"A STUDY OF THE CONTROL OF BED LOAD AND SUSPENDED LOAD IN CANALS AND DISTRIBUTARIES"

THESIS SUBMITTED

BY

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C_E_R_T_I_F_I_C_A_T_E

Certified that the thesis entitled "A study of the Control of Bed Load and suspended load in Canals and Distributaries" which is being submitted by Shri J.B.Asthana in partial fulfilment for the award of the Degree of Master of Engineering in Dam Design, Irrigation Engineering & Hydraulics of University of Roorkee is a record of the student's own work carried out by him under my supervision and guidance. The matter embodied in this thesis has not been submitted for the award of any other Degree or Diploma.

This is further to certify that he has worked for a total period of about one year and three months from June, 1958 to November, 1958 and from September, 1964 to May, 1965. He had worked on this dissertation for over six month's period here at this University.

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SYNOPSIS

This paper, as its name suggests, presents a study of the various measures adopted from time to time to control sediment load in canals and distributaries. There is pleanty of literature about the theories of sediment transportation, design of channels etc. From the practical point of view, however, they are not sufficient. In this paper, the problem of the control of semident in canals and distributaries, as it is known today, has been examined. The significance and implications of the various factors involved have been emphasised; controversial issues have been discussed; and efforts have been made to stress the practical systems of control, so as to choose in the first place one which is fundamentally well suited for the purpose; and having chosen it, have a sufficient knowledge of it's charecter, its possibilities and its limitations to enable the work up of the main lines of design with economy and imagination. This kind of approach - broad and general, but factual - seems to the author to be most useful to the Canal Engineer.

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LIST OF SYMBOLS.

a,A	a constant
b,B	Breadth a constant
c	Concentration a coefficient
C	Chezy discharge coeficient a constant
d,D	depth of flow
E	height
f	coefficient of friction
g	gravitational acceleration
h	height
hf	loss of head
l,L	length
k	height of roughness
M	Mass
m	mean grade of material
n	any number Mannings roughness coefficient
P	wetted perimeter
qo	critical discharge
q,Q	discharge
qs	Volume rate of sediment transport/unit width
R	Hydraulic mean radius Radius of curvature of flow
Re	Reynolds number
R_1	Radius of inner bank from reference axis
R_2	Radius of outer bank from reference axis
S	Slope every gradient
S_o	bottom slope
S_s	specific gravity of sediment
t,T	time

Contd.

- -

V_o	$\sqrt{To/p}$
v, V	Velocity
V_p	velocity in pocket
V_t	tangential velocity
W	Weight
w	fall velocity of sediment
X	distance from reference axis
x	cartesian coordinate
	distance in direction of flow
	power of a variable
y	cartesian coordinate
	power of a variable
	an angle
	specific weight
T	intensity of tractive force
To	critical tractive force
	specific gravity of sediment
	angle of offtake
	kinematic viscocity
	mass density

INTRODUCTION

The problem of the control of suspended load and Bed load in Canals and distributaries is one which is frequently encountered in the design and operation of irrigation and power canals. The difficulties which confront the Irrigation Canal Engineer either directly or indirectly due to the silt which is brought down in the rivers and enters the canal system cannot be over estimated. The sucess of a canal system depends on the degree in which the ill effects of this silt can be overcome or obviated.

The importance of the sediment problem was recognised ever since the present canal system controlled by Headworks has come into existence, espicially in the alluvial valleys of North India. India occupies the pride of place as the country, having done almost all the research work on this subject. Rivers in India carry large quantity of sediment during the Monsoon season. Particularly, the rivers originating from the great mountains of Himalaya transport heavy sediment consisting of boulders 2 to 3 ft in diameter, gravel, coarse sand etc. During floods several Million c.ft. of this material is transported per day. Some of the mature streams carry sediment load much larger than the Mississipi, Rio Pucrco, San Juin, Rio Grande, Colorado or Rhine.

The irrigation canals or Hydel canals taking off from such rivers draw an enormous quantity of sediment load. In the case of irrigation channels as the main canals bifurcate and branch out the capacity of the water to transport sediment becomes less and less, both as regards quantity and size of particles, as the canal discharge is reduced. Finally the small water channels for irrigating fields can carry only some sediments of the finer fractions. This results in the silting of canals and distributaries, and a very large amount of labour and expense is needed every year to maintain the canal system in efficient working order. The sediment has often to be excavated at huge expense and the areas required for depositing the excavated sediment becomes an important problem. Often it necessitates the closure of the canals, effecting the crops.

Thus the problem is of the greatest importance and needs the attention of all at the planning stage. The great need of India today is the proper harnessing and development of her irrigation and power resourses. It is hence fitting that Engineers and scientists in the country have been taking a keen interest in solving this problem.

Considering a river system as a whole, the problem starts at the very sourse of supply. The solution appears to be the control of sediment at the very sourse of supply and the discharging of the sediment present in the canals into natural channels. The problem is tackled in two ways. One is to fix the section and slope of the channel by taking into account the quantity and quality of silt so that it will neither scour nor silt up, and the second is to exclude or control the entry of silt into the canal system. This paper deals solely with the second aspect.

The design of canals, capable of transporting a certain amount and quality of sediment with a spefified discharge is essentially a problem of bed load transportation. Various formulae have been evolved by Dupit, Kennedy, Lacy, Einstein and others which are of immense practical value. Experience has shown that this is not enough. The physical conditions of the country often make it impossible to design a canal to carry all the sediment and it is necessary to reduce the silt charge in the water available before it can be carried in any channel even if designed in accordance with the best known principles.

The various methods of controlling Bed load and suspended load in a canal system, their principles, advantages and disadvantages with suggestions have been dealt in this paper. Recent advances have been made in the measures to be adopted for excluding or controlling the entry of silt from canal heads. Advances in model experiments have led to proper design of approaches and other silt excluding devices. Where, however, the silt exclusion from the head does not prove sufficiently effective, use is made of yet another device, namely of silt ejectors for ejecting silt from the bed of the canal by allowing the bottom silt laden water to escape out. Various other devices have also been discussed.

STATEMENT OF THE PROBLEM

Before going into the details of the control of sediment, it is best to have a proper perspective of the problem. The words "suspended load" and "bed load" used in this paper are the universally accepted divisions of the sediment load or silt charge. There is no line of demarcation between suspended load and bed load. In the same channel a given material may be carried in suspension in one reach and as bed load in another.

Silt originates from the disintegration of rocks by the climatic agencies of rain, wind, and frost and by chemical agencies in water and air. The effect of such disintegration is evidenced by the wearing down of mountains, the gullying of their slopes, the filling of the valleys, the extension of deltas into seas etc. The process of disintegration, erosion, transportation, sedimentation, mountain building and plain levelling are still in progress. Existing topography is a complex residual of these processes in which eons have gone by.

Man cannot hope to halt the process of mountain erosion and plain building. The land he cultivates could not exist but for these forces. He must accept that rains will gully his fields, or cover them with mountain debris, that the streams will continue to carry sediments that will fill the canals. Man's problem lies in utilising the agencies of sedimentation to his advantage where possible and in opposing the nullification of his endeavors by controlling them through whatever forces lie at his command.

A Transportation of Silt:

Flowing water has the power to transport large quantities of finely divided material as a suspended load and also to drag other materials along its bed as bed load. The higher the velocity and the more turbulent the stream, the greater the propartion of suspended load it is capable of carrying. When the velocity is reduced this material settles and the bed load movement is arrested. Material remains in suspention by virture of the vertical components of currents and eddies within the water prism.

The shape of the silt particle has an important bearing on the facility with which it remains in suspension or settles to the bottom. The finer the material, the slower it settles; rounded particles will settle much faster then flat scale-like particles; and disk- like particles require a long time. The velocity of water, the degree of fineness of the material, and the predominating shape of the particles are three correlated factors that determine the ratio between suspended load and bed load of a stream.

On mountain streams the bed load consists of boulders, cobble stones and coarse sand. The transporting of stones along a stream grinds them, ultimately, into material sufficiently fine to permit their being carried to the ocean as suspended load or as bed load of slow-moving rivers. By far the greater part of the sediment transportation by streams occurs at the time of a flood. The total suspended load carried into the oceans by the rivers of the U.S.A. per annum is estimated as 513,000,000 tons.

Table 1 gives the suspended silt contents of the rivers of the world:

TABLE 1.

DATE	RIVER	LOCATION	SUSPENDED SILT PER THOUSAND	SUSPENDED SILT MILLIONS OF TONS
1928-29	Colorado	Arizone, U.S.A.	18.2	480
1929-30	San Juan	Utah, U.S.A.	19.3	45.6
1929-30	Mississipi	U.S.A.	0.81	582
1931-32	Missouri	U.S.A.	4.10	253
1911-12	Rio Grande	N. Mexico.	35.0	85.2
1922-23	Nile	Egypt.	1.02	100.0
1920-21	Orange	O.R.Sta., C.P.	6.9	534
1900-01	Krishna	Madras	3.41	262
1901-02	"	"	23.3	....
1910-11	Indus	Sind.	4.02	708
1902-03	"	"	15.8	....
1894-95	Sutlej	Punjab	29.4	....
1895	Sussulpore	Bengal	40.0	....
1923-24	Yangtse	China	0.36	374
1919-20	Huang Ho	"	39.5	1250

We see that in India, all the great rivers are silt laden streams. The Indus with its principal tributary, the Sutlej, waters the westen portion, now mostly in Pakistan. The eastern portion is drained by the Brahmaputra; while between them lies the holy Ganges. The Indus Valley is quite arid and is covered with fine alluvial silts and wind blown sands. The ganges and Brahmaputra areas have a seasonal climate. Parts of these areas are seasonally dry; Irrigation is profitable and constitutes the profession of the majority of the population. From these areas enormous quantities of silts are supplied to the rivers so that the rivers flow on broad alluvial deposits, which although about 2000 ft. thick, are geologically recent.

Most streams run fairly clear at low stages, but while so doing they may move quantities of sand and detrius along their beds. At high stages a suspended silt load develops and the stream is said to run 'muddy'. At every reduction of velocity both suspended load and bed load are deposited, forming bars, berms, deltas, valleys and alluvial plains. Some rivers carry a suspended charge at all times. In areas of good rainfall, the streams are capable of carrying off all the detrius resulting from rock disintegration as fast as it accumulates. The result is that the stream runs clear except at times of flood. On arid areas the debris from disintegration may remain in place for many years until an unusual rainfall occurs, when great quantities of silt are transported. If the drainage area is large enough with many tributaries the 'unusual' rainfall is almost contineously occuring at some place or the other. This keeps the main river contineously supplied with an abundance of silt, which it must carry at all stages. Of such are the watersheds of the Colarads, Missouri, Rio Grande, Yellow and Indus Rivers.

Need for Sediment Control:

It has been shown that rivers, especially in alluvial plains of India, carry large quantities of suspende load and bed load. In general, diversion is by means of a weir or barrage. Several canals may head from one diversion dam and generally from both sides of the river. The trouble starts when excess of sediment enters at the head regulator and silts up the channels.

An ideal irrigation channel is generally just sufficient to pass the designed disharge under designed conditions of discharge and normal charges which the canal can carry downstream with a certain amount of latitude. It has already been stated that the silt carrying capacity of water depends on discharge, surface slope, grade and shape of silt and silt charge. Owing to the physical conditions of the land canals have to be designed with slopes flatter than those of the rivers from which they take off so as to command the areas to be irrigated, and the volume of flow in a canal is usually much less than that in the river from which it takes off, particularly during the rainy season and in some cases, during the early hot weather also, due to melting of snow. This sudden reduction in the volume and slope of flow in the canals is the main cause which leads to silting. Some times bad regulation, such as the sudden lowering of the pond level at the intake, may also cause large quantities of bed load to move from the river into the canal.

The physical conditions of a valley often make it impossible to design channals that will carry all the silt contained in the only water available for it. The velocity that is practicable for an irrigation canal in alluvium will generally be able to carry in suspension most of the finer sediment. As will be seen later, this fine sediment which reaches the fields is not harmful. ~~The~~ Therefore, preventing coarser sediment from entering canal system is the prime factor in the success of irrigation projects, in that it eliminates the high annual expenditures for sediment disposal and control in the canal system and upon the land, provides a more free passage of water through canals and renders structures servicable and operative.

Several measures can be adopted to control sediment load entering a canal system, and those will now be discussed in some details in the following chapters. Prior to that a few important implications will be considered.

Utility of Silt:

Silt is a good friend when properly tackled. Even the heavy particles of sand, which roll on the bed and are such nuisance to the Canal Engineer are said to be not without utility. Nature's economy is perfect. Silt is a good fertilising agent. It facilitates the early formation of beArms along canals which means a good saving in costs and even then results will not be so good. It reduces seepage by forming a good lining at the sides and bed of the channel and by making canal water turbid it prevents the sun's rays from reaching the bed and thus stops weed growth.

REVIEW OF WORK DONE ON SEDIMENT TRANSPORTATION

Engineers since the earliest times have experienced difficulties due to sediment carried by natural streams. However, the manner in which fluvial sediment is transported and deposited, a knowledge of which would aid in avoiding or overcoming many of these difficulties, has been investigated only in comparatively recent years.

The Chinese were probably the first to study silt problems in relation to floods, on the Yellow River. C.H. Pan, a high officer and river expert in the Ming Dynasty (1368-1643) devoted himself to studying the problems of the Yellow River for 27 years (1565-1591).

The first investigations were made in Italy in the latter part of the 17th century. The fundamentals of the sediment problem were investigated scientifically in France in the 18th Century, but so far as can be determined, it was not until the early part of the 19th Century that quantitative observations of sediment carried by natural streams were made.

Frizi (1762) and Guglielmini (1697), the two pioneers in the investigation of river problems, stated their rules bearing on the transportation of sediment and detritus in terms of discharge and slope.

Du Buat (1786), Bounicean (1845), Blackwell (1857), Gras (1857), Lechalas (1871), Suchier (1874), Deacon (1894) and Kennedy (1895), among the earlier investigators, and Schaffernak (1922), Fortier and Scobey (1926), Lacey (1930), Ho (1933) and Tu (1934), among the later ones, used either the bottom velocity or the mean velocity of the water as

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the principal determinant of competence in traction. Gras, Lechalas, Kennedy and the recent investigators have considered the bottom velocity, or the mean velocity and depth of flow, as the important variable. Lacey introduced another factor (known as silt factor f) to account for the characteristics of the detritus.

But this is only one side of the picture, The movement of detritus load is closely connected with the flow of water. Therefore, both the problems have to be taken into consideration at the same time.

In 1775 Chezy, a French Engineer, advanced the following formula for calculating the flow of water in open channels.

$$V = C\sqrt{RS} \qquad \ldots\ldots \quad \ldots\ldots \quad \ldots\ldots \ (1)$$

The coefficient C was supposed to care for all the various factors affecting the velocity. After some three-quarters of a century of use, it was found that C was not a constant but a rather complicated variable and Kutter's formula (expressed in English units).

$$V = \left\{ \frac{\frac{1.811}{NK} + 41.66 + \frac{0.00182}{S}}{1 + \left\{ 41.66 + \frac{0.00281}{S} \right\} \frac{N_k}{\sqrt{R}}} \right\} \sqrt{RS} \qquad \ldots\ldots \ (2)$$

was obtained. This was developed in 1869.

In an effort to modify the complex form of Kutter's formula, the Mannings' formula.

$$V = \frac{1.486}{N_M} R^{0.67} S^{0.50} \qquad \ldots\ldots \quad \ldots\ldots \ (3)$$

appeared some sixty years ago. After this came Bazin's formula

Credit for the first comprehensive formulation of the effect of relative roughness is due to Von Mises who derived from experimental evidence the dimensionally correct expression applicable alike to closed and open conditions:-

$$f = 0.0096 + 4\sqrt{\frac{k}{R}} + \frac{0.85}{\sqrt{R_e}} \quad .. \quad .. \quad .. \quad (5)$$

where f is the coefficient of friction, k the height of roughness (depending upon the size of the bed material), and R the hydraulic mean radius. At high values of the Reynolds number R_e, or for high magnitudes of $\frac{K}{R}$, the last term becomes negligible. The factor k, evidently must have the dimensions of length. Knowing the value of f the Equation

$$S = \frac{f}{4R} \quad \frac{V^2}{2g} \quad .. \quad .. \quad .. \quad (6)$$

can be made use of in design. Tables indicating the value of k for different materials used for the construction of channels have been prepared.

Thomas reviewed the application of such formulae employing the relative roughness to rigid and quasi-rigid channels with clear flow and a sediment transport.

The modern trend is toward using the logarithmic relationship, viz.,

$$\frac{V}{V_*} = 3.2 + 5.65 \log \left(\frac{\overset{*}{V}D}{y}\right) \quad .. \quad .. \quad .. \quad (7)$$

for infinitely wide channels of uniform depth D with smooth bed and for channels with rough beds. V is the mean velocity and

$$V_* = \sqrt{\frac{T_0}{p}}$$

where k is the size of roughness of the bed material, v the, kinematic viscosity of water which changes with its silt content T_0 the boundary shear stress of the water and p the density of the water.

Assuming the shape of the average channels between the semi-circular and uniform depth shapes, the formulae are

$$\frac{V}{V_*} = 3.45 + 5.65 \log \left\{ \frac{V_* R}{\nu} \right\}$$

for smooth channels (i.e. where $\frac{V_* k}{\nu} < 3$), and

$$\frac{V}{V_*} = 3.35 + 5.65 \log \left\{ \frac{R}{K} \right\}$$

for rough channels (i.e. where $\frac{V_* k}{\nu} > 70$)

White investigated under the controlled conditions of the laboratory the criteria for initiation of movement of sand particles. He distinguished between the case of purely viscous draf and the case in which the drag is due to eddies generated among the particles and derived coefficient from his data. An interesting conclusion was that lift on bed particles due to horizontal flow was negligible.

Kirkham pointed out that a modification was required in the relationship between suspended material and bed load given by Lane and Kalinske.

Kalinske discussed the criterion for initiation of sand particles on the bed, bed load transport and saltation. On the first point he concluded that experiments on bed sediment movement in laboratory or natural channels involving the measure of average velocity and drag cannot give fundamental data. Rather, what was necessary was the determination of values of standard deviation of turbulent velocity fluctuations and evaluation from such measurements of how the shear fluctuates. He derived an expression for rate of bed load movement based on the assumption that the velocity near the bed fluctuated according to the normal error law. Regarding saltation he calculated that the height of "bounce" of sand particles in water was only 1/800 of what it was in air, and concluded that in water streams when the velocity reaches sufficient magnitude to cause saltation, the turbulence would be such as to place the material in suspension and thus entirely obscure any saltation effect.

Kalinske also discussed the characteristics of turbulence in river hydraulics, presenting data indicating that the frequency of velocity fluctuation followed the normal error law. He also suggested that the depression of the filament of maximum velocity below the surface was caused partly by the passage of cortices upwards and exchanging momentum only slowly, resulting in an accumulation at the surface of water which waspart of slower moving eddies from the region of the boundaries.

Einstein gave a further discussion on a theory previously advanced with additional data collected in a small stream.

Kurdisny analysed experimental data of the initiation of bed movement, using the terminal velocity of particles as a parameter representative of the bed material after

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White and showing in graphical form relationship involving critical velocity, discharge and shear stress.

