4 MADHOPUR HEAD WORKS AT RAVI

The weir across the Ravi river was built in 1871 at the head of the Bari Doab Canal in the Punjab (Fig 1.8-a). The weir section consists of Upper and Lower walls 40 feet apart built in boulder masonry as shown in Fig. 1.8(b). Downstream slope of the weir is 1 : 10. the slope of the river bed above the weir is over 20 feet per mile, heavy boulders of size of man's head are carried down it. It was damaged severely in the first flood. Protective training works on the upstream side of the weir were constructed as shown in Fig. 1.8(a) to deflect the current away which caused concentration of flow on the left. Floods of 1875 undermined and completely swept away the sluices on the left side outflanking the weir. the damaged sluices were rebuilt but the weir was again damaged in floods of 1880. In 1885 the undersluices were again damaged. The undersluices as reconstructed in 1885 are shown in Fig. 1.8 (b).

1.5.5 KHANKI HEAD WORKS AT CHENAB

In 1887 the Lower Chenab Canal taking off from the river Chenab was constructed as a perennial canal without a regular head works. Soon after opening in May, 1887, the canal got choked with silt in a few weeks. After silt clearance, the canal silted up again within a fortnight. The construction of the Khanki headworks was undertaken in 1890-92. The weir across the river Chenab consisted of 8 spans of about 500 feet divided by 10 feet wide piers, a set of 12 undersluice spans of 20 feet on the extreme left and the canal head regulator with 12 spans of 24.5 feet each as shown in Fig. 1.9(a). The weir has been constructed in rubble masonry with upstream curtain wall 8' deep below the crest in brick masonry. Crest level of the weir is 723 and has hinged shutters to raise the water level. Downstream glacis slope is 1 : 15 and downstream floor is 45 feet long in the direction of flow and was laid at R.L. 715. It is note-worthy that Khanki weir was the first to have the guide banks. Bell who carried out extensive training of rivers for Railway bridges first suggested the idea of constructing guide banks to constrict the river to obtain uniform flow through the bridge or a barrage.

He recommended converging guide banks upstream to provide bottle neck layout. However, the guide bank could be straight or diverging upstream as well. The diverging type of Bells Bunds were constructed in 1892 for the first time for Khanki weir.

1.5.6 RASUL HEAD WORKS AT JHELUM

The head works for Lower Jhelum Canal was constructed in 1901 across the Jhelum river at Rasul as shown in Fig. 1.10(a). River slope was about 1 in 3400 and the weir was designed to be almost at the mean bed level of river to act as a bar. It has been constructed in rubble masonry resting on wells of 6 to 8 feet diameter filled with sand as shown in Fig. 1.10(b). Crest of the weir is at R.L. 707.2. Slope of the downstream glacis is 1 : 15, upstream floor is in level with crest, while downstream floor level is at EL 701.2. Length of the weir between abutments is 4400 feet. It was designed for maximum flood discharge equal to 875000 cusecs and corresponding upstream and downstream water levels are 724.2 and 723.5, efflux being 0.7 feet. The shutters over the weir crest were burried under silt and gravel deposited during the falling stages of flashy floods to a depth of over 2 feet. The shutters could not be worked unless the silt and gravel was removed. This demonstrated the defect and limitations of a low crested weir with rising shutters.

1.6.0 REVIEW OF STATE OF KNOWLEDGE ON WEIR DESIGN UPTO THE BEGINNING OF 1900 AD

The early structures constructed across rivers have been classified by Bligh according to 3 types :

Type A

Weirs provided with a vertical drop wall to raise the water level with a horizontal floor (water tight) on the downstream side.

Type B

In this type vertical wall at crest is provided. On the downstream side long sloping floor of stone with smooth water tight masonry of definite thickness and with curtain walls of masonry at suitable distance apart are provided to retain the rubble in position. The rock fill is made water tight either by stone masonry laid over it or by grouting.

Type C

The cross section is similar to type B, consisting of rock fill weir having a flat slope consisting of stone pitching going upto downstream river bed level. The rubble is retained in position by curtain walls of masonry forming square or oblong cells. The weir is not water tight.

The typical weir of type A is Narora at river Ganges, and Colerun at river Colerun, while Khanki weir on river Chenab and Rasul weir on river Jhelum are typical examples of type B, Okhla, Jobra and Beswada weirs on Jamna, Mahanadi and Kistna are respectively the examples of Type C.