Langbein discussed the formation of sand waves in the beds of streams and concluded from Gilbert's data that such waves occur only when the flow is torrential, i.e. when $\frac{V}{\sqrt{gR}} > 1$, but that this critical value decreased with increase in sediment load, particularly the finer sizes.

Anderson gave the results of comparison of observed vertical distribution of sediment in suspension in a river, with the theoretical distribution derived by Rouse from the logarithmic velocity distribution and momentum transfer theory of turbulent flow. A similar comparison was made by Vanoni of observed distribution in a laboratory flume. The conclusion reached was that the sediment is more uniformly distributed than the theory would predict but the shape of the distribution curves agreed satisfactorily with the theoretical. Dobbins extended the theory of equilibriu distribution of suspended sediment to explain the concentra tion changes in a stream or settling basin under non-equilibrium conditions, verifying the results with data of artificially created turbulence.

A development of Dobbins theory was advanced by Camp being an approximate application of the theory to open channel flow to predict the effect of turbulence in retarding settling.

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Krumbein examined experimental data on the effect of particle shape in sediment transportation. He concluded that dynamic concepts were more significant than geometrical concepts, and that although sphericity afforded a first approximation to the dynamic behavious of particles, it was likely that some additional shape factor accounted for relative behavious of particles of equal sphericity but different geometrical form. He proposed an overall form coefficient derived from settling velocities.

In another paper Krumbein discussed particle properties which are fundamental from a sedimentary point of view in a sense broad enough to interest both geologists and hydraulic engineers.

Thomas reviewed the work done up to 1945. He compared the mean shear stress over the boundary of "regime" channels, calculated by the Lacey equations, with shear stress required for initiation of movement of similar bed material, based on observations in the experimental channels, and additional data of actual channels were presented to show that boundary shear stress generally increased considerably with depth of flow. he described four ways in which parties may be lifted from the bed into suspension and discussed the importance of various factors. It was shown that the greater the size of the particles between 0.1 and 2.0 mm., the greater the ratio of saltating bed load to particles lifted in suspension.

Thomas came to the disappointing general conclusion on flow in channels with sediment transport that the fresh data and increased knowledge of the mechanics of flow have shown that the problem is more complex than had been so far appreciated, and that the existing slope formulae are inadequate and many yield results having appreciable error.

SHORT REVIEW OF WORK DONE ON BED LOAD TRANSPORTATION

Work done by experimenters like Du Boys, Gilbert, Schoklitsch, Meyer Peter, Mac Dougall, O'Brien, Chang and at the U.S. Waterways Experiment Station, Vicksburg, is reviewed and empirical relationships obtained by them are presented below:

(i) Du Boys:- Du Boys derived a classical relationship in 1879 in the form:

$$q_s = CT\ (T - T_o)$$

where

q_s = bed load
T = unit tractive force on the bed γDS
T_o = critical tractive force required to initiate general bed movement, and
C = a function of size, etc. of the sediment.

This type of relationship held the field of bed load formula for many years.

Basing upon the general acceptance of the Du Boys relationship when supplied with suitable transportation characteristics, for the particular sediment concerned and adoption of Manning's formula for open channel flow, Straub has shown that Du Boys formula can be written in an alternative form:

$$q_s = CT\ (T - T_o) = C\ \frac{\gamma^2\ s^{1.4}\ q^{0.6}}{(1.49/n)^{1.2}}\ \left(q^{0.6} - q_o^{0.6}\right)$$

where C is the sediment characteristic--an experimental coefficient depending upon the size, specific gravity and mechanical composition of sediment.

(ii) Dr. Schoklitsch formula:- Mainly using the classic flume data of G.K. Gilbert and his own additional experimental data Dr. Schoklitsch developed the following formula for uniform grain material:-

$$q_s = \frac{86.7\ s^{1.5}\ (q - q_o)}{\sqrt{m}}$$

where

q_s = bed load intensity in lb/sec/ft. width

s = energy gradient ft/ft.

q = discharge intensity in cusecs/ft.

q_o = critical discharge intensity in cusecs/ft.

m = mean grade of material in inches, and

q_o = is given by

$$q_o = \frac{0.00532\ m}{s^{4/3}}$$

q_o was deduced by plotting qs : s for the same q and finding out s_o for no movement (q_s=0). This "Static threshold discharge intensity" --q_o for s_o, he called the critical discharge.

Dr. Schoklitsch's general formula was derived for uniform sands from 0.30 mm. to 7.01 mm. with q=0.0672 to 2.69 cs/ft. and the experimental flumes sloping from 12.0 to 55.0 per 1,000. The experimental flumes of Gilbert were 0.23 ft. to 1.96 feet wide.

Dr. Schoklitsch in deriving his formula allowed for the critical discharge q_o but made no allowance for the shape of the channel ($\frac{B}{D}$ ratio) nor did he allow for the effect of the side walls; and it may be stated that the effect of the side walls was quite high as the flumes used in the experiments were quite narrow.

(iii) Meyer Peter's formula:-~~E~~Meyer Peter of Zurich derived a formula from flume experiments which were carried out in 1.16 feet and 6.56 feet wide flumes.

His formula is:

$$q_s = \left\{ \frac{q_e^{2/3}\ s}{0.758} - \frac{m}{3} \right\}^{1/3}$$

where

q_s = bed load intensity in cu.ft/sec/foot.

q_e = effective discharge intensity allowing for side wall effect (Einstein)

s = energy gradient in ft/ft.

m = mean grade in ft.

Meyer Peter stated that his formula was found to hold good for his sand mixtures from 6 mm. to 40 mm., the equivalent or effective diameter being the size that is exceeded by 65 per cent . of the mixture by weight, i.e. 65 p.c. coarser than the equivalent size. In this case the discharge intensity was calculated by Einstein's method of eliminating the effect of of side walls. Meyer Peter made a constant deduction which depended on the mean grade of the sand or gravel mixture.

(iv) Mac Dougall: Mac Dougall conducted tests on bed load transportation at the Massachusetts Institute of Technology. The form of equation that fitted his data is:

$$q_s=C_s s^x (q-q_o)$$

the constants C and x are in terrelated and are dependent on m, the mean diameter of the grade of material and M-Karamer's uniformity modulus. Values of C and x range from 100 to 1,000 and from 1.25 to 2.0 respectively.

Mac Dougall has contributed to the problem by arriving at a approximate method for predicting the movement of bed load from an analysis of the material. His conclusions are based on observations of only three mixtures and his final curves have been drawn from only three points.

(v) Work done at U.S. Waterways Experiment Station, Vicksburg:- The U.S.Waterways Experiment Station conducted series of flume tests for evaluating the laws of bed load transportation so that the river hydraulicians might be able to calculate the action of a given bed material under given conditions.

The tests were carried out in a rectangular tilting flume 48 feet long, 2.3 feet wide and 1.3 feet deep. The materials used for testing consisted of eight sand mixtures of mean grain size 0.20 mm. to 0.59 mm. and one gravel mixture of mean grain size 4.08 mm. All the eight sand mixtures were tested with slopes 1.0, 1.5 and 2.0 per 1000 and the gravel mixture with slopes 3.0, 4.0 and 4.5 per 1000 in a tilting flume.

As a result of these tests the following formula was derived:-

$$q_s = \frac{1}{n} \frac{(DS - D_0 S_0)}{K_1}$$

where

q_s = Bed load intensity in lbs/ft.

n = Manning's coefficient.

DS = Product of depth and slope.

$D_0 S_0$ = Product of depth and slope at the time of movement--as determined from the linear plot of q_s n against DS.

It was hoped that the flume studies would help in ascertaining an accurate determination of the tractive force required to move the material at any point in the Lower Mississipi; but it was concluded that such sweeping conclusions could not be arrived at as a result of these studies. "The flume studies" it was claimed, "satisfactorily achieved the discovery of many of the basic laws underlying the subject of bed load movement".

(vi) O'Brien:- Prof.O'Brien derived quite a different type of relationship for the bed load for graded sands:

$$q_s = C\left(\frac{V^3}{D}\right)^x$$

In his flume tests on Columbia River material, Prof.O'Brien found that experimental data showed considerable variation when rate of movement was plotted against the following parameters:-

(i) Velocity at constant distance from the bottom,
(ii) Mean velocity of the flume,
(iii) U.S. Waterways Station method, and
(iv) The method of Schoklitsch.

In all cases the disagreement was greater than could be accounted for by the inaccuracy of measurement. Good results, however, were obtained by plotting rate of movement versus $\frac{V}{D^{1/3}}$ and that is the reason why he adopted this form in his final formula though it was quite a different form of bed load formula.

(vii) Y.L.Chang's work:- Y.L.Chang made laboratory investigations of the problem of flume traction and transportation. His laboratory flume was 18 feet long and only 12 inches wide which could be contracted to 6 inches width. The average diameter for 13 mixtures used ranged from 0.134 mm. to 8.09 mm.

A formula of the Du Boys type appeared to fit best to the experimental data of Chang. So he suggested that his formula can be written as:

$$q_s = \frac{km}{T_o^2} T (T-T_e)$$

where T_o is defined by

$$T_o = 0.0175 \left[\frac{\sigma - \rho}{\rho} \; m_o^{1/3} \right]^{\beta}$$

where β = 1 for $\frac{\sigma-\rho}{\rho} \cdot m_o^{1/3}$ greater than 1

and β = 1/2 for $\frac{\sigma-\rho}{\rho} \cdot m_o^{1/3}$ less than 1

where o is the ratio between the longest and the shortest diameter of the sand grain.

It may be noted that in contradiction to the U.S.Waterways Experiment Station formula--the bed load in Chang's formula varies directly as Manning's n. In support of his formula he stated that the bed load is always inversely proportional to a function of grain size; and according to U.S.Waterways Experiment Station Manning's n is also inversely proportional to grain size as ripples are higher with finer materials, as a result q_s should vary directly as Manning's n.

(viii) Remarks by Brown and Joe Johnson:- Carl B. Brown in his paper 'Sediment Transportation' remarking on the empirical formulas proposed from time to time stated--

> "Unfortunately, inherent difficulties in measuring the the rate of movement and of reducing the sediment characteristics to a single coefficient have resulted in discrepancies of as much as 1000 per cent. when these formulas are applied by different observers to the same field conditions".

Joe W.Johnson in his interesting discussion on Chang's paper statistically compared the various bed load formulae and remarked--

> "In flume studies there are three sources of error which in many instance have seriously limited the value of the results. These sources of error are:

(a) use of water surface slope instead of the slope of the energy gradient,

(b) the neglect of the retarding effect of the channel side walls, and

(c) the use of the relatively short collection period during which the bed material is trapped".

He made similar remarks in the discussion on H.A.Einstein's paper on "Formulas for the transportation of bed load".

Moreover in an overall examination of the work done on bed load transportation one finds that the laboratory flumes were as short as 18 feet and as narrow as even 6 inches. Even the U.S.Waterways Experiment Station flume was 48 feet long and 2.3 feet wide and 1.3 feet deep. With such short lengths of flume it is believed that the flume length in which equilibrium conditions would reach would be quite short as quite a good length at the upper end would be affected by the entry conditions and a portion of the lower end would be affected by the exit conditions.

Secondly it is difficult to obtain accurate measurements of slope in short flumes.

Again with narrow flumes it is felt that the effect of the side walls would be quite sufficient to vitiate the results and the results of different flumes of different widths with varying B/D ratio are not strictly comparable.

However the similarity of the various bedload formulae has been presented in the following Table. Also the similarity of Kennedy formulas with other Indian and foriegn formulas has been presented in a seperate Table to give a perspective of the present stage of work done on bed load movement.

TABLE: SIMILARITY OF BED LOAD AND 'KENNEDY' FORMULAS

Du Boys (Straub)	$\frac{V}{D^{5/9}} = C \frac{1}{n^{4/3}} \left(\frac{q_s}{q} \right)^{1/3}$
Schoklitsch	$\frac{V}{D^{2/3}} = C_2 \frac{M^{1/6}}{n} \left(\frac{q_s}{q} \right)^{1/3}$
Meyer Peter	$\frac{V}{D^{2/3}} = C \frac{1}{3n} \left(\frac{q_s}{q} \right)^{1/3}$
WES	$\frac{V}{D^{m'}}{}^{*} = C_5 \frac{1}{n} \left(\frac{q_s}{q} \right)^{1/(2m-1)}$
Shields	$\frac{V}{D^{5/12}} = C_6 \frac{m^{1/4}}{n} \left(\frac{q_s}{q} \right)^{1/4}$
Brown Einstein	$\frac{V}{D^{2/5}} = C_7 \frac{m^{3/10}}{n^{3/5}} \left(\frac{q_s}{q} \right)^{1/5}$
Brown Kalinske	$\frac{V}{D^{11/24}} = C_8 \frac{m^{1/4}}{n^{5/4}} \left(\frac{q_s}{q} \right)^{1/4}$
Kennedy	$\frac{V}{D^{0.64}} = 0.84$
Lacy	$\frac{V}{D^{1/2}} = 1.15\sqrt{f}$
Blench	$\frac{V}{D^{1/2}} = \sqrt{b}$
Inglis	$\frac{V}{D^{1/2}} = C_9 \frac{g^{1/3}\ w^{1/4}}{1/6} \left(\frac{q_s}{q} \right)^{1/4}$

$* \; m' = (m+3)/(6m-3)$

TABLE: SIMILARITY OF BED-LOAD FORMULAS

Du Boys (Straub)	$q_s = A_1 \; T(T-T_o) = B_1 \; n^4 \; \frac{V^4}{D^{2/3}}$
Schoklitsch	$q_s = \frac{A_2}{m^{1/2}} \; S^{3/2} \; (q-q_o) = B_2 \frac{n^3}{m^{1/2}} \; \frac{V^4}{D}$
Meyer Peter	$q_s = (A_3 \; q^{2/3} S - A_4 m)^{3/2} = B_3 \; n^3 \; \frac{V^4}{D}$
Waterways Experiment Station, U.S., Paper No. 17 Vicksburg, Misc. January, 1935	$q_s = \frac{A_5}{n} \; (T-T_o) = B_6 \; \frac{n^4}{m} \; \frac{V^{8x}}{D^{x/3}}$
Shields	$q_s = \frac{A_6}{m} \; q \; S \; (T-T_o) = B_6 \frac{n^4}{m} \; \frac{V^5}{D^{2/3}}$
Brown Einstein	$q_s = \frac{A_7}{m^{3/2}} \; T^3 = B_7 \frac{n^3}{n^{3/2}} \; \frac{V^6}{D}$
Brown Kalinske	$q_s = \frac{A_8}{m} \; T^{5/2} = B_8 \frac{n^5}{m} \; \frac{V^5}{D^{5/6}}$

SILT CONTROL IN CANALS & DISTRIBUTARIES

The various measures adopted for the control of sediment can be best ennumerated according to their place of application right from the sourse of supply to the field channels. Needless to say, the sediment problem is a complicated one and sediment engineering, a science dealing with the formation, transportation, deposition, measurement, analysis, treatment and control of sediment is still in its infancy. The problem bears special importance to our poor country where economy is imperative. In the Fourth Meeting of I.A.H.R at Bombay in 1951, Mr. Frield (USA) described the American experience in the State of Pennyslavania Coal Mines where 30 million tons were dredged out in the head reach. The desilting was done by dredging. The silt excluders at the head constructed by the Army Corps of Engineers cost 50 Million dollars! Cheaper measures have to be provided in less prosperous countries.

As the problem of sediment control is as old as it is important, several measures have been adopted from time to time. Before the construction of permanent structures like weirs and barrages on the rivers, the methods of sediment exclusion was quite simple. The ~~methods~~ offtaking channel was sited such that the canal took off at a suitable curvature of the river bend where the sediment intensity was low. When rivers shift and the meanders move downstream, the canal head has also to be shifted. With permanent weirs and barrages the sediment control is not so simple. The position of head regulator is fixed and the sediment entry into the canal is increased and more important. In old weirs, the canal took off from one bank of the river only where as on barrages like the Sutlej Valley Project or the Emerson Barrage the canals take off from both banks of the river.

The subject can broadly be divided into two parts:-

1. Silt Control in Canals.
2. Silt Control in distributaries or offtaking channels.

Some of the devices can be applied for canals as well as distributaries with some modifications and will be indicated likewise.

Before taking up the study of Silt Control measures it is useful to ennumerate the basic principles of their design. All devices ar-e based on the observations that:-

(i) In a flowing stream the concentration of silt in the lower layer is higher than that in upper over.

(ii) The effect of spiral flow manifests itself in the heavy silt load being kept off from the concave bank in curved flow.

(iii) The Silt is churned up due to turbulence. The effect of velocity on sediment transportation has been dealt with already.

In the following pages, the first two principles Viz: Distribution of Sediment and Effect of Curvature will be discussed in some defail.

DISTRIBUTION OF SEDIMENT

The distribution along a vertical is of importance rather than across a crossacction and this will be explained. On channels in Northern India the silt distribution on a vertical p plane is found to follow a pattern as shown in fig.1. The average concentration of silt charge by weight over the entire cross-section, if taken as 100, the conceäntration near the surface is found to be 60 to 70 percent of the average and that near to bed about 130%.

The average concentration is found near 0.6 D from the bed which is fortunately also the location for average velocity.

The finer particles held in suspension are amenable to theoeritical treatment and the concentration C at any height y is above the bed can be determined from a known concentration Ca at a reference point at height 'a' above the bed, by the following equation.

$$\frac{C}{Ca} = \left(\frac{D-y}{y} \cdot \frac{a}{D-a}\right)^{w/KV*}$$

where w is the fall velocity of a grain in still water

D is the depth of water.

K is Karmans constant (= 0.4)

V_o is shear velocity ('= $\sqrt{T_o/p}$, T_o being the intensity of s ear stress at the bottom)

Apart of this finer material tends to deposit at the sides to form the banks. Howes points out that berms consist almost entirely of material finer than 0.08 mm. while the bed for the most part consists of material coarser than 0.08 mm.

Experiments by Vogel showed that the very fine material ((loess) distributed very nearly in proportion to the water distribution between the main stream and the branch. This is as expected, as the particles suspended at higher elevations in the stream would divide in accordance with the division of water in each channel.

Therefore, the main problem which is posed before a canal engineer is that of excluding the heavier, slow moving bed material which has a tendency to deflect into the offtake.

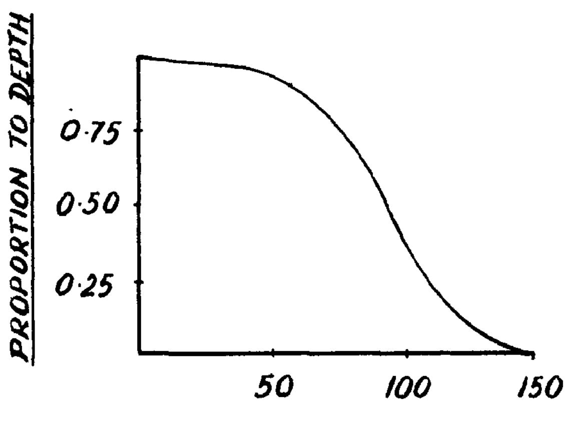


FIG. 1 DISTRIBUTION OF SILT

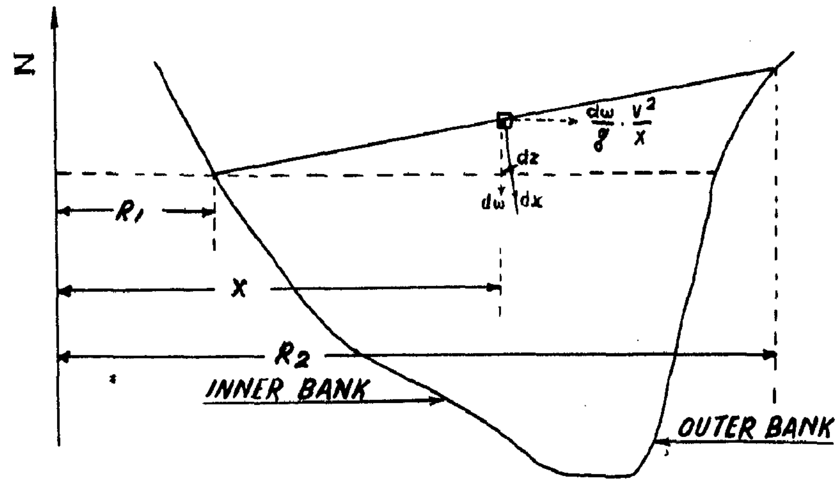


FIG. 2 EFFECT OF CURVATURE

EFFECT OF CURVATURE

Water when deflected into an offtake enters it in a curve. The flow is somewhatsimilar to flow around a bend. This resemblence was observed by Ismail, Lehavsky and others. The two apposite spirals of the straight channel flow reduce to a single spiral which is counterclockwise when the curve deviates to the right and is clockwsie in case the deviation is towards left, looking in the downstream direction. If the curve is followed by a long tangent, the spiral flow developed in the curve will per-sist for some distance downstream.

Due to the centrifugal force, the water surface at the outer bank is raised above the inner one. The amount of this superelevation can be found out considering the equilibrium of water filament under the forces acting viz., (i) due to gravity and (ii) due to centrifugal force. The centrifugal force is equal to

$$\frac{dw}{g} \cdot \frac{V^2}{x}$$

Where dw is the weight of the particle

x is the the distance of the particle from the reference axis.

g is the acceleration due to gravity.

V is the velocity of the filament on the surface at a distance. x.

The water surface assumes a position perpendicular to the resultant of these forces, as shown in the figure. From the geometry of the figure, transverse slope;

$$\frac{dZ}{dx} = \frac{dW}{g} \cdot \frac{V^2}{x} / dW = \frac{V^2}{gx}$$

This on integration gives

$$gZ = V^2 \log x + C$$

At the inner bank Z = 0 and x = , and if it is assumed that the velocity is the same everywhere in the cross section, the equation for water surface becomes ;

$$Z = \frac{V^2}{g} \log \frac{x}{r_1}$$

Hence the differnece of elevation between the outer and inner bank is ;

$$h = \frac{V^2}{g} \log. \frac{r_2}{r_1}$$

Woodward (Quoted from 2) assuming the velocity at the banks to be zero, and that at the centre a maximum value V_{max}, varying in between according to a paralic curve gave the value of superelevation as ;

$$h = \frac{V^2_{max}}{g} \left(\frac{20}{3} \frac{r_c}{b} - 16 \frac{r_c^3}{b^3} + \left(4 \frac{r_c^2}{b^2} - 1\right) \log_e \frac{2r_c + b}{2r_c - b} \right)$$

where is the radius of curvature of the stream lines

b is the width of the channel.

Chow (2) considers that the low of free vortex is applicable as long as the flow in the channel is subcritical and finds that

$$h = \frac{C^2}{2g \; r_1^2 \; r_2^2} \left(r_2^2 - r_1^2 \right) .$$

where C is a constant called circulation constant and defined by v. r = const.

The equation ( ) is most accurate of the three, the second best being equation ( ).

SILT CONTROL MEASURES IN CANALS

It is a well known and well tested maxim that "Prevention is better than cure. So, to begin at the origin:

Silt Control at the Origin:

The charecter of the drainage area and its vegetable covering are all important factors. If the rocks of the area are sedimentaries, such as sand stones, clays and shales, disintegration processes produce large quantities of fine soils. If forests exist this soil is effectively held in place against ordinary rains, but the forest rapidly loses its efficiency to prevent erosion in times of intense rainfall. If the area is too dry for forests, the soil may still be held effectively by substantial growths of grasses and small brush. If too arid for small growth, the soil lies at the mercy of every shower.

Hence silt control on the water-shed is the primary cure of the overall silt and erosion problem. It is well known that a good growth of grasses is very effective in holding the soil. Where rainfall is sufficient to maintain forests, extensive soil removal is prevented during ordinary rainfall. Of course these measure are not called for, for the express purpose of control of sediment in canals. But this is one of the many benefits of water-shed management.

Bank cutting is a fruitful sourse of sediment. The process of moving soil from the mountains to the sea consists of an infinite number of starts and stops. A bar is formed during a flood that may not be moved for many years. An alluvial valley is built up during centuries of sedimentation and then deleted under a new set of cultural or climatic conditions. Alluvial streams build their banks higher than the surrounding plains, a process which, however, cannot continue indefinitely. A flood breaks the banks and the river finds a new channel, cutting out the deposits it had itself laid down in earlier years. The Yellow River built up its channel until the water surface was 25 ft. above its plain. During the floods of 1851 to 1853 it broke its bank, inundated 50,000 sq. miles of cultivated valley, took a toll of a million lives and found a new month 500 miles to the north of its former outlet - earning for itself the name "China's Sorrow".

The most appropriate measure against this is River training. Again, as in the case of water-shed management, River training on a big scale, with guide banks, embankments, spurs and like, forms a part of overall Flood Control measures; of which silt control, though an important aspect, is a subsidiary one. Specialised River training near the headworks for the specific purpose of sediment control will be discussed in greater details in the following pages.

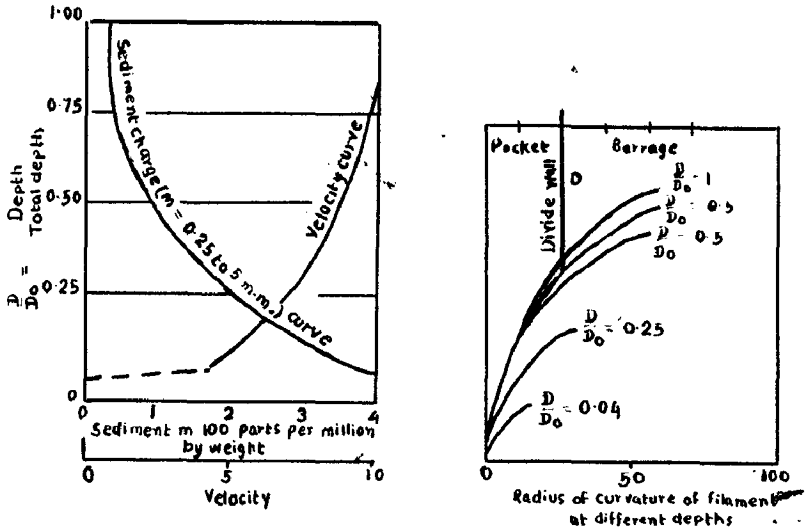
Silt Control by Reservoirs:-

R_I_V_E_R A_P_P_R_O_A_C_H

Of all the silt exclusion devices the earliest, the cheapest and the simplest is to take off the canal from outside of a curve, i.e. from the concave side of a river bend. The effect of this curvature is that the surface water tends to flow towards the outside, and the bed water, from the lower stratum of the river which contains much more heavy silt, flows over towards the inside of the curve. The most suitable position of a canal offtake is near the downstream end of a concave bend. This has been done for the Sakri Canal in Bihar, where the right bank of the Sakri river was advanced downstream to the canal offtake forming a concave bend and was pitched with stone to maintain it in that position. It is advisable to select a site with a suitable curvature and to maintain the river along that curvature.

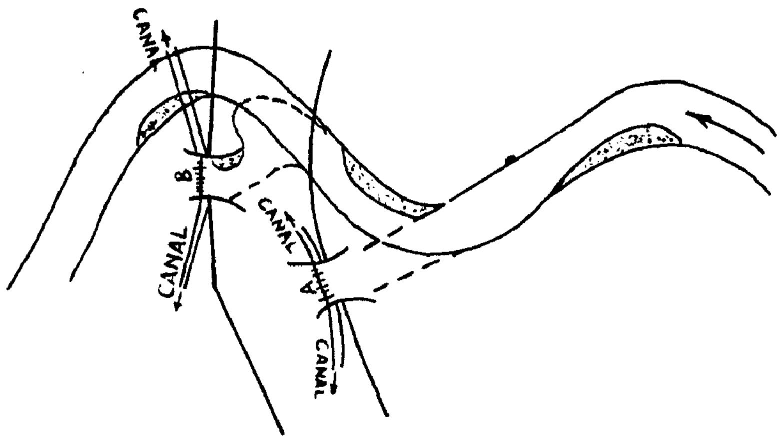
The effect of Curvature:-

Let us examine the flow of water in a curved channel, having velocity and sediment charge distribution in the approach channel as shown in Fig. 3 Since the surface currents are moving faster than bottom currents, the force required to change the direction of bottom currents is less than that of surface currents. The transverse slope set up due to the change in the direction of flow will act on each filament of flow as an unbalanced force which deflects it in such a way that the radius of curvature of each filament is approximately propartional



(a) Velocity and sediment charge distribution curves in channel.

(b) Path of filaments in a curved canal at different depths.



(c) HYPOTHETICAL CASE

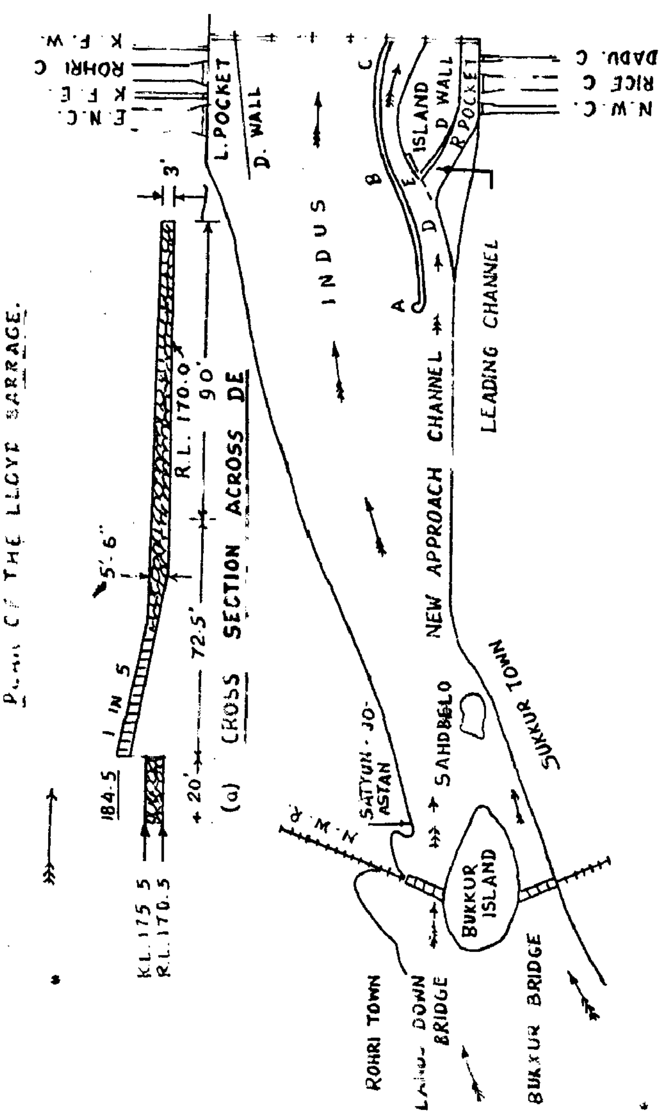
FIG. 3

to the square of the velocity. In Fig. 3 the paths of filaments at diffrent values of $\frac{d}{d_0}$ = depth/total depth are drawn, taking the factor of proportionality equal to 1. It will be seen that if a divide wall be located at D the filaments in nearly two-thirds of the depth near the bottom which contain about 80% of the sediment charge can be made to go clear of the pocket. This shows the effect of curvature.

River Training:-

The realisation of the above condition depends on the attainment of a correct approach of stream by providing training works and making the current follow the guide bank at a favourable curvature. Training works have been constructed on many rivers in the Punjab in order to secure proper curvature of the approach channel. One of the most efficient are the training works at the offtake of Sirhind Canal on the Sutlej at Rupar, where a series of armoured spars have been built along the left Bank to obtain the required curvature.

When there are canals taking off on both sides and the river has not got suitable curvature, an artificial channel can be created. An illuminous case in point is at Sukkur, Sind, Fig. 4 In this case the natural curvature is favourable for the canals on the left bank. The canals on the right bank silted heavily and an approach channel ABC was introduced with a concave curvature at the entry. From this approach channel another leading channel joins the right bank pocket so that the division of water takes place along the line DE at the main approach channel just before



LLOYD BARRAGE

the reverse curvature of the main approach channel is about to start. The banks dividing the natural river section to form the approach and leading channels are made of dredged earth protected by stone on the slopes and in the apron. The stone is loosely dumped below water level and hand picked above.

The usual practice in some parts of Sind is to construct the barrage outside the main river channel in some minor creek, which is dry in winter and then to divert the main river through the barrage. It has to be noted that to a great extent location of the weir with respect of the river loop is responsible for the approach condition for a number of years to come and determines the future need of training works to rectify the approach. Consider a hypothetical location of the barrage at A & B in Fig. 3. After diversion, the probable course of the river for some years before any further training works are constructed will be as shown in this figure. With the barrage located at B an oblique approach to the nose of the right guide bank will be obtained almost at the start. For the location at A, to start with a straight approach can be had easily; the obliquity of approach will occur when the upstream loop travels by progressive caving of the outer bank to the guide bank head. This may not occur for many years, but when it does the guide bank head works as a spur Experiments have shown that invariably a scour at the nose and shoal formation below the spur and guide bank head will result in such a case. The approach conditions for some of the weirs, Fig. 5, show clearly that for a straight approach the island may form any where between the guide banks, is but with an oblique approach the island invariably forms along the attacked guide bank. In these weirs the clear waterway between the guide bank, is kept to pass the maximum flood, which occurs once in a decade with least afflux. The dominant discharge is also very much less than the maximum flood and therefore shoals and islands form within the guide banks without any exception; even the peak flood lasts for a few days and is generally unable to completely wash out the well consolidated islands.

Slope:-

The slope in the river channel approaching the intake is also of considerable importance. As soon as the slope increases, the sediment movement in the river channel considerably increases. A rising water increases sediment movement and intensity, firstly by scouring down the bed to increase the cross-section for the increased dishharge and also by increasing the river slope, particularly in a flood. It was found in the Chenab Canal in Punjab, that a river slope in excess of about 1 in 4000 brough in heavy sediment load into the canal. An attempt was therefore made to keep the slope between 1 in 4000 and 1 in 5000. This was attained by closing the canal and flushing the approach channel, if necessary, or by simply raising the water level in the river at the canal offtake.

Another method adopted at some of the Headworks, notably ~~at~~ Khanki and Rasul was to divide the river approach channel into two channels, thereby considerably reducing the discharge approaching the canal offtake and thus reducing the sediment movement.

With the increase of dicharge in the approach channel the sediment intensity in the channel increases very considerably. It has been observed that increase in discharge. Lane observes that the sediment intensity is increased roughly about the square of the increase in discharge i.e. if the dishharge is increased three times, the sediment intensity increases nine times. Therefore, discharge in the approach channel should be kept to the minimum required for feeding the canal.

The approach channel should not be allowed to silt up beyond a certain level. At Khanki and Merala Headworks soundings of the approach channel were taken daily and when sediment deposit reached about 7 ft. the channel was flushed. A very deep approach channel is also not desirable. With such a channel the discharge drawn to the pocket would also be great and at Headworks like Madhopur and Tajewala where a still pond system is not practicable the sediment entry into the canal will be very heavy. At Madhopur, when the approach channel became deeper the sediment entry into the canal increased considerably specially the gravel entry.

At certain headworks canals are fed by the water flowing parallel to the weir and then entering the pocket round the divide wall since a still pond system of regulation is adopted in the pocket; most of the sediment settles down in front of the weir before the water reaches the pocket.

SILT CONTROL AT HEADWORKS

The design of the canal headworks should be such that it caters for minimum silt entry into the canal. The following parts of the design effect sediment conditions entering a canal:-

1. Type of Headworks.
2. Effect of alignment of the head regulator.
3. Effect of sill level of the regulator.
4. Effect of shape of guide banks.
5. Effect of length and shape of divide wall and pocket.
6. Effect of distance of sluices.

Type of Headworks:-

Both from experiments and observations on the prototype it is shown that narrow and deep weirs induce more favourable coditions at the head regulator from the point of view of sediment control. In wide weirs formation of islands and their development and erasion, frequently cause unfavourable conditions, which can be obtiated to a great extent in narrow and deep weirs.

Allignment of the Head Regulator:-

The canal regulator should be so aligned that a suitable curvature of flow is induced in front of the regulator, in the pocket. The upstream end of the regulator slightly projecting into the river gives good results as it pushes the bottom current away from the regulator. The regulator should not be set back and should preferably be at right angles to the barrage. A study of the designs of the existing headworks in the Punjab show that to achieve the same object diffrent designs have been adopted, some contrary to the other. While generally a splay of 1 in 4 is given for setting back the regulator from the line at right angles to the barrage, at Rupar this is given in the reverse direction, yet at Trimmu and Kalabagh Headworks which are the latest in the Punjab, no splay is given, the head regulators being at right angles to the barrage.

The following table shows the angles and splay of the diffrent canal regulators:-

TABLE 2

HEADWORKS	Angle of regulator with respect to weir line.		Angle of regulator with respect to upstream.	
	Angle	Splay	Angle	Splay
Suleimanki headworks				
Left Pocket	14.0	1 : 4.2	15.5	
Right Pocket	14.0	1 : 4.2	12.5	
Ferozepur headworks				
Left Pocket	14.0	1 : 4.0	15.0	1 : 4.0
Right Pocket	14.0	1 : 4.2	14.0	1 : 4.2
Islam headworks				
Left Pocket	14.5	1 : 4.8	16.5	1 : 3.5
Right Pocket	14.5	1 : 4.0	61.0	1 : 3.5
Panjnad headworks				
Left Pocket	13.5	1 : 3.8	15.5	1 : 3.8
Right Pocket				
All American Canal	21.0	1 : 2.5		
Nangal Hydel Canal	12.25	1 : 4.0		1 : 3.0
Rasul Headworks	13.5	1 : 5.0	35.0	1 : 1.4
Khanki Headworks	15.0	1 : 3.8		
Rupar Headworks	15.0	in the reverse direction		

No splay is there in the case of Trimmu, Kalabagh, Merala and Madhopur Headworks. Generally a splay of 10 to 12° setback induces a suitable curvature of flow in the pocket. This feature of the design, however, has to be combined with other elements of the design of the headworks.