A comparative study of different design elements and hydraulic data for some typical weirs prior to 1900 such as Narora, Okhla, Madhopur, Rupar, Khanki and Rasul are given in Table 1.II.

The following are the typical features of these headworks :

(1) The weirs generally consisting of a raised crest block, with sloping back fill of stone on the upstream and downstream side were constructed across the river to head up water to a height sufficient to supply the canal. The crest was usually fitted with some form of shutters which could be raised to head up supplies or opened to permit the passage of floods.

(2) A regulator at the head of the canal situated in the river bank just upstream of one end of the weir and consisting of a series of openings through which water could be admitted into the canal.

(3) The undersluices which were really the part of the weir immediately adjoining the canal regulator, consisted of a series of openings controlled by sluice gates. The sill level of the undersluices was kept lower than the weir crest and much more lower than the sill of the regulator. The undersluices helped the river to scour a deep channel along that bank from which the canal took off. The undersluices were used for silt exclusion from the canal by resorting to occasional flushing of the undersluices.

(4) The more common type of weir consisted of raised crest block above river bed level with stone filled between cross walls at suitable intervals apart. The downstream slope varied between 1 : 10 to 1 : 15. the slopes upstream of the crest block were usually kept 1 : 1 to 1 : 4. Later, upstream floor had to be added to keep the upstream scour away from the crest block. In some weirs the top surface of downstream slope was made of stone on end laid in mortar. Still later, for Khanki and Rasul weirs rubble masonry was used to make the top surface of downstream slope impervious and strong enough to withstand high velocity.

(5) To overcome the dangers to the weir due to development of cross flow during shutter regulation, a practice of constructing longer groynes to separate each weir bay was evolved. Such groynes were constructed with some success on Rupar, Khanki and Rasul weirs. However, new problems were encountered which necessitated careful regulation.

(6) The necessity of constructing training works such as series of spurs to ensure suitable approach to the weir was realised quite early for Narora weir when the river had to be trained to avoid very oblique approach to the weir. Later with the development of the concept of Bells Bunds attempts were made to give a general direction to the flow as to pass normally and uniformally through the weir. Bells Guide Bunds were first constructed at Khanki and Rasul. A series of spurs were also considered necessary inspite of the Bells Bunds and had been constructed upstream of Rupar and Khanki to avoid formation of heavy embayment at guide bank nose.

(7) The concept of energy dissipation by proper formation of hydraulic jump to ensure adequate downstream flow depth and floor length was not realised clearly upto this period. A long downstream glacis slope was laid with a horizontal stone apron in rubble masonry blocks or dry stone crates of various arbitrary lengths at the downstream end of the slope. The apron was generally laid at low water level or river bed level. Such weirs during high floods invariably worked as highly drowned structures with very little dissipation of energy.

(8) Nearly all these weirs had very wide waterway with raised crest of weir to provide an efflux of not more than 2 to 4 feet. The looseness factor for these weirs for maximum discharge varied between 1.7 to 3.0. The high crest with shutters on it induced general silting and shoaling upstream almost to the pond level with downstream retrogression. The control at the weir was generally lost over a long period and subsequent raising of the weir crest had to be done periodically.

(9) In the beginning, the total length of the hydraulic structures, from consideration of sub-soil was fixed arbitrarily. The hydraulic gradient given by the ratio of the total head across to the length of the weir was denoted by a coefficient which was supposed to depend upon the type of the soil on which the weir is built. The historical development of the hydraulic structures design with respect to the under seepage flow has been discussed in detail in Ref. 1.

(10) For designing a structure from considerations of surface flow, Bligh[3] thought the optimum length of the floor measured from the toe of the crest block to the end of the loose protection can be related by the formula

$$L = 10C \quad \frac{H}{10} \quad \frac{q}{75}$$

where H, the drop is the difference of level between the top of the masonry crest of the weir or the shutter and the low water level downstream of weir or the normal bed level of the river whichever happens to be higher Fig. 1.11(a). In this relation q is discharge per foot run and C is the coefficient characteristics of the material of the channel. The value of C as generally found for some of the safe works, was assumed between 9 to 15. A value of C in this range gives nearly the length of downstream structure equal to the existing lengths of some of the weirs considered safe (see Table 1.III).