Sill Level of the regulator:-

It is usual to keep the sill level of the canal regulator at a sufficient height above the floor or the crest of the undersluices of the weir. In fact on many headworks the sill level had to be raised subsequent to the original construction of the structure, with a view to reduce sediment entry into the canal. However, experience has shown that High Sill level alone cannot prevent sediment entry into the canal. This raising, however, gives a sufficient depth of water in front so that silting in the pocket can be watched and steps taken to improve matters before this sediment starts flowin g into the canal. On main canals which have large discharge capacity the high sill level is unable to neutralise the unfavourable curvature of flow in the river nor can it prevent the pocket from silting up.

Effect of High sill level:- A little thought will show that with the still pond system of regulation the sediment that once enters the pocket will either deposit in the pocket or will be drawn into the canal. When water is drawn into an offtake from a pool, there is always a region above the regulator in which higher velocities are developed than the normal velocity, if any, in the pool. The velocities decrease as we recede from the regulator till they become equal to the normal velocity of the pool. This reach can be termed as the suction reach of the regulator. From the point of view of sediment withdrawal the active suction reach will be that in which sand particles to be excluded cannot deposit but are sucked into the offtake. With the still - pond system, unless the coarser particles deposit out of the active suction reach of the regulator, they will pass into the canal. Again, for a given crest height with semi open flow more sediment is likely to come within the suction reach of the canal and therefore a greater amount of sediment will be passing into the canal in this case then in the still pond system. Increase in the crest height will decrease the suction reach of the canal to some extent; but it appears that a propertional decrease in extent of the suction reach is not attained by increasing the height of the crest. Experiments, carried out in the Hydraulic laboratory of the Punjab Irrigation Research Institute, Lahore, with still pond conditions, show that by increasing the height of the crest, sediment entry into the canal diminishes at a decreasing rate. Fig. 6 below shows the result of tests.

The experiments show that the regulator crest should be higher than the underslince crest level or the bed level of the pocket, but raising the crest of the regulator can by no means be prescribed as an effective curve for excluding the sediment from the canal.

Whatever may be the height of the sill the floor of the pocket must silt up eventually to the sill level unless expedients are adopted to check this. This is proved by the fact that in spite of raising the sill level on certain headworks the trouble has not diminished. For the time bein-g it might lessen but when the void is filled the bed adjusts itself to the new level of the sill and then more or less similar conditions develop. Therefore the high sill level of the regulator alone is of no material help in preventing the sediment entry into the canal.

The height of the regulator sill above the floor of the pocket and the crest of the undercluices or the weir is generally of the order of 7 to 8 feet as given in the following table:-

TABLE - 3.

Sl. No.	HEADWORKS	R.L.of Pocket Floor	R.L.of Upstream Crest	R.L.of Permanent Sill	Max.of working Sill.	HEIGHT OF THE SILL
1.	Madhopur Headworks	1129.5	1129.5	1136.5	1136.5	7.0
2.	Merala "	729.0	792.0	795.23	804.0	12.0
3.	Rupar "	857.0	857.0	866.0	.871.0	14.0
4.	Khanki "	714.0		726.0	726.0	12.0
5.	Panjnad "					
	Main Line Canal	320.0	325.0	332.5	332.5	12.5
	Abbasia Canal	320.0	325.0	330.0	330.0	10.0
6.	Tajewala Headworks	1052.5		1058.0		5.5
7.	Islam "					
	Mailsi Canal	441.33		445.0		3.67
	Bohawalpur Canal	441.33		445.0		3.67
	Quaimpur Canal	441.33		446.0		4.67
8.	Ferozepur Headworks					
	Dipalpur Canal	630.5		638.0		7.5
	Eastern Canal	630.5				7.5
9.	Nangal Dam	1100.5	1121.0	1121.0		21.6

At some of the old canal Headworks like those of Sirhind Canal and Upper Bari Doab Canal the regulator sill was raised later. It helped to reduce the quantity of gravel and boulder into the canal in the first few years after remodelling. Later on the sediment entry again became excessive and measures for its exclusion had to be sought for.

Cantilever Platforms:-

Construction of cantilever platforms in front of the head regulator in the canal pocket is considered to be one of the measures for excluding sediment from entering the canal. A cantilever platform was constructed in front of the western Jamuna canal head regulator at Tajewala. The platform cavered all the 12 bays of the regulator in level with the crest.

Experience shows that cantilever platforms at large canal headworks are of no use at all. The platform gets itself buried in the deposit of sand and gravel in front of the regulator and at certain times induces greater flow of gravel and sand into the canal than would be the case in its absence. The fact is illustrated from the case of western Jamuna canal head regulator at Tajewala where a large number of observations were made to study the effect of construction of the platform. It was observed that:-

(a) A very high deposit occured in front of the cantilever platform and in certain cases the level of the silted bed was higher than the platform.

(b) The average depth of deposit over the floor of the pocket was of the order of 8 to 10 feet.

It can thus be concluded that the cantilever platform is of no advantage whatsoever, when in its presence such high deposits can take place and especially over the platform itself.

Shape of Guide Banks:-

The shape of the guide banks upstream of the head regulator helps to some extent to secure a suitable approach to the canal pocket. The shape in each case is governed by the existing river approach upstream of the headworks. Several shapes of guide banks have been investigated in the pPunjab. These are: (i) parallel guide banks (ii) Converging guide banks (iii) diverging guide banks (iv) bottle neck guide banks etc. The bottle neck guide banks adopted at the Sulemanki S.V.P. on the Sutlej has not proved to be of much sucess as large islands have always formed upstream of the canal pockets. While designin-g Kalabagh Headworks, Thal Project, on the Indus, the diffrent model tests made indicated the superiority of slightly diverging type of guide banks. With the help of this type the main river flow upstream of the pocket was situated at some distance from the pocket and therefore the main silt front moved away from the pocket to the weir. At most of the places the influence of guide banks is masked by the large Islands that have grown in the river upstream.

Length and shape of Divide wall and pocket:-

On almost all headworks in India, a Divide wall is constructed at the end of the undersluices running upstream from the barrage or weir, generally parallel to the head regulator, to seperate the undersluices from the main weir structure. The idea is to produce a still pond pocket in which silt would deposit instead of going into the canal. The silt in the pocket is subsequently scoured out through the undersluices. While the pocket is being flushed the canal has to be closed, which interferes with irrigation. The length of the divide wall and its distance from the canal offtake are very important.

Experiments conducted at the Malikpur Hydraulic Station, in connection with the design of the Kalabagh Headworks, Thal Project, with different length of the divide walls at diffrent positions were examined for their effect on the sediment entry into the canal for diffrent river conditions. The main conclusions obtained were:-

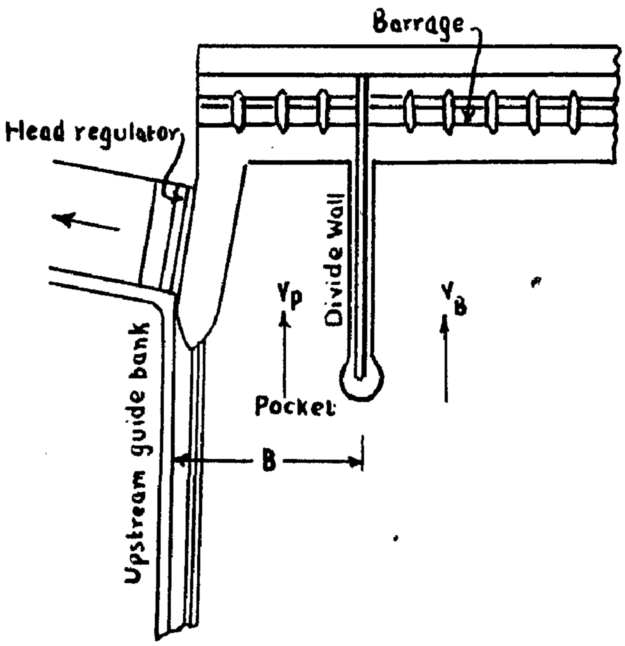
(i) The shorter the length of the divide wall the smaller was the quantity of gravel and sand entering the canal

(ii) The greater the distance of the divide wall from the regulator the smaller was the quantity of gravel and sand entering the canal.

It was also concluded that if the whole of a weir or barrage were at the same level, a divide wall may not be necessary. Where the undersluice floor is at a lower level, a divide wall becomes necessary to seperate them from the higher weir level. In such case it was found that a divide wall of length equal to two-thirds of the length of canal regulator is most suitable for sediment control as this length helps to form a suitable curvature of flow.

Effect of divide wall:-

The main function of the divide wall is to seperate the flow on its two sides i.e. in the main weir and the pocket, and thereby to afford more consistent approach to the head regulator under diffrent working conditions. Inglis, in his experiments in 1933, probably for the first time, showed that for a straight approach when the current bifurcates at a divide wall the distribution of sediment takes place according to the ratio of velocities on the two sides of the divide wall. If V_p be the velocity in the pocket and V_B the approach velocity to the barrage Figure , then the sediment entry into the pocket can be decreased by increasing the ratio V_B/V_P. This will be maximum for a given V_B when V_P is minimum. This can be realised in this case only when the undersluice gates are completely shut and the pocket is only drawing enough supply to feed the canals. The physical explanation of the above phenomenon lies in the fact that the sediment movement is known to vary as the 6th power of the stream velocity, or for coarser



EFFECT OF DIVIDE WALL
FIG. 5

material, as found by Schoklisch, the sediment discharge per unit width is proportional to (slope)$^{2/3}$. When V_P is made less than V_B by closing the gates of the undersluices, the slope in the pocket will become flatter than that on the barrage side, and the greater partion of the sediment charge will go to the barrage, whichever law of sediment motion is assumed. Thus V_R/V_P ratio can be taken as an approximate criterion for sand exclusion. If V_R/V_P ratio is greater than unity, bed lines are sharply bent around the divide wall and material is diverted into the river. If V_R/V_P is unity, bed charge is proportionately divided between the pocket and the river, and for V_R/V_P 1 bed material, if in movement, would be heavily attracted into the pocket.

Bed movement in a channel depends upon the tractive force and grade of bed material, In a river when discharge is low and comparitively little bed movement occurs, no trouble would be experienced even if canal draw-off was large and V_R/V_P 1.

With increase in discharge, there is a corrosponding rise in tractive force and for a certain discharge - depending upon the slope, depth and grade of material - appreciable bed movement begins. If at this stage, considerable bed material is moving along the bed and V_R/V_P 1, sand exclusion will be well-nigh impossible. For still higher discharges, the ratio V_R/V_P increases with consequent improvemen t in exclusion, even though there might be an increase in bed charge. A barrage or weir should be designed to give satisfactory exclusion for river discharges when considerable bed movement occurs and V_R/V_P is likely to be unfavourable.

A word of warning is that a high V_R/V_P ratio only is not a sufficient guarantee for trouble free operation of canals. It should be noted that in the Sukkur barrage as originally designed, V_R/V_P for the left and right pockets, for river discharge 250,000 cusecs, was 1.23 and 1.72 respectively. The left bank canals being on the outside of a curve functioned satisfactorily while the right bank canals silted up to the extent of 5 ft. in one season, due to unfavourable curvature.

The width of a pocket is decided by the maximum discharge of the canals and should be wide enough to ensure a smooth flow and V_R/V_P ratio greater than unity at low river discharges when appreciable bed material is in movement.

A study of the headworks of the Punjab canals shows that on diffrent works diffrent lengths of the divide walls have been adopted. At certain places very long walls for exceedin g the length of the regulator have been constructed, as shown in table:- ~~below:-~~

TABLE-4

Sl. No.	HEADWORKS	Length of Canal Regulator FEET	Length of the Divide wall FEET
1.	Khanki Headworks Lower Chenab Canal	650	550
2.	Merala Headworks Upper Chenab Canal	350	450
3.	Ferozepur Headworks Bikaner Eastern	210	710
4.	Ferozepur Headworks Dipalpur	260	730
5.	Emerson Barrage Rangpur Canal		620
	Haveli Main Canal		531 & 1000
6.	Punjnad Headworks Abbasia Panjnad	400	800
7.	Rasul Headworks Lower Jhelum Canal	224	600
8.	Rupar Headworks Sirhind Canal	321	760
9.	Islam Headworks Quampur Canal	190	650
10.	Islam Headworks Mailsi Canal	170	650
11.	Sulemanki Headworks Sadqui Canal	290	680
12.	Sulemanki Headworks Pakpattan Canal	195	680

No divide wall exists at Tajewala, Madhopur and Nangal Dam. The length of the divide wall adopted in the past in the Punjab is many times the length of the Canal regulator except at Khanki or Merala. It has no relation to the length of the regulator. Even small lengths of the regulator at certain works have long divide walls. Experience has shown that there is no merit in an unduly long divide wall which helps exclusion in no way. Investigations carried out in connection with the Thal Project showed the following effect of divide wall length on exclusion: Silt entering left pocket with a 600 ft. long divide wall for a river discharge equivalent to 200,000 cusecs was six times as much with 300 ft.long divide wall. A pocket of 7 spans width proved much superior to a pocket of 4 spans width, the amount of silt entering wider pocket being ½ as much with narrow pocket; this bein g due to increased velocities in the pocket.

The further the nose of the divide wall is pushed upstream of the weir, the less is the effect of regulation on curvature of flow, which helps so greatly in getting efficient exclusion, particularly at low river discharge stages. It intercepts the sand front, which would in its normal course move away from the regulator to the weir, and pushes it towards the regulator and into the pocket. A point advocated in favour of a long divide wall that it served as a trap for coarser bed material, is unsound; and was clearly shown by the right pocket of the Sukkur Barrage. of course a long divide wall forms a large pocket but only to fill it up with the increased sediment load which it brings to the pocket. Another objectionable feature of a long pocket is that it takes much longer to scour away the silt, and the period of canal closure being limited, the advantages of a short pocket are obvious.

The length of the divide wall should not exceed the length of the canal regulator. It is preferable from the point of view of sediment control to keep the length of the divide wall equal to 2/3 the length of the regulator as has been shown in the several experiments carried out in the past as well as recently in connection with Nangal Dam. In the case of more than one canal, the divide wall shuuld be taken up to the last regulator. Extent of scour around the nose of divide wall should also be considered, as bed material thrown in suspension due to intense terbulence is likely to be drawn into the first canal. It should be, therefore, seen that the effect of scour at the nose does not reach as far as to effect exclusion.

Design of the nose of divide wall influences exclusion to some extent. The steeper the slope of nose the greater is the depth and extent of scour hole around it. Exclusion is better with scour hole round the nose of divide wall, if V_R/V_P ratio is favourable, due to bed material flowing around it and into the river due to curvature. Flat slopes tend to reduce the scour depth; this no doubt, makes the design safer, but the effect on exclusion is lost to some extent. Advantage of this fact can be taken by giving the nose a flat slope on the pocket side and a steep slope on the river side, with the result that exclusion would be materially improved due to greater pulling action on the river side consequent to deeper scour than the pocket side.

A pocket slightly converging towards the undersluices is to be preferred to a straight pocket as this helps scouring operation; but the convergence should not exceed 1 in 10 as this makes the barrage spans adjacent to the divide wall partly ineffective. The effect of splay sharper than 1 in 10, on exclusion, is adverse and with increase in convergence, the of slack flow is increased resulting in a heavy deposit of material in the pocket. This was observed in model experiments done for Lower Sind Barrage.

Tests conducted at the Denver Hydraulic Laboratories of the Bureau of Reclamation, U.S., also showed that a curved divide wall was even more efficient than a straight one in diverting coarse sediment away from the canal headworks. As a result of model tests two curved walls of steel sheet piling were constructed upstream of the Superior-Courtland diversion dam on Republican River, 50 ft. and 100 ft. in length respectively. By the installation of these minor structures, water carrying about 2/3 of the sediment is twisted away from headworks and is carried away into the river, while relatively sediment-free portion of the flow is channelled into the canal.

Position of Undersluices:-

Sluices have been considered necessary next to the canal offtake, to maintain a well defined channel in front of it to feed the canal. When it is decided to construct more than one undersluice in the weir, the most suitable position for locating the undersluices has to be determined, both from the point of view of sediment control as well as passage of flood. From the point of view of sediment control at a canal headworks where a still pond system is adopted, a suitable position will be one in the centre of the weir or slightly more towards the canal pocket. This position of the undersluices would help to form a suitable curvature of flow upstream of the pocket. This curvature would push the main load of sediment on to the second set of undersluices. The position of the undersluices should be such that they work in coordination with the pocket. In order to induce this curvature of flow the central undersluices were specially constructed, for the first time, at the Khanki Headworks on Chenab, as the right under sluices could not work in conjunction with the pocket for sediment control. For the passage of floods these may be sufficient. While no definite conclusion could be drawn from the working of these sluices, it seems that they are a bit too far away from the canal offtake to be of much practical use. On a barrage, any part of it could be used for creating the curvature of flow provided it is safe from other considerations. This method needs further investigations before it could be adopted for general use. During investigations for new barrages it has been found that if a second set of undersluices is to be provided it should be in or near about the centre if the approach to the pocket is from upstream along the guide bank. Another point which arises in this connection is whether or not one level of the barrage with no special undersluices would be better or depressed sluices in the barrage would be more suitable, The provision of depressed sluices helps to maintain a well defined channel in front of them and flushing can be easily secured. It is considered that suitably located depressed undersluices induce minimum sand entry conditions into the canal. This point, however, requires further detailed investigation which is being done with the help of large scale models.

R_I_V_E_R R_E_G_U_L_A_T_I_O_N

The method of regulation at the weir influiences considerably the sediment entry into the canal or power channels. The method is of special importance to barrages, weirs or hydraulic structures at which the channels take off from only one side. The measure which has been found successful and is most commonly adopted for river regulation in the Punjab and elsewhere in India, is what has come to be known as the 'still pond' system.

Even as late as the thirtees there had been two main schools of thought for excluding heavy bed silt from offtakes. The older school advocated semi-open flow e.g. the partial opening of a few undersluice gates near the bed at the side of the approach channel furthest from the canal offtake, near the divide wall with the idea of drawing bed silt away from the canal face. The other school advocates still pond running. The main feature of this system is that the waterlevel in the river at the canal offtake is kept at a steady level and no water is escaped through the sluices as long as the canal is open.

Still Pond System:-

In this system of regulation the undersluices have to be closed as long as the canal is flowing, and this is done till the sediment charge in the river becomes excessive or when the river discharge exceeds that which can be passed by the main barrage alone at the authorised land. Now, with a still pond system the velocity V_P in the pocket has to be kept below the critical velocity that can move the sediment of the grade which is considered necessary to be excluded from the canals, generally 0.2 m.m. and above. Assuming a still pond system with the pocket drawing only the discharge equal to the canal discharge the velocity V_P is calculated for a barrage not equipped or proposed to be equipped with silt excluders. The critical velocities calculated from Lacey's formula $V_o = 1.15 \sqrt{fR}$, where $f = 1.76 \sqrt{m}$ are tabulated in column 10 of Table V.