TABLE 1.III

River	Weir	Type	C	H	q	L Calculated	L Actual
Ganges	Narora	A	15	10	75	150	140
Colerun	Colerun	A	12	4.5	100	106	72
Chenab	Khanki	B	15	6	150	185	170
Jhelum	Rasul	B	15	8.5	135	160	135
Kistna	Beswada	C	12	13	223	236	220
Mehnadi	Jobra	C	12	10	140	163	143
Jamna	Okhla	C	15	10	140	210	210

For the length of the apron to be made solid and impermeable to withstand high velocity over the structure downstream of the crest block was recommended by Bligh to be given by the relation.

$$L_1 = 4C \quad \frac{H^1}{13} \qquad 1.2$$

where H_1 is the head measured from the top of the gate to the solid floor level at the downstream end (see Fig. 1. 11 (b).

It will be noted that the discharge per foot run which controls the velocity of flow is present in the formula 1.1 but it is missing in 1.2. The inclusion of only H or H_1, the head across is justified if we are considering the uplift pressure or the exit gradient but for consideration of surface flow conditions the discharge per foot run q cannot be neglected and the relevant drop is the HFL minus the Tailwater Elevation. These formulae are therefore at best only empirical and as such do not take into consideration all the essential hydraulic parameters which determine the optimum dimensions of floor length or flow depth over the downstream apron. It would suffice to note that at the time when these formulas were put forth, the concept of energy dissipation through hydraulic jump was not fully understood.

(ii) The formation of scour on the upstream side of a structure very often occurred due to creation of parallel flow and formation of vortices and swirls caused by unequal or irregular gate operation. On the downstream side the formation of high velocity jet and its continuation along the downstream slope of the weir on to the erodible bed resulted in heavy scour. The devices such as sills, baffle blocks etc for the correction of the vertical velocity distribution downstream within the confines of masonry structure and for creating a ground roller at the bed downstream of the solid apron were still unknown. One redeeming feature of the design however was that the downstream apron levels were kept at low winter water level which in most cases during high floods provided nearly enough downstream depth for the formation of a jump having very high drowning ratio when little energy dissipation occurred.

1.7.0 WEIRS AND BARRAGES CONSTRUCTED DURING 1900-1950

During this period remodelling and renovation of some of the older weirs had to be carried out. The new barrages constructed during the period 1900 to 1950 in Northern India (now Pakistan) relate to the following projects :

(1) Upper Chenab Canal Project-Marala Weir.
(2) Lower Bari Doab Canal Project-Balloki Headworks.
(3) Sutlej Valley Canal Project consisting of the following barrages at :

    (a) Ferozepur
    (b) Sulemanki
    (c) Islam
    (d) Panjnad

(4) Sukhur Barrage Project
(5) Haveli Canal Project-Trimmu Barrage
(6) Thal Canal Project-Kalabagh Barrage

In the period prior upto the first World War in 1914 Upper Chenab Canal taking off from Marala Weir was opened in 1912. The tail of U.C.C joined river Ravi just upstream of Balloki Barrage. At Balloki, the Lower Bari Doab Canal took off on the left flank and U.C.C. joined the river Ravi at the right flank. thus balloki Barrage was designed originally as a level crossing to divert water of River Ravi into Lower Bari Doab Canal as well as transfer surplus Chenab water into Ravi to feed the Lower Bari Doab Canal. The Marala weir in plan and section is shown in (Figs. 1.12-a and 1.12-b) and Balloki in Figs. (1.13-a and 1.13-b).

In the period 1922-1929 the work on Sutlej Valley Project comprising headworks at Ferozepur, Sulemanki, Islam and Panjnad was undertaken. In these weirs some radical changes form the old conventional designs of weirs at Marala, Khanki and Rasul were made. Unlike older headworks, canals form these weirs took off from each bank, making the river and canal regulation fairly complicated. The centre of the weir had to be used for escaping surplus supplies. The operation of primitive shutters was considered extremely difficult more especially at night and consequently these weirs were designed as barrages and were considered an innovation. The substitution of shutters with gates of greater height also made the lowering of weir crest levels possible resulting in consequent reduction in the length of the weirs. The barrages as such were found to be more economical than weirs. They were also provided with deep sheet pile cut offs, the use of well foundations having been reduced to the minimum. A reference to Fig. 1.14 (a-d), showing sections of the new weirs will clearly indicate the departure from the older type designs. In the design of weirs for Sutlej Valley Project the river flow was concentrated into a narrow channel by a pair of long guide banks form 2500 to 3500 feet in length. This not only induced a clear and direct flow on to the weir but also reduced the number of training spurs. Many older design concepts such as undersluices, divide walls, relative elevation of weir, undersluices and canal head regulator crests continued to be adopted for these new weirs also.

1.7.1 Sukkur Barrage Project (1923-33)

Almost contemporary with Sutlej Valley Project in 1923-30 the construction of the World's most gigantic project namely the Sukkur Barrage Project was also undertaken. The barrage was about a mile long and was designed in certain features after the Esna Barrage on the Nile. It consisted of a wide masonry floor founded on the sandy river bed protected by stone apron on the upstream side and concrete blocks as well as loose stone apron on the downstream side. Below the floor there were sheet piling both on the upstream and downstream side. The floor of the barrage was depressed below the lowest average bed level of the river at the site so that it should not interfere with the flow of silt through the Bukhur gorge. The weir was designed on Bligh's theory and the length of creep worked out as 315 feet and the balance was provided by vertical sheet pile lines at the upstream and downstream ends as well as in between as shown in Fig. 1.15(a) Fortunately, these sheet pile lines provided a sufficient factor of safety even when checked with the later theories developed.

Sir Claude Inglis[5] predicted from his experiments on Sukkur Barrage that the barrage as located below Bukkar gorge would pass comparatively silt free water into left bank canal and there would be tendency for shoaling in the river along the right bank and consequently more silt will enter the right bank canals. The prediction came true. In 1938 there was rise of more than five feet in the bed level of the canal. After carrying out further experiments some measures shown in Fig. 1.15 (b) were adopted to induce artificial curvature of flow in the right bank for sediment exclusion.

1.7.2 HAVELI AND THAL CANAL PROJECTS - (TRIMMU, AN JINNAH BARRAGES)

Trimmu Barrage

Trimmu Barrage located below the confluence of rivers Jhelum and Chenab was the first barrage constructed in Indo-Pakistan Sub-continent during 1937-40 on Khosla's theory. Again this was the first weir for which the downstream floor has been designed as flexible raft with a maximum thickness of 3.5 feet only. The section of the weir is shown in Fig.1.16(a). The designers in this case had the full facility of testing the design in the laboratory prior to undertaking the actual construction. The location of the barrage, the length and shape of the guide banks, the design of the silt excluder and the section of the weir with energy dissipation devices were all finalised after detailed model studies. Even the river diversion procedure through the barrage after its construction was decided after a thorough testing on the model. The location of the barrage with the layout of guide banks is shown in Fig. 1.16(b). The idea of silt ejection from the canal by constructing a silt ejector at some suitable distance below the main head regulator was practically put into practice when Haveli canal silt ejector was constructed at 2000 feet downstream of the main regulator.

Jinnah Barrage

Jinnah Barrage, 3797 feet long was built on Indus as it just debouches from hills near Kalabagh. It was designed for a discharge of 950,000. It was the second barrage on Indus, Sukkur Barrage being the first. This barrage was also constructed after thorough model studies on different designs, operational and construction problems. It has a silt excluder covering two bays of the left pocket, diverging guide banks, two fish ladders. Silt Ejectors were also constructed in the Thal Main Canal. The section of weir, and Barrage and guide banks in plan are shown in Fig. 1.17(a and b).

The hydraulic data of barrages constructed during 1900 to 1950 is summarised in Table I.IV and I.V.

1.8.0 DISCUSSION OF MAJOR ADVANCEMENTS IN THEORY AND DESIGN OF BARRAGES CONSTRUCTED IN THE PERIOD 1900-1950.

Major advancements in the theory of design and methods of construction of new weirs occurred in this period. Rapid developments were made in the period after the first World War. The most important developments mainly relate to the following aspects of design problem :

(1) River training and control for the weirs and barrages.

(2) Change from weirs equipped with shutters to gated weirs or barrages.

(3) The development of theory of hydraulic jump and its use in the design of stilling basins.

(4) Replacement of creep theory by seepage theory based on flow potential for computing uplift pressures, exit gradients and seepage discharge.