Sl No	Name of HEADWORKS	Name of HEAD REGULATOR	Canal discharge in cusecs	Width of under sluices feet	Pond level R.L	Upstream fl. level R.L	Depth of water feet	$V_P = \frac{Q}{BD}$	$V_o = 1.15 fR$	$\frac{V_o}{V_P}$
1.	Islam	Mailsi	4883	261	452	438.25	13.75	1.36	3.53	2.596
2.	Panjnad	Panjnad Abbasia	10,599	261	337.5	320	17.5	2.32	3.99	1.72
3.	Sulemanki	Sadiquia Fordwah	8227	275	569	552	17	1.76	3.89	2.21
		Pakpattam	6058	275	569	552	17	1.29	3.89	3.015
4.	Kotri	Fulleli Pinyari	37252	395	68	35	33	2.86	5.31	1.857
		Kalri	8458	261	68	35	33	0.98	5.13	5.235
5.	Rasul		4093	295	711.5	701	10.5	1.31	3.13	2.389
6.	Merala		13528	318	808	792	15.9	2.67	3.8	1.423

In all the cases the ratio V_o/V_P is greater than 1 showing that the conditions will be favourable for sediment deposition. The low velocity in the pocket will continue to deposit sediment and raise the bed till the velocity in the pocket again increases to such an extent that the sediment starts entering the canal again At such times known from the daily soundings taken in the flood season, sluicing operations - when the canal is closed and the deposited sediment is washed down by opening the undersluice gates - become necessary. The frequent sluicing operations required in the flood season and the loss due to closure of the canal is in fact the main objection against this system is ~~in fact the main objection against this system~~ of regulation.

Basic Principle:

As seen above, if the high discharge is diverted away from the canal regulator, the heavy sediment charge will also be pushed away from it. On this principle, escape of discharge at the end of the weir furthest from the canal regulator would be helpful in bringing about favourable conditions for sediment in the pocket. In fact the ideal condition would be to have as much discharge as is required for the canal in the pocket but this cannot be always secured. The regulation then from the point of sediment works out as follows:-

(i) Start escaping surplus discharge in the river through the bays at the extreme right end and then as the discharge increases open bays of the river towards the canal side. Of the weir bays, the one closest to the pocket to be opened last of all. If it cannot be helped it may be opended but it is preferable to keep it closed.

(ii) At the undersluices no escape is to take place so long as the canal is running i.e. a still pond system will have to be maintained. If it becomes absolutely necessary to open undersluices, those bays away from the regulator may be opened first.

This regulation is further facilitated and its effect enhanced on sediment control in the pocket where the weir is divided into two parts by the growth of large belas as at Khanki and Rasul Headworks. At Rupar Headworks also it was found that a still pond system worked very efficiently and the Sirhind canal silted up whenever a departure was made from this procedure.

Semi-open Flow System:-

In this system of regulation the undersluice gates are kept partially open while the canal is in flow. Obviously, when open or semi-open flow is adopted the velocity in the pocket will be more than that in the case of the still pond system, with the result that V_B/V_P will be comparitively smaller and therefore more sediment will be entering the pocket. Since in this case the velocity is very likely to exceed the limit for the deposition of sediment it will not deposit in the pocket but move on to take its chance through the canal head regulator or the undersluices. The results of experiments carried out on the distribution of sediment at bifurcation by various workers show that the offtaking channel draws according to one authority as much as 90 percent of total bed load and according to another about 66 percent of total bed load with about 37 percent extraction ratio. Whatever be the exact percentage it appears that it would draw more than its normal share of sediment charge. This is due to the fact that the top layers having more velocity, but less sediment charge than bottom layers, tend to continue in their path to the undersluices, and slow moving, easily deflectable, heavily charged bottom layers. For the open or semi-open flow system of regulation it has been shown that greater sediment charge enters the pocket at the first bifurcation at divide wall nose; now, if at the second bifurcation at the regulator the offtaking canal draws more than its share of sediment charge this system of regulation should obviously be less efficient than the still pond system. A point in illustration is the working of semi-open flow in Lower Chenab Canal. Up to the end of September, 1929 when the still pond system of regulation was being followed, there was only 0.2 feet of silt in the first mile of the canal. In 1930 on account of extensive repairs in hand below the weir, several freshets had to be passed through the undersluices with canal open. The results of soundings in May and June, 1930 showed a silting of 4 to 8 ft. in the first mile below the head regulator.

Still Pond Vs Open Flow:-

There has been much controversy in the past as to which of these two methods is more effective for excluding coarses bed material from the canals. The problem has been thoroughly investigated with the aid of models and the principles underlying each method area now better appreciated. As explained earlier, the pysical concepts of factors affecting sediment exclusion and the actual practice clearly show the superiority of the still pond system of regulation over the open or semi-open flow.

On rivers, where it is not practicable to adopt the still pond system of regulation, such as the River Ravi at Madhopur where the Upper Bari Doab Canal takes off and on the Yemuna at Tajewala where the western Yemuna and Eastern Yemuna canals take off, and the sluices have necessarily to be used for purposes of regulation, regulation of supplies to reduce sediment entry into the canal becomes an important problem. The most suitable method in such cases, from the point of view of sediment control in the canal, is to open first those sluices which are far removed from the offtake and increase the opening towards the canal only when the first ones are fully open.

At Merala Headworks a large central island exists which does not extend to the weir leaving a gap of 500--600 ft. For the passage of the surplus discharge during the monsoon season the method of regulation adopted in this case is the same as that used at Khanki or Rasul Headworks. In this case, the canal can be fed ~~f~~ from any part of the weir even from the extreme right hand side. Precautions should be taken out not to overdevelop the left channel leading to the pocket. As the winter supply has also to take place through this channel, it has great tendency to overdevelop. In spite of best efforts mode, it was not possible to reduce the discharging capacity of the left channel at Merala with the result that heavy sediment load continued to fill the pocket.

Canal Closures:-

Another common measure adopted almost all over the world is the closing of the canal when the river stage is such that the water is heavily laden with sediment. The canal may also be closed when the river bed level in front of the canal offtake is so high that the sediment is likely to be carried into the canal. The exact time of closing and the period for which it should last can be judged by experience at each river and actual observation of suspended silt in the river and canal. Such observation should be made at least once a day for use in river regulation, This method is mostly used for irrigation channels as power channels cannot be closed without shutting off the electric supply. The closure of the canal may be divided into two parts:

(i) On account of floods in the river:- In floods when the sediment charge is very high the canals are closed at headworks, to avoid heavy sediment entry into the canals. The river discharge at which the sediment entry gets very excessive is diffrent at different headworks; and mostly depends on the nature of the river, headworks situation, design of headworks and approach conditions where headworks are located in sub-mountain reaches the limiting discharge is as low as 30,000 cusec, while in the plains it may be as high as 150,000 cusec.

Such flood clasures last only for the time the river remains high. During such closures regulation is so done that the pocket and approach is also flushed at the same time. Interest of Irrigation and Power demands that there should be no canal closure unless the indent for water is nil. But in actual practice the canal has to be closed very frequently and the period of closure may be as large as 15--20 days or sometimes more. Closures of the Upper Bari Doab and Western Yamuna Canals on account of floods for diffrent years is given in Table .

TABLE-VI.

YEAR	UPPER BARI DOAB CANAL Duration of Canal Hours	YEAR	WESTERN YAMUNA CANAL Duration of Cnals Hours
1932	227	1941	59
33	166	42	446
34	153	43	249
35	223	44	35
36	134	45	60
37	67	46	82
38	116	47	148
39	179	48	448
40	78	49	275
41	81		
42	497		
43	302		

The floods lower the ramp but it reforms quickly after reopening of the canal.

(ii) Sluicing Closures:- Sluicin-g closures are carried out to flush out the approach channel and the pocket. The sluicing closure may be divided into

(a) Normal Sluicin-g Closures:- This is done before the river starts rising in the monsoon season i.e. in May or June, so as to wash the sediment and deepen the channel and carry the extra water without excessive sediment movement when the river rises. The canal is closed for a short period and the undersluice gates raised to open out the approach channel and flush the pocket

(b) Special Sluicing Closures:- Closures also become necessary during the monsoon when there are frequent fluctuations in the river discharge due to rains and the approach channel shows signs of silting. The higher the ramp level, the greater is the sediment entry into the canal. If formation of ramp can be prevented no grand or coarse sediment can enter the canal. The crest of the regulator is generally 8--10 ft. higher than the floor level. Even if the top of the ramp is maintained 3 ft. lower than the crest level, no gravel or boulder would jump over the crest and enter the canal. In order to Lower the height of the ramp below the crest of the regulator, at certain headworks like Madhopur, the canal is closed for a short period say 15 minutes to a couple of hours and the undersluice gates opened to wash out the ramp. This helps in lowering the ramp level and reducing the sediment entering the canal.

During the sluicing closure very high velocities are generated and the whole bed in the pocket is churned up. Large surface ripples form and the river bed is scoured out. Table shows average ramp before and after sluicing closures for diffrent periods:-

TABLE VII.

Period in Minutes	Average ramp before closure in Feet.	Average ramp after closure in Feet.
15	3.91	5.80
20	4.46	5.98
30	4.66	5.89
45	4.41	4.90
60	3.39	5.05
130	3.60	5.16
180	3.34	4.47
360	3.50	3.60

It is concluded that:

1. A sluicing closure for 15--30 minutes is more useful for clearing the pocket then a longer closure of 2 hours or more. The period should not exceed 2 hours in any case.

2. The closure done in low river discharge is not very effective. Even a longer sluicin-g closure in low river is not of much use.

3. The sluicing closure done in high floods is not very useful. Even if the ramp is washed, it reforms immediately when the canal is reopened after the closure.

4. The sluicing generally affects 3/4 the length of the regulator. However, it depends upon the approach conditions.

5. Partial closures is not as useful as full closure.

SEDIMENT EXCLUDERS AND EJECTORS

Not withstanding all the measures mentioned for sediment exclusion, very large quantities of sediment load enter the canal at the head. Very often, even in comparitively low discharges in the pocket, sediment load is very high. Special arrangements called Sediment Excluders and Ejectors are structures for reducing the sediment entering a channel. Excluders which are located upstream of the canal head prevent the sediment from entering the canal; while Ejectors, also called Extractors or Eliminators, are usually located down stream from head regulator on the canal and eject the sediment after it has entered the canal.

A great variety of devices have been used for Excluders and Ejectors. This subject has been studied most extensively in India, but some work has been done in other countries also. The importance of this subject and the awareness of Indian Engineers can be judged by the fact that this subject of Excluders and Ejectors is under discussion at the Annual Meetings of the Research Committee and the Central Board of Irrigation and Power, India, since 1938.

It is necessory to have a clear perspective of the conditions obtaining in channels in connection with silt before a decision for silt prevention devices is taken. It should be remembered that the silt excluded from a channel has to be carried by another channel whether it be the parent or a sister channel. If a parent channel is divided into a number of small channels the total charge that these channels can carry will be less than what the parent can carry. This is because the capacity to sustain a charge, which is weight of silt per cubic foot of water increases with the disharge. All devices should take advantage of the basic facts that:-

(i) In a flowing channel the concentration of silt charge as well as grade of silt in the lower layers is higher than that in the upper ones. Heavy sand is carried as bed load

(ii) The silt is churned up on account of roughness of bed or any obstruction across it

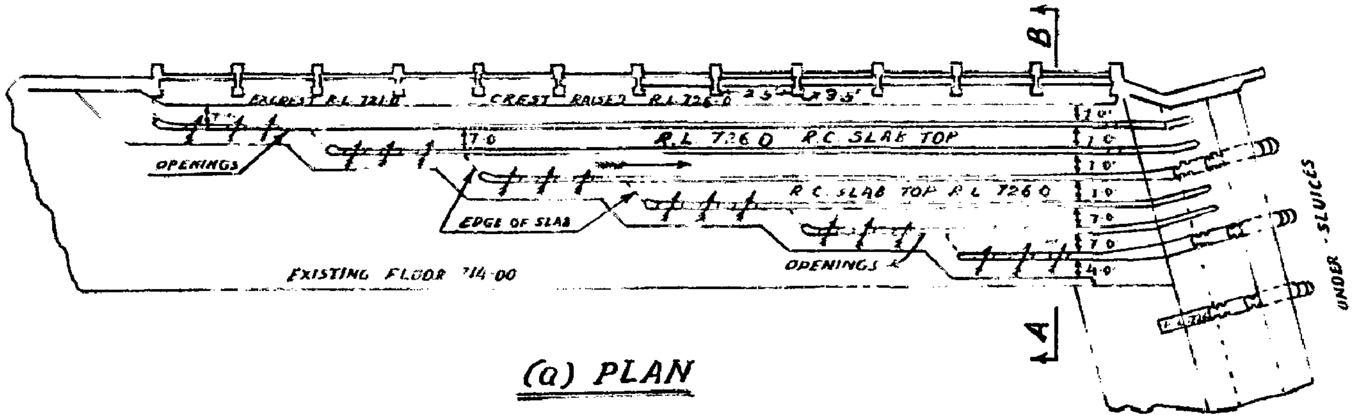
(iii) The higher the velocity, the more is the bed load thrown up.

SEDIMENT EXCLUDERS

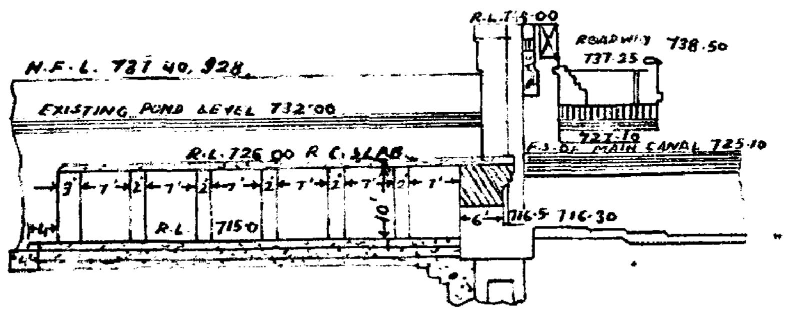
Tunnel Type:

Mr.F.V.Elsden was responsible for originating the idea of tunnel type excluders in 1922. The first excluder was built in 1934 by Mr.H.W.Nicholson at Khanki at the head of lower Chenab Canal. Many more have since been constructed mostly in Punjab. The principle involved is to pass the heavily laden slow moving hed water through the tunnels and the undersluices to the downstream, leaving clearer top water to be drawn by the canals. This device consists of a slab usually of R.C.C. placed horizontally in the parent channel opposite the offtaking head, supported on piers or walls at such a level as to prevent the heavily silt laden water entering the offtakes. These tunnels terminate close to the undersluices.

Appended Figures show the Tunnel Type Excluders adopted at Khanki weir on Chenab, Trimmu weir at comfluence of Sutlej, Ravi and Chenab and the Kalabagh Barrage on Indus. The Khanki type has tunnels of varying lengths and the type at Trimmu and Kalabagh has tunnels of equal length. The Khanki type has the defect that as water approaching the excluder has to turn round a large angle, it gets disturbed and therefore picks up silt. Providing a divide wall here is disadvantageous as water in that case has to take an S-Curve to approach the tunnels. This sharp change from

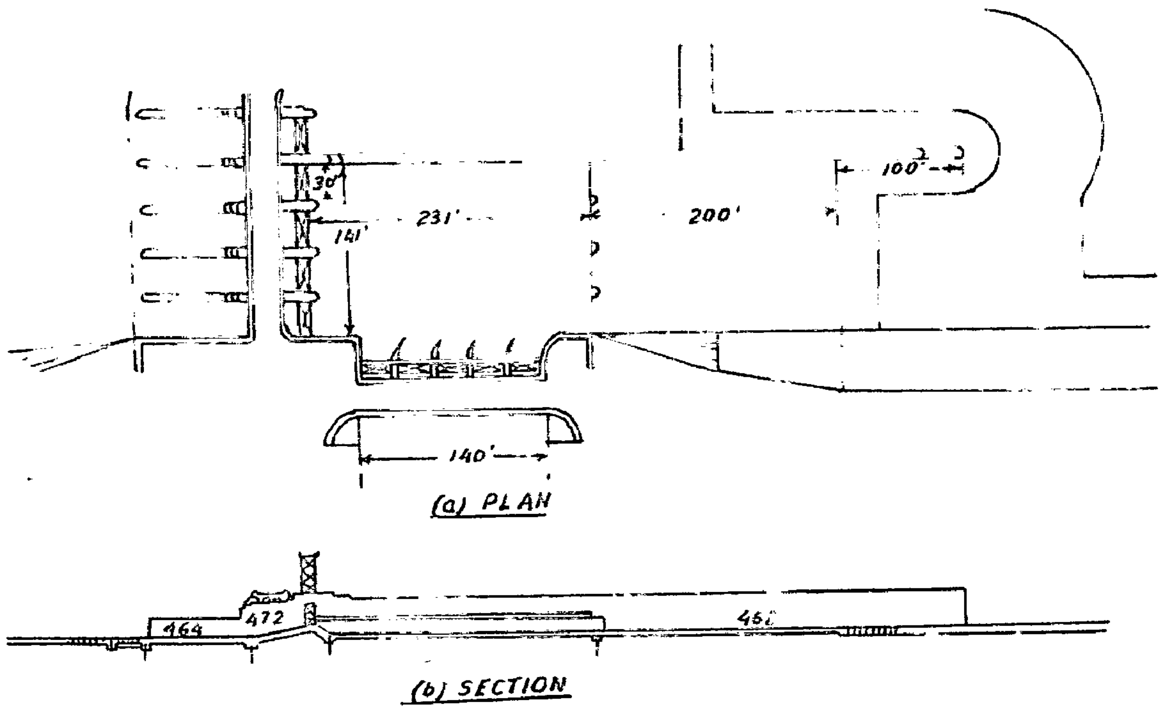


(a) PLAN

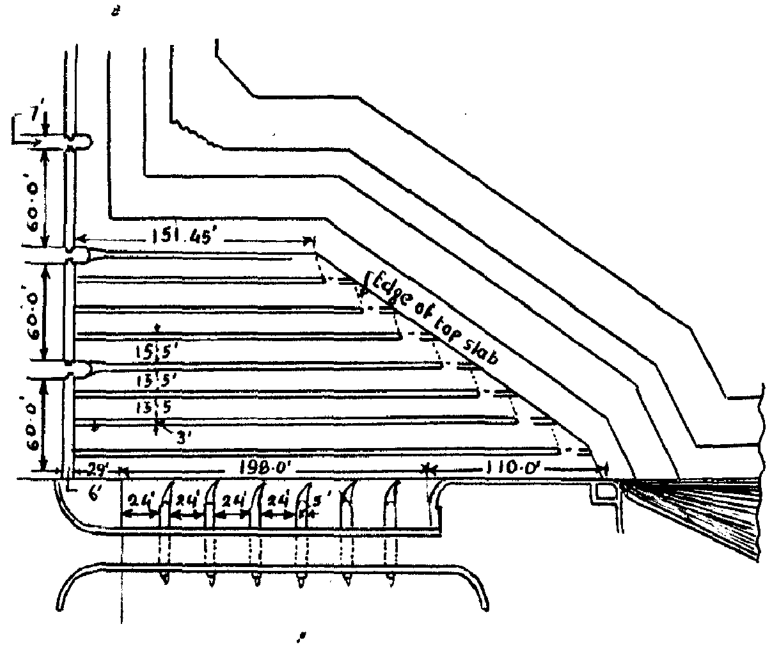


(b) SECTION ON "AB"

SILT EXCLUDER
(KHANKI ON CHENAB)



SILT EXCLUDER (TRIMMU ON SUTLEJ)



KALABAGH HEADWORKS

FIG. 8

one direction to another of water curvature causes turbulence and frustrates the object of the excluder. The Kalabagh and Trimmu type was an effort to overcome this difficulty but here the canal supply at the edge of the diaphragm is not under control.