(5) The development of model science and the use of model as a tool for solving hydraulic problems of weir design and river control.

(6) Evolution of energy dissipation and scour control devices as a result of model studies.

(7) The development of idea of sediment exclusion at the head works by silt excluders and from canals by silt ejectors.

(8) The study of accretion and retrogression phenomenon in rivers specially at weirs and barrages and the provision for the same in the design.

The historical development of the ideas in the above fields are associated intimately with the new weirs and barrages constructed in this period. In fact some of the new concepts and devices incorporated for the first time on these barrages were put to practical test.

1.8.1 RIVER TRAINING AND CONTROL

Sir Francis Spring[6] an eminent Railway Engineer who wrote famous publication on river training and control in 1903, which todate remains authentic guide book on the subject, discussed the river behaviour of alluvial meandering rivers of Northern India at some of Railway bridges. Prior practice was to design the waterway for a bridge or a barrage for almost full Khadir width with the result that shoals and 'belas' masked the greater part of the waterway. The main flow passed only through part of the waterway. Sir Francis Spring discussed the function of the guide banks, their length and layout and the shape of the guide bank head. All future bridges and barrages were provided with guide banks.

The method of constructing weirs having raised crest block with flap type shutters on the top and rock fill between curtain walls made impervious and rigid at the top by grouting or ashlar rubble masonry did not provide easy regulation. The falling and raising of shutters apart from inducing cross flow imposed great restriction in river control at the head works. The experiment of low crested weir with shutters for Rasul proved a failure and the damages caused in the first few floods forced the engineers to raise the crest of the weir. In fact it was realised that if the crest has to be taken low as it had to be when the waterway is confined and restricted by guide banks regulator gates in each bay of the weir were essential. Thus barrages with restricted waterway and a lower weir crest emerged. The gate regulation on a deeper crest replaced the falling shutters on high crest level so commonly used in the nineteenth century. The conventional undersluicing pocket with a divide wall and a canal head regulator having raised crest continued to be a feature as in the previous designs.

1.8.2 THE THEORY OF STANDING WAVE OR HYDRAULIC JUMP AND DESIGN OF STILLING BASINS

The mathematical theory of standing wave or hydraulic jump was first worked out by Jhon Belanger in 1840, who used Bernoulli's principle. He, however, completely ignored the accompanying energy loss. Using Belanger's notes, Jacques developed a correct equation for the jump utilising $\frac{V_1 D_1}{g}$ as a flow parameter. This is reproduced in Bresse's[7] Course De Mechanique Appliquee (Paris, 1860). This was also included in 9th Edition of Encyclopedia Britanica by W.C. Unwin in his article under hydrodynamics. The theory of standing wave or hydraulic jump also formed a subject of a paper by Gibson[8,9] to the Institute of Civil Engineers (1910). Kennison[10] presented a paper on the same subject in 1916 to the American Society of Civil Engineers. Later works of Woodward Sherman[11] (1917), Jullian Hinds[12] (1920) and Bakhmeteff[13] (1932) in USA, emphasised and elucidated the theory and its practical aspects of design. In India Montagu[14] (1929) and Crump[15] (1930) introduced the theory to Irrigation Engineers in the form so that it could be practically used in the design of hydraulic structures. With the development of the theory there was now a rational basis available for fixing the level of the d/s floor. Bakhmeteff having worked out the length of the jump, some yardstick was available for fixing the apron length of the stilling basin. Theory and its application to design of hydraulic structures need a separate paper.

1.8.3 CREEP THEORY REPLACED BY POTENTIAL THEORY OF SEEPAGE FLOW

Detailed discussion of the historical development of the theory of potential flow and its application to design of hydraulic structures has been made in Ref.1. During the period 1929-32, Khosla repudiated Bligh's creep theory which had so far been the basis of design of 'hydraulic structures on Permeable foundations.' In the years 1932-35, very extensive work was done in this field in the Irrigation Research Institute, and all the barrages after 1935 such as Trimmu on Chenab and Kalabagh on Indus were designed on Khosla's theory as far as under seepage flow was considered. The design aspects of the problem relating to under seepage have been comprehensively dealt in a book written by the author (see Ref. 1.).