Silt tunnels are generally suitable for use in deep channels and for small offtakes. The main points in the design are the level of the Platform, its length, width and shape. The regulations of these tunnels should aim at keeping the tunnels clear of deposit and dividing the top water and silt laden bed water smoothly so that no eddies are generated. Otherwise the coarse grade material will be thrown in suspension and the very purpose of providing tunnels will be foiled.

Mr. H.W.King divides silt tunnels under two main categories viz: (a) A platform with a curved extension of the D/S wing wall of the offtaking channel built on top of it so as to guide all the clear surface water flowing above the platform into the offtake and (b) A simple platform with no guide wing built on it. The heavy silt suspended in water can be made to drop into the lower layer of water by constructing a short reach of smooth pitching in the parent channel just U/S of the offtake, and pitching the side slopes in this reach. At the D/S end of this pitching the horizontal slab is constructed so that it divides the water. In order to prevent any of the silt free water from passing past the offtake, down the parent channel, a curved wing guide is built on top of the slab as an extension of the D/S wing wall of the offtaking head.

Level of Platform: In fixing the level of the Tunnel Slab the two factors to be considered are the height necessary for the 'tunnels' to perform their function satisfactorly and the discharge required to fill the offtaking channel. It is obvious that the higher the Tunnels are made, the more silt will be excluded from the offtake. To guard against the Tunnels getting blocked by jungle and debris a height of at least 2' is the bare minimum.

Width of Platform: The design of the width should be such that during the minimum supply in parent channel, there shall be enough water passing over the platform to fill the offtake, and some to spare.

Curved Vanes

A submerged curved vane of 1/3 the depth of pond, pointing downstream and projecting from a bank produces a local concave curvature which deflects the bed material away from the bank into midstream. The top sand free water, however, continues to flow in the original direction and is then drawn off by the pocket. It is to be emphasized that the vane should be located sufficiently far upstream so that any turbulence which throws the bed material into suspension dissipates itself before the draw-off into the pocket begins.

A curved vane is wasy to construct and maintain, and offers very little obstruction to the natural waterway is, therefore, a very useful excluding device, if located correctly.

The following points are worth noting in this connection:-

(i) A curved vane is effective so long as a deep channel is maintained along the bank.

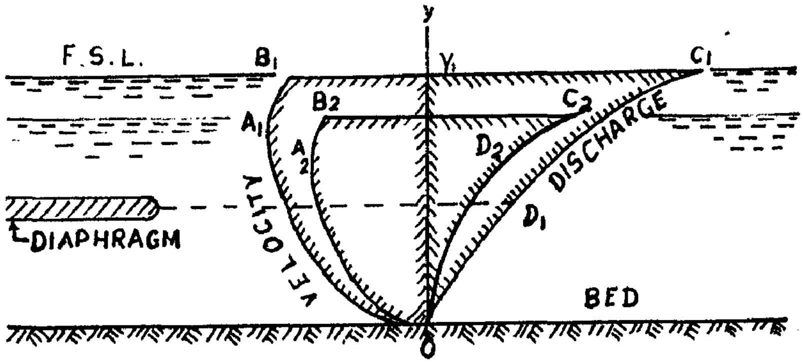
(ii) A vane on one bank is likely to attract flow towards that bank. To neutralize this effect, a similar vane on the other bank may be necessary.

(iii) Accretion is likely to take place between the curved vane and off-take due to persistent slack flow. If this occurs the bed material instead of being diverted into the midstream, hurdles over the vane and is likely to enter the offtake thus making the vane ineffective.

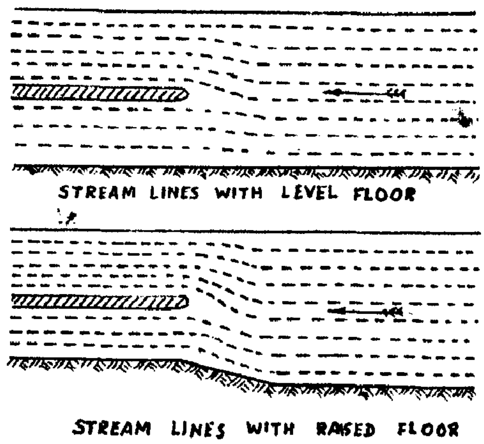
(iii) Skimming Platform:

These have the same action in a small part of the channel where they are constructed as the tunnel seperators. The principle is to skim off the top water which has a smaller load of sediment and allow it to pass into the canal, while the remaining water which has a smaller load of sediment and allow it to pass into the canal, while the remaining water with its heavier load of sediment flows down the river or stream. Naturally they seperate out only those particles which are too heavy to be lifted up over the platform by the currents which rise from the bottom and flow over the platform.

The late Mr.F.V.Elsden in his paper I.B.No.25 on "Irrigation Canal Headworks" suggested an early design of a Skimmer gate shown in figure.



EFFECT OF DIAPHRAGM ON VARYING SUPPLY



EFFECT OF DIAPHRAGM ON STREAM LINES

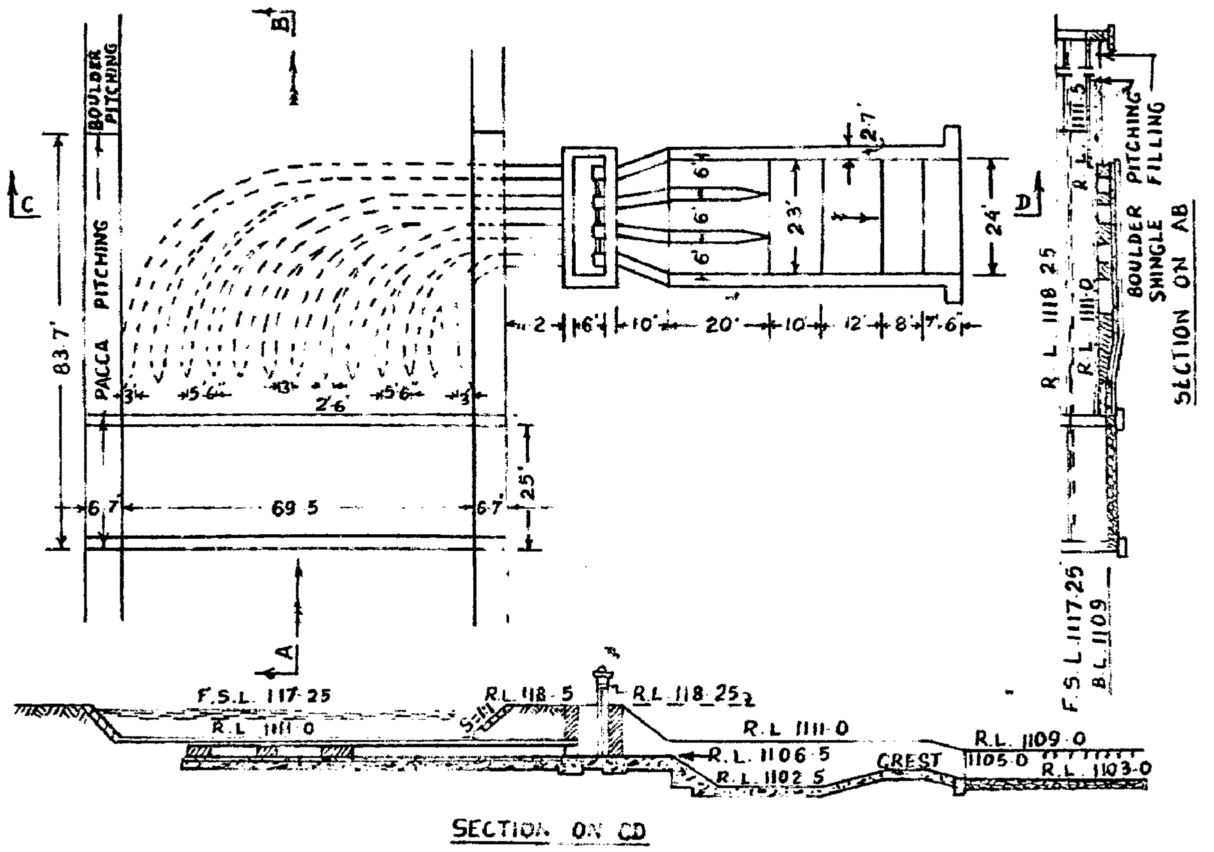
SEDIMENT EJECTORS

However efficient the excluder design at the head may be, a great deal of sediment load especially that in suspension would enterthe canal. This material has to be removed from the canal in order to maintain the discharging capacity of the channel. To effect this efficiently King's Vanes, sediment deflectors and ejectors have been devised.

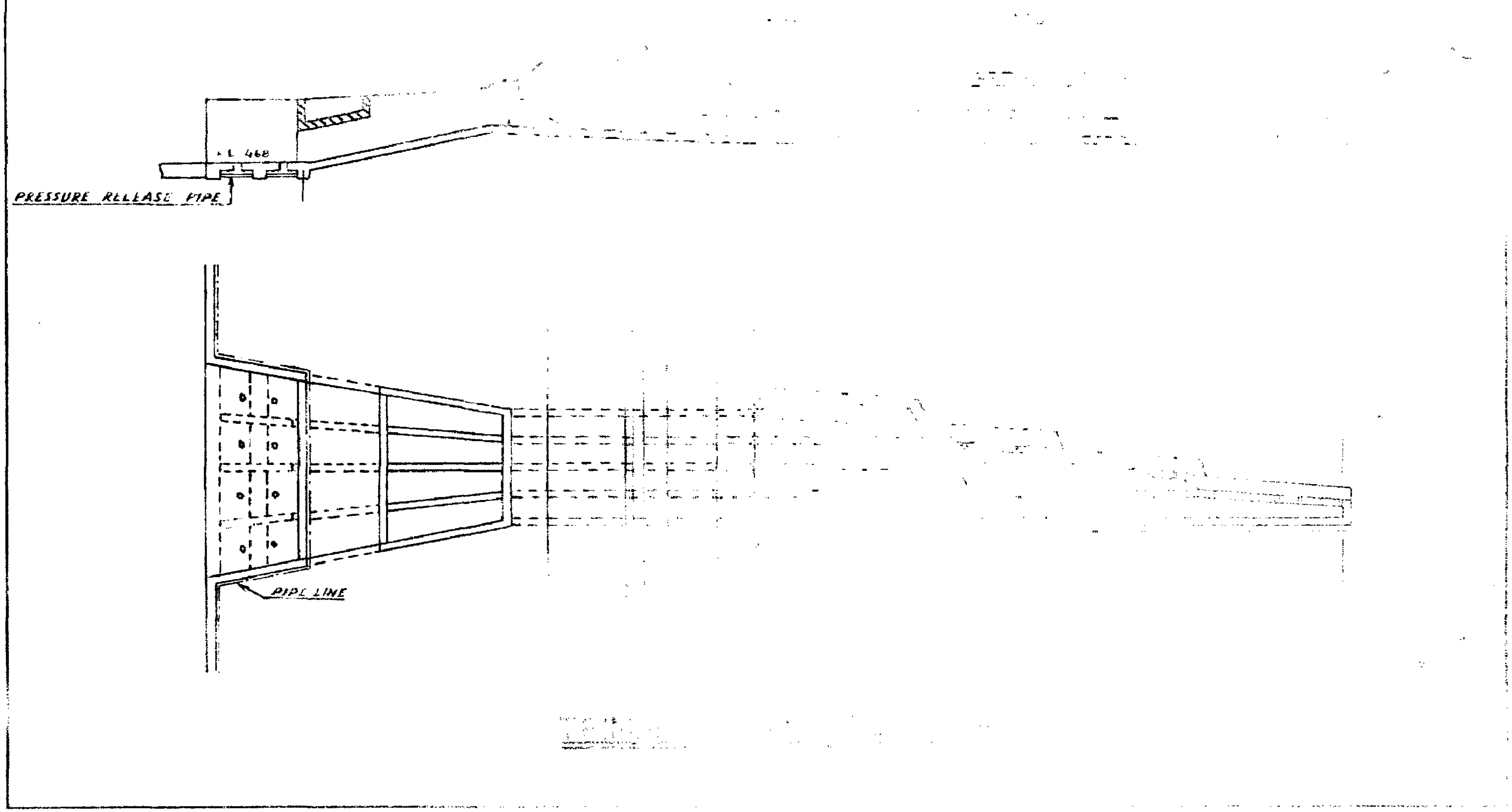
A remedy which has proved fairly successful in places where there is a good draft channel to carry away the ejected silt is the silt tunnel. It consists of a number of walls with vertical faces about 7 ft. apart parallel to the head regulator. They are covered over with slabs and thus form a number of tunnels placed side by side. All these tunnels have exits.at the undersluices. In some cases the tunnels are divided into two chambers, the upper and the lower, and are controlled by two gates. The lower is opened under normal flow conditions of the river and carries shingle and silt rolling along the bed. The upper is intended to function during the floods when the river brings in lot of heavy silt. In these tunnels much higher velocities than are possible in the system of vanes are generated. The flushing is therefore more effective.

In a large canal system one ejector may not be sufficient to remove the sediment trouble. A series of ejectors at different places on the canal systems may be

Contd.



ISLAMPUR EJECTOR



EL 468
PRESSURE RELEASE PIPE
PIPE LINE

required. In the head reaches of the canal on account of high velocity the sediment load consisting of large size particles are kept in suspension and, therefore, can escape extraction from the extractor constructed in the head reaches. In the lower reaches the velocity being smaller the material which was being transported in suspension drops on to the bed and forms as bed load. An ejector is required in that reach to deal with the bed load in transportation there. Similarly it applies to the lower reaches till all the harmful sediment load is extracted. A battery of ejectors instead of one may be required to deal with the sediment problem in a large canal system, e.g., the Upper Jhelum canal.

The question of escape discharge is very important since discharge for running the ejector has to be accommodated all along the canal system up to the point where the ejectors are constructed. It may, however, be observed that when a battery of ejectors is required the discharge used at each site will be less since it is required to deal with one grade of material. The sediment extractor should be as near the head regulator as possible since most of the coarse silt in suspension in the pocket settles down in the first mile or so. If the extractor is not located near the head regulator the sediment deposited in the canal will not reach the extractor. This sand may in winter be picked up bythe clear water and passed down the canal. This point was fully considered in connection with the location of extractors in the Haveli Main Line and Thal Canal.

Saddle syphon

Silt tunnels have been used as silt ejectors in canals. Since these are not automatic in action, the success of the ejection of silt depends on timely openings of the entrance to the tunnels.

A saddle siphon can also be used instead of these tunnels. During rains the water level of the canal rises due to inlets for run-off from the surrounding country. If a siphon can be made to prime at a small specific depth above the full supply level, all the extra water brought by the rains can be utilized in flushing a channel of its silt. During the period the river is in flood, a small discharge over and above the F.S.L. can also be allowed in the ~~chan~~ canals to prime the siphons to flush out silt. Figure shows a device which has been worked successfully on a canal in the Hydraulic Laboratory at Krishnarajsagar. This design can be placed at the positions where relieving weirs are now located i.e. near the junction of a cross drain with the channel. It can also be placed near the head regulator of the channel.

The design consists of:-

(1) A guide vane,

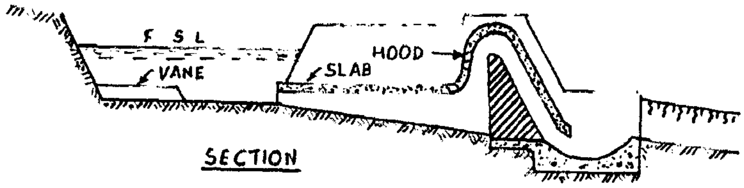
(2) An inlet tunnel or silt trap,

(3) A saddle siphon , and

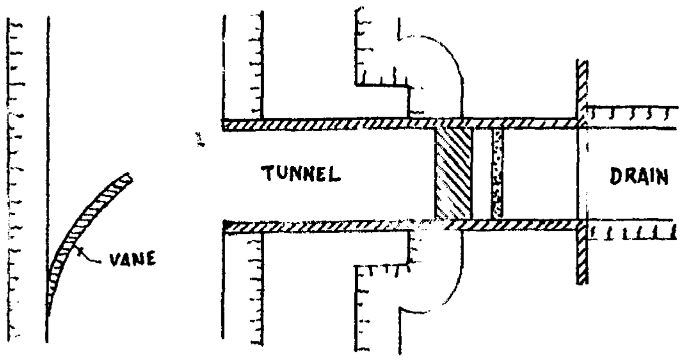
(4) Protective works in the rear of the siphon.

Guide vane. Its purpose is to guide the bed silt into the inlet tunnel. It is built protruding into the channel as shown in Fig. It need only be about 6 inches in height or so, just sufficient to divert the bed silt into the inlet tunnel. It should not materially reduce the waterway of the canal section nor should its height be too small to allow the bed silt to pass over it.

Inlet tunnel: The purpose of this is to act as a silt trap to retain all the silt led into it by the guide vane. It consists of a rectangular chamber with a sloping bed. It is covered over with slabs



SECTION



PLAN AT BED LEVEL

SADDLE SYPHON EJECTOR

so that no air enters the siphon when it has primed and the silt accumulated in the trap is not carried away by the canal discharge when the siphon is not functioning.

Saddle siphon. The purpose of the saddle siphon is to such out all the silt accumulated in the tunnel and therow it out into the natural drainage channel. The siphon is designed to prime quickly for small depths of flow over the F.S.L. This has been made possible by having the water seal in the form of abasin at the outlet end of the siphon. The tip C of the hood is kept at the same level as the bed of the natural drainage channel, so that even for very small overflow at the siphon crest, the water basin will get filled up to the tip of the outer shell of the hood and thus create a water seal. In a few moments after the water seal is created, the air inside the siphon is sucked out thus priming the siphon. The inlet mouth A is kept as deep as possible so as to prevent formation of vortices in the inlet tunnel chamber as vortices hamper lifting of the silt. The weir section of the siphon is designed for stability against water pressure. Its crest is curved and made concentric with the crown of the hood.

Protective works:- Suitable protective works will be necessary for a short length beyond the siphon outlet to prevent scour in the bed of the natural drainage channel. The nature of protective works naturally depends upon the velocity at the outlet and nature of the bed of the drainage channel.

The details of a siphon silt ejector installed at the Hydraulic Research Station at Krishnarajasagar, Mysore, are shown in Fig. The difference of water level between the F.S.L. of the channel and the bed of the drainage channel is only 2.75 ft. The siphon is able to discharge 8.4 cusec per foot length and can generate a velocity of 8.4 f.p.s The siphon has been able to eject 80 per cent of the silt accumulated in the tunnel chamber and some of it was as big as $2\frac{1}{2}$ in.in diameter.

The discharge in the siphon is calculated by the formula.

$$Q = cA\sqrt{2gh}$$

where Q = discharge in cusec.