1.8.4 DEVELOPMENT OF MODEL SCIENCE AND USE OF MODELS IN DESIGN OF HYDRAULIC STRUCTURES

One of the most important development during the later half of the 19th century was the development of the model science.

In the past, some of the flow problems were solved by mathematicians by the application of principles of classical hydrodynamics dealing with an ideal fluid neglecting the fluid properties such as viscosity, surface tension etc. The hydraulic engineers on the other hand, as practical men followed the empirical approach and used constants and coefficients to fit the observed data and thereby produced working rules and formulae. Such coefficients were supposed to take care of all known and unknown factors. The experimentation in hydraulics started in right earnest with Leonardo da Vinci in the fifteenth century when he studied flow phenomenon and illustrated eddy motion, waves and jets from weirs and orifices.

In the eighteenth century, Dubuat, the French Hydraulician made series of experiments on flow in pipes and channels and resistance of immersed bodies. His experiments set an era of experimental hydraulics. Dracy and Bazin[16] in late 19th century made use of first flume 1478 feet long, 6.5 feet wide and 3.12 feet high to study the effect of roughening of conduit surface on friction loss of head.

The last 30 years of the 19th century saw the development of three emperimental techniques for small scale testing which has continued to play leading role in the present day laboratory practice, one involved the wind tunnel, another the moveable bed river model and a third a towing tank. Dealing with developments in the hydraulic field, Froude[17] and Fargue[18] are the pioneers.

William Froude (1810-79) and his son Robert Froude (1846-24) initiated the towing tank research technique and enunciated the Froudes Law so commonly used in model science. Fargue constructed the first river model of Garonne river in 1875. His model was made to 1/100 scale and depth and time scales were arbitrarily chosen. Osborn Reynlod[19] in 1885 for the first time constructed and experimented with a tidal model of river Mersey on properly corelated scales. After completing the study he remarked; "From what I have seen from this model I consider it madness to embark on any Engineering Project without the help of the model studies". Today models play an important role in solving problems of design of hydraulic structures and river training and control problems. Previously the engineers were training the rivers, and constructing the weirs and other hydraulic structures without the help of model studies on the basis of existing knowledge and past experience of successes and failures. This process in fact of experimentation on full scale is very aptly described by Francis Spring when he wrote "Engineers have gone on blundering benefiting rather by chance than design by the experience of their predecessors and each considering himself lucky if he escapes disaster at the hands of the tremendous forces of nature amongst which some of the most potent for good or evil are great rivers with which he has to struggle". How right he was only engineers dealing with great rivers can realise.

The rapid increase in the number, magnitude and complexity of hydraulic flow problems resulting from the fact theat larger structures for higher heads and velocities had to be designed, forced the engineers more and more to resort to model testing. For example in the past it was enough to determine the .coefficient of discharge of a spillway, an orifice or a weir but it is no more sufficient now. It is now essential to keep the boundary geometry of the structure in view, to think of its profile, over which the water has to pass at very high velocities. A small error may cause cavitation, destructive pitting action and vibrations. Determination of detailed pressure distribution to avoid zones of separation and regions of cavitation is essential.

From the brief introduction of a hydraulic model as tool for getting an answer to design problems, one should not form the impression that a model would provide quantitative answer to any engineering problem. Hydraulic model at the best is to be taken as a tool having limitations in giving quantitative and qualitative answers even in the hands of an experienced model interpreter.

1.8.5 ENERGY DISSIPATION AND SCOUR CONTROL DEVICES

The dissipation of kinetic energy of the water flowing over a hydraulic structure so that if will not cause undesirable erosion or scour downstream of the structure has always been a problem contronting the engineers. Scientists have used the junp as device to mix chemicals and aerate water, but hydraulic engineer is mainly interested in its function as energy dissipator for high velocity water emerging at the feet of the spillway slopes. His objective was to determine the forms of the jump that are most efficient as energy dissipators avoiding the forms of the jump that do not effectively dissipate energy. Further investigations have shown that the junp efficiency can be related to the Froude Number of the incoming high velocity flow. The elevation of the stilling basin floor has now to be fixed by providing the sequent depth in the stilling basin to form the hydraulic jump. If the tailwater is well below the designed value, the jump will sweep out, and the high velocity at the erodible bed downstream of the stilling basin will cause heavy scour thereby endangering the safety of the work. Obviously in the design there must be provision n for the retrogression in down stream water levels. The length of the stilling basin has also to be related to the length of the hydraulic jump so that the flow as it emerges from the unerodible part of the stilling basin to the erodible bed downstream has reduced bed velocities and consequently little erosive action. Again the jump should not be such that excessive waves are generated to cause damage to the banks.

In actual practice and from model studies it was clearly demonstrated that a hydraulic jump at sequent depth on a horizontal floor is a very sensitive phenomenon. The jumps can move considerably downstream with a decrease in sequent depth. Studies carried out in the Laboratories in Europe, America and Indo-Pakistan Sub-continent have evolved different devices such as dentated sills, baffles, baffle blocks and staggered blocks etc which help to

(a) Move the jump up.

(b) Prevent sweep out of the jump for given decrease in tailwater levels.

(c) Reduce the length of the basin.

(d) Correct velocity distribution in the vertical section downstream of the hydraulic jump to conform nearly to the normal velocity distribution in a channel.

(e) Reduce the localised scour depth considerably as well as shift the point of maximum scour well away from the downstream end of the structure forming a reverse ground roller to pile up material against the downstream toe wall.

The phenomenon of dissipation of energy in the hydraulic jump, the design of stilling basins for maximum energy dissipation and the devices for the stabilisation of the jump in the stilling basin and for prevention of excessive scour need a separate paper.

1.8.6 SEDIMENT EXCLUSION METHODS AND DEVICES

In spite of the fact that the advantages of the raised crest of the regulator, the use of the still pond system of regulation, the function of the divide wall and the pocket, the creation of curvature of flow and its bifurcation for sediment control at canal head were appreciated by engineers and were made use of as early as the year 1900, all the implications and limitations of these measures were however not fully realised. A lot more work was needed to know the optimum width of the pocket, the optimum length of the divide wall, the effective height of the crest of the regulator, and the real implications of the order of velocity in the pocket as compared to the general velocities on the river side as far as the sediment exclusion was concerned. The Sirhind Canal at Rupar takes off from the concave bend of the river. The engineers of those days had even stabilised the outer bank of the curve by means of a series of spurs with a right channel bifurcation about 3 miles upstream of the weir. The survey plan of the river (1917-18), as reproduced from the Punjab Irrigation Branch[20] Paper No. 21,22 in Fig. 1.7-a clearly shows the stabilised curvature and right hand channel with steeper slope to withdraw heavier sediment charge towards the inside of the curve. This idea was further developed and used at Khanki.

In the Lower Chenab Canal taking off from Khanki weir the silt trouble was as old as the canal itself. In 1907 and 1909, a large sum of money had to be spent on silt clearance. Some of the devices, such as, the still pond system of regulation which had proved successful at Rupar were also tried there. The crest of the barrage had to be raised twice by 2 ft. First in 1911-12 and again in 1921-22, because the control was being lost and the still pond system could not be applied due to lack of command in the river. In 1931-32, the right channel was developed by linking the middle bay No.5 by means of a bund with the upstream bela which extended to two miles. For the maintenance of the curvature at the off-take, Bay No.4 was depressed, and Bay No.8 was also depressed to keep the right channel open and clear. As stated by Khosla[21] in his paper, see Fig. 1.9(a) "The bifurcation of the channel two miles upstream and creation of a curvature for the offtaking channel had considerable effect in reducing the silt entry".

In this period another achievement of great significance was the development of the idea of silt excluders, to be constructed in the undersluice. The Lower Chenab Canal taking off at Khanki continued giving silt trouble. The controversy of semi-open flow and still pond system and the drawbacks of frequent closures for sluicing due to application of still pond system was very much mooted among the engineers of the time. Elsdon[22] suggested that since bottom flow could be drawn by the undersluices and the top layers made to flow into the canal, the silt entry into the canal could be very much minimised. He suggested a hinged flap at the uper end of a diaphragm to vary the line of separation of two stream without causing disturbance to incoming flow. The idea was not accepted as practical. However, in 1934 Nicholson designed and built the first silt excluder at Khanki.

1.87 ACCRETION AND RETROGRESSION IN STREAMS AS A RESULT OF THE CONSTRUCTION OF WEIRS BARRAGES & REGULATORS.

A rising trend of gauge height at a particular gauge site in a river for the same dischage indicates accretion and falling trend for a similar fixed descharge a retrogression tendency. The changes in level at a gauge site for the same discharge could be due to any of the following causes.