A = area at the crest in sq.ft.

h = effective head in ft. (difference of level between the maximum water level of the canal and tail water level of the drainage channel)

c = coefficient of discharge = 0.7

The velocity of 8.4 f.p.s. has been sufficient to clear silt of even 2½ in. diameter. The maximum size of the silt which can be lifted by the siphon depends upon both the depth of the silt chamber as well as the velocity generated in the siphon. This relationship between depth, length of inlet tunnel, and the velocity necessary to remove a certain size of silt, has to be determined experimentally. On account of the flushing action of the siphon, accumulation of silt in the natural drainage channel is not pronounced.

VORTEX TUBES

Another device especially useful in case of heavy silt poad is the Votex tube. These have been quite successfully tried in the United States augmented with Riffle deflectors. It will be pertinent and useful to give a report of some of the designs tried there.

A sand trap consisting of riffle deflectors and vertex tubes was decided on as a means of trapping the Bed load entering from the Arkanas river into the heading of the M V canal. This canal had a capacity of 250 and a length of 54 miles.

Sand traps of various types were designed and tested in the Hydraulic Labs. at Fort Collins, Colo., by the Irrigation Div. of the Soil Conservation Service in Cooperation with the colorado Agricultural Expt. Station. The diffrent types of devices were developed with the intention of covering the range of field conditions as to hydraulic charecteroties and the nature of the material to be removed. The various combinations of width of channel, depth of water,, velocity of flow and amount and nature of sand to be trapped precluded the use of any one design to meet all requirements. None of the devices so far developed is capable of capturing and removing suspended material.

The Riffle Deflectors and Vartex Tube type of sand trap was chosen for installation at the heading of the 'Minnequa-Union Caal, because the combination of riffles and Vortex tubes results in a very efficient means of dealing with the heavy bed load. Also such an arrangement of riffles and tubes permits successful operation of a moder ate canal velocity.

Sand Traps For Narrow Channels:

For a relating narrow channel 8 - 12' wide, having velocities ranging from 4 - 7' with discharges 25 - 300 , the vertex tube sand trap has been perfected to eleminate the Bed Load of sand and moderate size gravel. Because of the velocity of flow it is assumed that the elevation of the tap of the tube can be so placed as to provide near critical flow over the tube, a condition necessary for optiminum vortical action and maximum transportation of material with this device.

The vortex tube is merely a tube of slight taper 1 to 50 from the outlet end, open at the t op side with edge, or lip, parallel to the axis and laid across the width of the channel at an angle of approximately 45°. The slope of the tube toward the outlet should be about 1 to 50.

For channels of greater width, moderate velocities in the order of 2.5 to 4 fts. and capacity of flow ranging from 50 to more than 500 , the riffle deflector type of trap has been developed. In general plan the structure is a rect. flume, of either wood or reinforced concrete and of suitable width and length. The floor slopes traversely 1 to 15 with the side opposite the outlets at grade of channel. On this sloping surface are placed the curved deflectors which in plan are parabotic.

Approx, $Y = 0.025 D^{1.9}$ for a structure 16' wide. In this eqn., Y is the offset from the line normal to the axis of the structure and D the distance from the side wall. The riffle has a vertical U/S face of 12 to 18" and the top side, 3 to 6' wide, slopes to zero depth at the D/S edge. Five to 10 riffles placed adjacent to each other constitute a set, the no used depending upon the nature of the bed load. For relating coarse material fewer riffles would required. The end of the riffles are in a time 2.5 to 4' from the wall of the structure, on the outlet side.

At the end of the first riffle, U/S side, is placed a vortex tube, set at 45° the top side of which is at the floor line, Additional tubes are placed at the U/S end of each riffle flowing D/S. If the bed load is fine sand in considerable quantity, each riffle should be provided with a vortex tube. For heavy sand and gravel, fewer riffles may be used and tubes added at every other riffle, These several tubes discharge into a compartment at the end of the structure and the total accomulation of sand is led away through a common outlet.

The sand trap or desilting works for Colorado Fuel and Iron Corp.'s division works consists of two channels in each of which are placed nine concrete riffle deflectors with an equal number of short vortex tubes with outlets discharging into a longitudinal compartment common to both channels. The length of riffle, sidewall to river end of the vortex tube, is 21', and the distance from the end of the riffle to the sidewall of the compartment is 4'. The ht. of the riffle, U/S Face, is 16" and the width or base is 7', as measured parallel to the axis of the structure. The base of the riffle at the outerwall is at the grade of the canal; at the other end it is about 18" lower.

Rifflesare parabolic in plan and laid out by offsets from a line normal to the sidewall according to the approx.eqn. $Y=0.028^{1.93}+0.44$ where Y is the offset and D the distance out from the wall (In feet, both). The curve of the raffle was determined from experimental riffles used in model tests in the lab. which proved satisfactory. The vortex tubes are cost of concrete to a diameter of about 8" and length about 6' and set at 45° to the channel axis. The compartment has an inside width of 4' 8" side walls, and flever that slopes towards the D/S or outlet end with a drop of 4" to 6" in 75'.

The effluent from the compartment is carried back to the river channel through a 24" concrete pipe. This discharge line orginally reached out to nearly the middle of the river channel at a point about 200' D/S from the diversion dam. At the outlet end of this pipe were placed a 60° pointed D/S, which was intended to increase the draft through the pipe. Nothing is known as to whether much was accomplished by this special fitting.

It was fully expected that by proper sluicing through the 30' radial gate openings in the dam the river bed between the dam and the end of the outlet would be secured out and a greater diffrence in the effective head between the water surface in the compartment of the sand trap and that of the river would be realised. Because of the excessive amounts of heavy gravel moving in the river, the sluicing operations caused the filling-in of the bed of the channels reducing the velocity in the outlet and eventually coverning the end of the pipe. A part of this line was then abundoned and an extention of about 300' of 24" pipe was laid parallel to the river channel and outletted on the bank of the stream. About 200' of this new line has an adverse grade of 1%; however it has proved effective in carrying away the sand and gravel accumulated in the compartment of the sand trap.

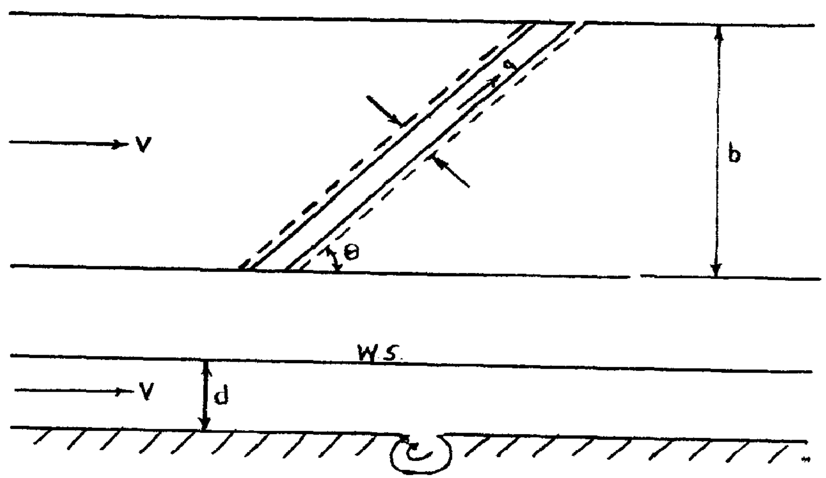
At the outlet end of this new line is a tide gate which during brief periods of extreme high stage of the river caused by flash floods is closed to prevent backwater from moving into the compartment of the sand trap and to prevent material from bein g carried into the pipe which might under certain conditions clag the line.

At firs-t the outlet of the several vortex tubes were at full opening. Later a simple slide gate, placed vertically on the wall inside the compartment, was provided for each tube. Experience in the operation of the sand trap indicates that better results are obtained by throttling down the tube discharge.

No quantitative tests of the efficiency of this large sand trap have been made. It is probable that special apparatus will be necessory to examine this matter effectively because of the nature of the problem and the conditions under which such tests can be made. Visual observations over the past several months indicate that the sluicture is operating satisfactorily.

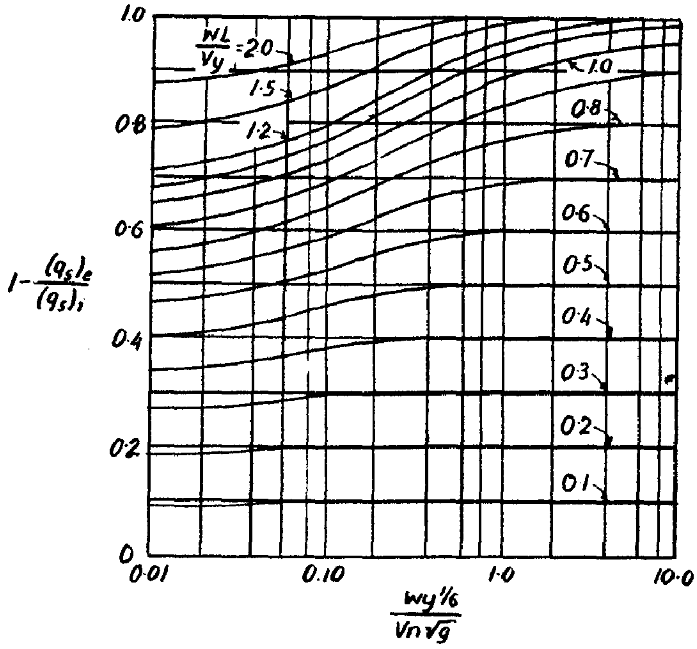
Another type of sand trap devised at the Fort Collins Labs. is a practical device called the riffle vane type. In it's present state of development it is not entirely ~~misume~~ immune from trouble. The names formed of 8 or 10-gauge sheet metal, are curved in plan and of special design. These names, when properly spaced and set in a line normal to the axis of the structure, will cause the bed load to move laterally accross the channel immediately D/S from the line of deflectors.

This devise is intended for use at ordinary velocities found in irrigation channels having a moderate width, and not for excessive bed load. Because of the nature of the names and spacing, it is found that debris moving along the bottom of the becomes lodged in the vanes and interferes with their action.



SCHEMATIC LAYOUT OF VORTEX TUBE.

FIG. 13



SEDIMENT-REMOVAL FUNCTION FOR SETTLING BASINS.

FIG. 14

(i) <u>Settling Basins:</u>

Settling basins are enlargement of the channel in which the velocity and the terbulence are reduced, and the sediment settles to the bottom from whence it can be flushed or pumped out. This type of arrangement will remove more finer sediment than any of the other types. Theoritically, as much sediment as desired can be removed from the water in a settling basin, but it is rarely practicable for irrigation purposes to remove a high percentage of particles smaller than sand i.e. 0.06 mm. diameter from the water. Of course none of the devices discussed will seperate material much finer than sand.

At the head of the All American Canal, 72 circular basins are provided. The silt is scraped to a central duct by rotary scrapers and then discharged into a drain. They deal with 80% of the canal flow amounting to 12,000 cusecs and the silt is discharged back into the parent river. The All American Canal supplies the Imperial Valley of South California and was designed with a minimum slope of 0.000053 for a capacity of 15000 cubic feet of water per second when in full operation, and for the transport of 12,000 tons per day of sediment having a mean diameter of 0.075 mm. Had the water been admitted with its normal load of 60,000 tons per day of sediment with a mean size of 0.080 mm. a slope some three times as large would have been necessory for stable conditions.

Effective design of a desilting basin depends upon an analysis of the effect of turbulence upon the rate of deposition. Camp has evaluated the turbulent transport function for two-dimensional flow by making use of the same two simplifying assumptions: (1) that the fluid velocity is the same at every point in the channel, and (2) that the mixing coefficient is also the same at every point. On these premises, he derived a functional relation for the ratio of the sediment leavin g the basin to that entering,

$$\frac{(q_s)_e}{(q_s)_i} = \phi \left(\frac{\omega}{\sqrt{T/\rho}}, \frac{\omega L}{Vd} \right)$$

in which $(q_s)_e$ = quantity of sediment of given particle size in effluent

$(q_s)_i$ = quantity of sediment of given particle size in influent

ω = fall velocity of the given particle size

$\sqrt{T/\rho}$ = shear velocity = $\frac{V_n \sqrt{g}}{1.49 d^{1/6}}$

d = basin depth

L = basin length

V = mean velocity of flow in basin.

This function has been evaluated analytically, as shown in fig. and verified experimentally by Dobbins. It provides a practical and relatively easy means of estimating the effectiveness of a settling basin in removing suspended sediment, a nd, conversely, of designing a basin to remove sediment of a size which cannot be transported in the canal which the basin will serve.

Application of the foregoing function to the design of a settling basin requires the following steps:

Determination of the maximum size of sediment which the cannal will transport, by application of procedures outlined in Article 16. This will be the minimum size which the settling basin will be designed to remove, unless smaller material is to be taken out because of its detrimental effect.

Preparation from the sediment size-distribution curve of a table of size ranges, mean sizes, and quantities of sediment in each size range.

Determination of the fall velocity for each mean sediment size.

Preparation of a systematic series of combinations of settling-basin dimensions, L, and B, and computation therefrom of the mean velocity V for the design discharge.

Selection of the Manning n and evaluation of the term:

$$\frac{1}{1.49}\,\frac{\omega}{\sqrt{\tau/\rho}} = \cancel{V}\,\frac{\omega\, d^{1/6}}{V\, n\sqrt{g}}$$

Determination of the ratio $(q_s)_e/(q_s)_i$ for the various combinations of dimensions by solution of the functional relation and use of fig.

Selection of the basin dimensions on the basis of cost of construction weighted against sediment removed.

Perhaps the outstanding example of a desilting works in America is that provided for the All-American Canal. A series of basins has been constructed in which the sediment load will be reduced from an estimated 60,000 tons per day to 12,000 tons per day in a flow of 15,000 cubic feet per second. Each basin is 269 feet long and 12.5 feet deep, with a mean velocity of 0.24 foot per second. The time of retention is approximately 14 minutes. Investigation by the procedure of Camp indicates that, of the 60,000 tons per day entering, 55,000 tons will be removed, a figure which does not eiffer greatly from the design data of the Bureau of Reclamation based on the earlier work of Hazen.

(ii) <u>Silting Tanks & Traps:</u>

It after exhausting all the measures for reducing silt entry at the head it is found that the channel is incapable of carrying even the suspended silt thrown to it's lot then these silting tanks and taps can be resorted to. A silting channel for which the levels of the commanded area do not permit steepening of the gradient, can be rebened of silt trouble by diverting the water through an artificial tank formed by constructing banks on the slides of a natural depression. On account of the larger sectional area provided by the silting tank much of the silt trouble will drop and the water emerging from it will be so free of silt that it may even pick up silt after coming out from the tank. The main drawback of this arrangement it that only the length downstream of the tank can receive benefit and also that a depression has to be tooked for every time a tank will last for a pretty long time. Sometimes on account of perfectly clear water running through the tank the trouble of weed growth may set in. In such a case it will be preferable not to make the width of the tank every large. By experience it can be known what section for the tank would be just sufficient.

Silt traps are pockets in the canal bed which on account of increased section reduce the velocity of the water in the canal and thereby induce deposition of silt which is flushed out periodically through an escape.

A. Proportionate distribution of Silt:

Entry of sediment into a channel can be tackled either by exclusion of coarse silt where this type of silt is present or distribution of the total sediment load between the distributaries. The exclusion of coarse silty is achieved by diverting through silt excluders heavily charged lower layers of water away from the channel and can be well adopted at heads of Main Canals taking off above a weir or barrage where a fairly, big fall is available. In case of distributaries, branches, minors, however, exclusion of silt on the above lines is not generally practicable for obvious reasons.

Attempts have been made to distribute the total sediment load amongst the various offtakes proportional to the disbharge drawn by each, or, when desirable in proportion to the carrying capacity of each distributory. In cases where a very large percenrage of sediment load consists of five silt (0.075 mm.) as in Sind, silt exclusion by mechanical devices is not practicable. In cases, therefore, where an offtaking channel is drawing silt in excess of that drawn by other channels taking off from the same canal, the most practical solution is the distribution of silt amongst all the offtakes in proportion to discharge drawn by each. In Sind this has been effected by a divide wall suitably positioned. The divide wall has been found satisfactory in cases of offtakes carrying 10% or more of total discharge in parent channel.

The report of the work done at the Development and Research Division, Sind, is of some intrest. The following relationships were formulated with the help of laboratory model studies:

$$A = 0.66\ Q + 13.65$$

where A = percentage area required for proportionate silt distributieng head as compared to the area of parent channel at nose of divide wall.

Q = percentage discharge of offtakes as compared to that of parent channel

with reference to velocities,

$$V = 1.15\ Q + 50.05$$

where V = percentage mean velocity required in the offtake at the entrance as compared to the mean velocity in the parent channel.

These emperical relationships can be developed on a more rational basis taking the sediment charge as an essential factor and rules of practical utility evolved. Further work on this problem is direly required. At present individual problems are solved with the help of model studies.

B. Inclined Offtakes:

In very few cases could the difference of opinion be so keenly divided as it lately was on the cannal approaches from rivers. Right angled approach channels and offtakes were once very much preferred. There was also a time when a reversely inclined approach was consided to be the best. It has only recently been definitely affirmed that perpendicular offtakes bring in more silt. The reasons are stated below:-

(a) The first and the most potent reason is that as the top water has greater velocity than the bottom water, its momentum is also greater. It is therefore less easily deflected into the canal than the heavily loaded bottom water. It the drawing force of a regulator is expressed in terms of velocity and it for the sake of simplicity two dimensional resolution of velocities alone is considered, the vectorial diagrams of bottom and top waters for right angled offtake will be as in figure. The resultant velocity of the slow moving bottom water will be found to be more inclined towards the canal head than of the top water moving at a greater velocity. Similar diagrams drawn for an inclined approach, show the conditions considerably improved. The difference in the inclination of resultant velocity of the bottom and top waters is very much reduced. An inclined offtake is thus enabled to attract more of top water. As top water is not so much laden with silt, drawing it in greater proportion ultimately reduces the silt charge in the canal. On the same analogy more light could be thrown on the offtakes reversely inclined. In figure are drawn vectors separately for water moving at bottom and top layers from the river into the downstream inclined offtake. From these it will be seen that instead of matters improving they get worse. The resultant vector for bottom water is much more inclined in comparison to the top water than even in the case of right angled offtake. Consequently from the point of view of silt drawoff the upstream inclined offtake i.e. the offtake inclined to the downstream side of the river is better and next to it is the right angled offtake. As the angle of inclination of the offtake with the upstream side of the river increases, the quantity of heavy silt entering into the canal reduces.

(b) Any obstruction or unevenness in the bed or sides of a parent channel close to an offtake vide fig. causes the water passing it to be pushed away from it. The force with which the water is thrown away increases with the velocity of water. The faster moving top water is pushed off more quickly than the sluggish bottom water which therefore finds entry into the offtake.

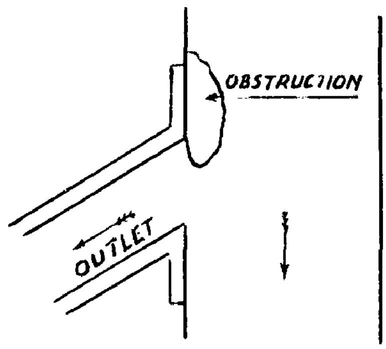
(c) Water entering the offtake approaches in a curve so that the phenomenon noted on the bends of streams and channels is brought in to action whereby free water level at the outer bank is higher. It is maintained by centrifugal force is proportional to $\frac{V^2}{R}$, the fast moving higher layers of water take to the outside of the curve and therefore are drawn away by the undersluices, the lower layers enter the canal.