(1) River meanders and consequent change in river length.
(2) Construction of weirs, barrages or dams across the river.
(3) A change in silt charge, either in quality or grade or both Higher silt charges and courser bed material requiring steeper slope.
(4) Artificial changes in discharge, and slope by regulation for a given section.
(5) The set of the river towards the gauge site.

The changes in river levels for a specific discharge can be due to any one of more causes in a river reach well away from the confluence of one river with the other or from outfall of the river into the sea. For a gauge in a river reach downstream which it outfalls into sea or joins another river, the gauge level for a specific discharge can also be affected by :

(1) The cycle of high or low tide in the sea.
(2) The change in the extent of delta at the river mouth.
(3) For a guage site above confluence of a river with another river, the stage in the outfalling river will cause a sympathetic change in the gauge if it happens to be in the back water range.

The effect of weirs and barrages on river regime cannot be discussed in this paper. Tracing the historical development of the concept, it is to be noted that Messers Trench and Nicholson[23] in 1930 first summarised the effect of weirs and barrage in river regime as below :

(i) On heading up, the river slope upsteam is flattened, thus raising the levels upstream by the amount of heading up imposed by the weir. This effects the river regime upstream.
(ii) The water level downstream of the weir is lowered due to the reduced supplies and retrogression of levels consequent on reduced silt charge. The retrogression is followed in a period of some years by a restoration of the levels downstream to pre-weir levels.

Foy[24] during 1944-45 made a detailed analysis of specific discharge gauge data with the help of Irrigation Research Institute, and generally corborated the conclusions of Nicholson and Trench.

In Article 1.8.6, it was indicated that for proper dissipation of energy and control of scour, the water levels downstream of a structure must be such that the actual depth is equal to or greater than the conjugate depth. If a structure is constructed without providing in the design for feature retrogression, the actual level attained downstream can be lower than the designed level. In such a case the hydraulic jump can sweep out of the stilling basin causing heavy scour and failure of the work. An example of failure of a structure due to this cause is found in Islam barrage on Sutlej river, which was completed in 1928. On August 24, 1929, a discharge of 228500 cfs was experienced with downstream level of 448.6 which was about 4 feet lower than the level required to from a jump at the toe of the structure. As a result six bays of the weir[25] were washed away.

The work started by Trench, Nicholson and Foy on changes in river regime was continued for Punjab rivers in the Irrigation Research Institute[26] and the analysis was updated to 1956.

1.9.0 BARRAGES CONSTRUCTED DURING THE PERIOD 1950-75

The period between 1950 to 1975 can be divided into two parts. In the first period upto about 1962 three new baggages were constructed on the Indus river at Kotri, Taunsa and Guddu. In 1960, after the treaty on the distribution of waters of the Indus river system between Pakistan and India, according to which waters of the rivers Ravi, Beas and Sutlej were allocated to India, and the flow of Western rivers, the Indus main, the Jhelum an the Chenab averaging about 137 million acre feet of water, were reserved for the use of Pakistan. It became very essential for Pakistan to ensure supplies to areas which were formerly dependent upon the Western Rivers. Very extensive program of replacemnet works involving the necessity to impound, regulate and transfer water across the entire breadth of Pakistan had therefor to be undertaken by the Government of Pakistan. In the second period from 1962 to 1975, five new barrges, namely Sidhnai on Ravi, Rasul on Jhelum, Qadirabad and Marala on Chenab and Chashma on the zindus had to be constructed to control regulate and transfer the water through the inter-river link canals to rejuvinate the existing irrigation system. A total length of canals nearly 375 miles long had to be constructed with a large number of regulators, canal falls etc. Apart from the above, Mailsi Siphon-cum-Barrage on Sutlej river was constructed to allow the Mailsi Bahawal link canal to cross the river Sutlej and Balloki weir had to be remodelled. The barrages and links constructed are marked in Fig.1.18.

Two dams, one at Mangla on Jhelum river and the other at Tarbela on Indus were also planned and executed to make up the shortages and augment the supplies. Mangla dam has a storage capacity of 5.88 MAF and height of about 386, feet whereas Tarbela dam has a gross storage capacity of 11.1 MAF and the maximum height above