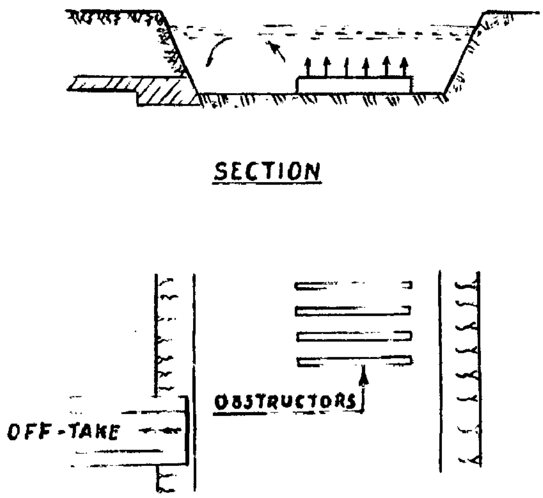
This will be evident from a dish containing water, revolving round its axis. The water at the periphery of the dish will be at a higher level than at the centre. So also on a curve the level of water on the outer side is higher.

The lamina of water in immediate contact of bed is prevented by viscosity and friction from attaining a high velocity with which the remaining layers of water move. It has, therefore, not enough centrifugal force to keep it away against the tendency of flowing inward caused by higher level of water on the outside of the curve resulting from centrifugal force. Thus water at the bottom has more tendency of flowing inward caused by higher level of water on the outside of the curve resulting from centrifugal force. Thus water at the bottom has more tendency of running into the offtake than the top water.

An offtake with its axis in line with the current, see fig. is therefore ideal from this consideration. But there will be difficulty in scouring out the pocket. This question will be dealt with at greater length when the design of approach channel is discussed.

<u>Silt prevention by obstructions:</u> Advantage is taken of the high velocity possessed by the upper layers of water. Either bed is roughened or one or more obstructions as shown in fig. are placed in the parent channel upstream of the offtake. They are removed a little from the edge to throw up the bed silt to be carried away by the fast moving top





OBSTRUCTION IN CHANNELS

water direct into the channel past the offtake so as not to be sucked in by the latter. When there is no obstruction, on account of the pull exercised by the offtake, the slow moving bottom water heavily loaded with silt is diverted easily and drawn into the offtake. But after the obstruction is put in, the bed silt is thrown up to the surface so that when the bottom water is drawn by the offtake it is comparatively silt free. The obstructions or the roughening of bed should not be so stationed that instead of working beneficiently they may cause more silt to enter the offtake. This will happen if it sis situated too far upstream of the offtake or too close to the edge of the parent channel on the side of the offtake as in fig.

C. Raised Cill:

The early efforts of engineers in Punjab to control silt in offtaking channels include a variety of arrangements based on raised cill. These arrangements were in fact quite inefficient and belied the faith pinned on them. But they were clearly better than nothing. The arrangement consists of a simple raised cill or wall, across the month of the offtaking channel at such a height as to allow only as much water as is required to fill the offtaking channel. The mouth of the offtake is usually bell-mouthed to enable a long cill to be made so that it can be made at a high level and still allow the required discharge. The idea is that the clearer surface water goes over. This was probably the first attempt at silt exclusion tried in India.

In 1908 two engineers Mr.Gordon and Mr.Scratchley introduced the system together with a skew head and curved upstream wing wall in Punjab mainly for distributaries. This was certainly an improvement on the existing measure of a simple right-angled offtake.

In 1916 the late Mr.F.W.Woods elaborated it in his 'Rising Cill' device otherwise known as "Woods Rising Cill Gates". It consisted of a raised masoury cill across the offtaking regulator with the regulator gate housed behind the cill. When the offtaking channel is fully open with gate raised as required when the supply has to be reduced so as to form an 'Overshot' regulator. For better efficiency the regulator was made very wide so that the raised masoury cill could be built at a higher level. A study of the working of this arrangement at Nasrana Distributory - Jhang branch of The Lower Chenab Canal - shows as in other cases that it is rather expensive and quite inefficient.

Another variation of the raised cill is the "Gibb's Semi-circular Wall" opposite the offtake and completely enclosing it is built a semicircular wall of such height as to allow the required discharge into the distributary at the lowest level of the canal. This has an additional disadvantage over a simple raised cill that as the wall is protruding into the main canal it will obstruct the parrage and throw up bottom water into the surface from where it will be drawn into the offtake.

D. Kings' Vanes:

Silt vanes and silt vanescum-curved wing are probably the best of the devices to be described hereafter.

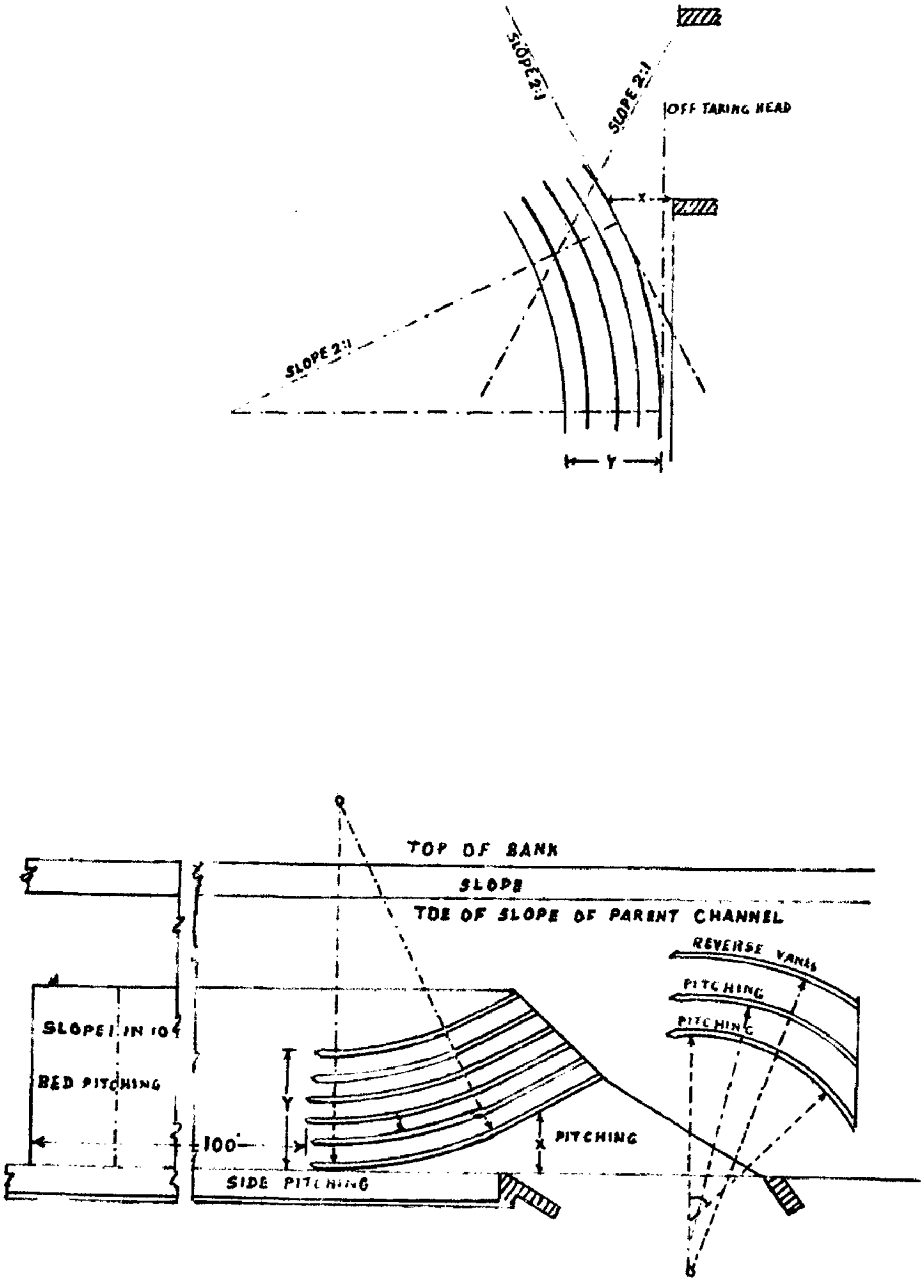
Silt Vanes are vertical diaphragm walls parallel to each other, starting in line with the current and terminating at an angle with it. They are constructed for a short height so as to bandle only the bottom most heavily loaded layers of water. Their object is to divert the bottom water smoothly away from the offtake.

Designs of Vanes. (i) Radius. The radius should not be so sharp that water on meeting the silt vanes instead of being guided by the vanes jumps over them.

Radius depends upon velocity and the larger is the radius the more efficient will the vanes be. The radius may not be less than 25 ft. but more than 40 ft. should be aimed at.

(ii) Downstream ends of vanes. In order to prevent water approaching the offtake crossing the vanes at right angles the downstream ends of vanes should be tangential to lines not less acute than 2 : 1 (27°). If the tangents make greater angle than 27° with the axis of the parent channel, water approaching the offtake may pick up silt from in between the vanes.

(iii) Length of vanes. The length of vanes should be such that the drawing action of the offtake will have no effect on the silt: if the vanes are short the silt may again enter the offtake. To safeguard against this the vanes should


SLOPE 2:1
SLOPE 2:1
OFF TAKING HEAD
X
SLOPE 2:1
Y
TOP OF BANK
SLOPE
TOE OF SLOPE OF PARENT CHANNEL
REVERSE VANES
PITCHING
PITCHING
SLOPE 1 IN 10
BED PITCHING
Y
100
X
PITCHING
SIDE PITCHING

SILT VANES

go beyond the line drawn at 2 : 1 slope from the down-stream end of the offtake, see fig. Actual length beyond this line may vary from 2 to 5 ft. according to the discharge.

(iv) Distance of vanes from the offtake. If the vane nearest to the offtake i.e. the longest vane is situated too close to the offtake the water passing above the vane may set up eddies and pick up silt. If the eddies are to be avoid the longest vane should be sufficiently away from the offtake. Distance X in fig. may accordingly be as given in Table

(v) Width of channal covered by the vanes. This width marked Y in fig. depends upon:-

a) depth of the parent channel,

b) height of the vanes,

c) functioning of the offtaking channel i.e. if it is badly silting channel or otherwise.

The width will be comparatively more for small offtakes than for large ones. In general the width may be half the bed width of offtaking channel. Table gives approximate dimensions of Y.

TABLE: VIII

DIMENSIONS FOR SILT VANES RECOMMENDED BY KING

Width of taking channel=W	For strong effect			Cheaper design (less effect)			Minimum dimensions recommended.		
	Value of X	Value of Y	Value of R	Value of X	Value of Y	Value of R	Value of X	Value of Y	Value of R
1.	2.	3.	4.	5.	6.	7.	8.	9.	10.
2	4	2	30	3	2	25	3	2	25
6	6	4	40	4	4	30	4	4	30
10	7	6	45	5	5	30	5	5	30
16	8	9	50	6	6	35	6	6	35
25	10	13	50	6	9	35	6	8	35
40	12	20	65	9	15	50	6	10	40
50	12	25	100	9	18	75	6	12	50

When silt vanes alone are used, care should be taken to see that there is sufficient water passing over the vanes to fill the offtake with plenty of spare.

Height of vanes: For strong effect height = $\frac{1}{4}$ to 1/3 depth of parent channel. Less height will ordinarily do.

Width: The thinner the vane the better it is, but from consideration of stability thickness may be 0.4 ft. for vanes less than 1.5 ft. high and 0.8 ft. for higher vanes.

Spacing of vanes: The width of channels between vanes may ordinarily be $1\frac{1}{2}$ times the height of vanes.

Bed and side pitching: The bed of the parent channel covered by vanes and for a distance of 50 to 100 ft. upstream of the vanes should be pitched smoothly so that the suspended silt may fall as low near the bed as possible. To minimise tendency to silting the pitched floor on which the vanes are built should be 0.5' heigher than the normal bed of parent channel. Flooring should be sloped off at 1 : 10 for a distance of 15 ft. at upstream end as shown in fig.

If some silt is found collected at the downstream end of vanes some reverse vanes. Fig. may be necessary.

The difference between silt vanes and tunnels is that the silt vanes are curved and open to sky while tunnels are straight and covered. There is no possibility of silt being picked up from tunnels.

Silt vanes are not suitable.

(a) where the discharge of offtaking channel is more than one third of that of the parent and

(b) where a small offtaking channel is situated between two large branches and has its bed at high level.

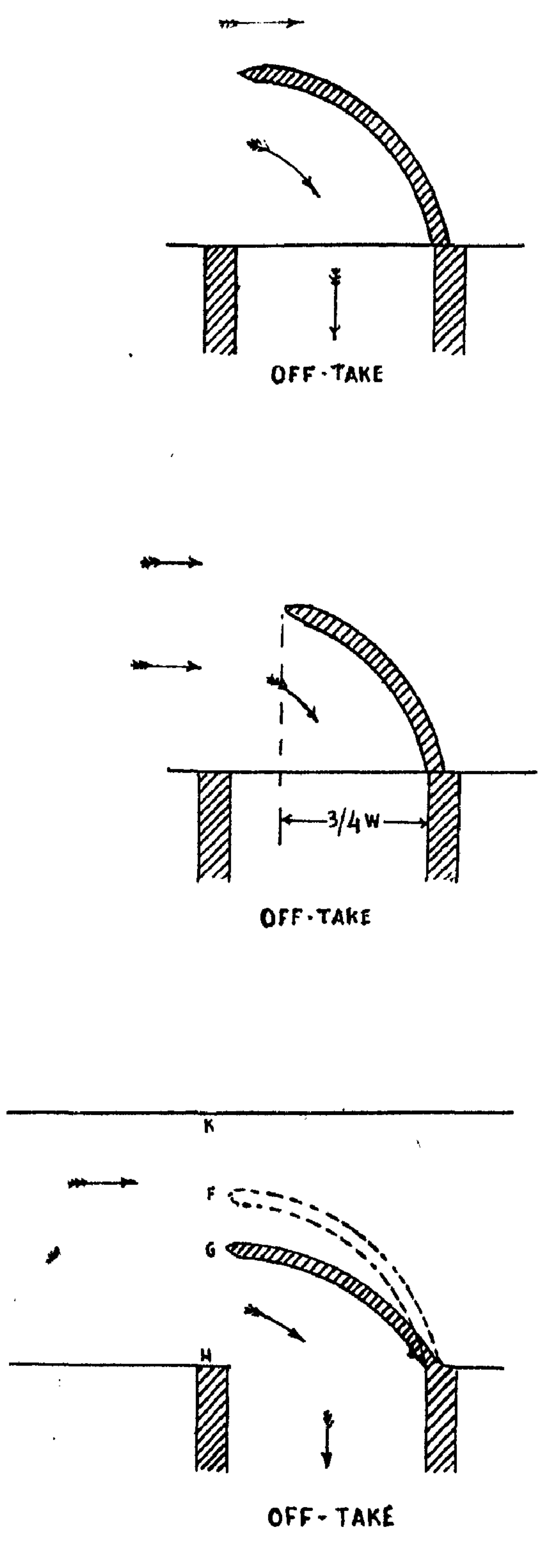
A silt platform will probably meet such cases.

(c) Where a fixed discharge is run, the King's vanes are capable of excluding practically the whole of the bed charge but their efficiency also depends upon whether the direction of the current is in line with the vanes at the start. Since where the discharge varies considerably the latter condition cannot be ensured, their efficiency under such conditions is soubtful for then they throw up coarse material into suspension and also form sand banks downstream of the vanes.

Consequently they should be avoided where the water level is likely to surge very much.

[illegible]. Curved Wing. This is a curved vertical wall constructed in the parent channel with its downstream end in continuation of the downstream abutment of the offtake Fig. . Without a curved wing the offtake takes more of bottom water and less of top water for the reasons explained in para . The curved wing embraces the surface as well as the bottom waters and forces their entry into the offtake, which then takes the same proportion of silt as the parent channel.

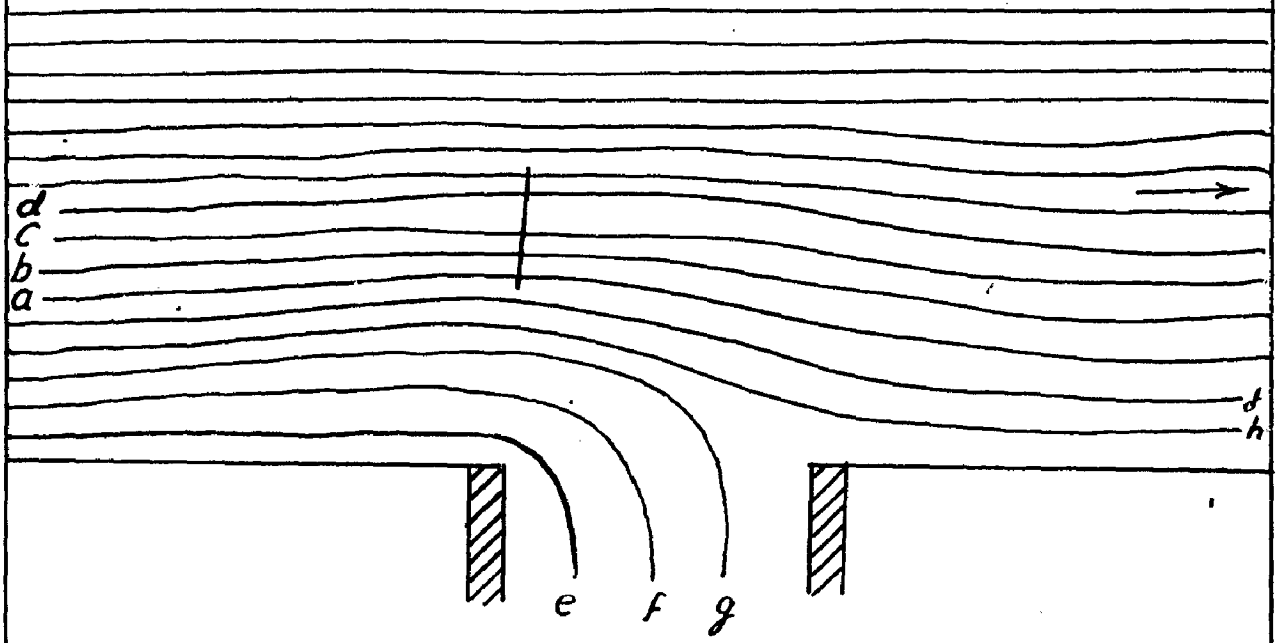
[illegible] Design. The curved wing need not have its upstream end tangential to the central line of flow of the parent channel nor should the downstream end be tangential to the downstream abutment of the off-taking channel. Curved wing may extend upto the point opposite to the upstream abutment of the offtake Fig. , but if conditions do not permit it may go only up to $\frac{3}{4}$ width of the offtake, Fig. The curved wing ordinarily projects far enough into the parent channel so as to enclose sufficient discharge of the parent channel to fill the offtake when the parent is carrying lowest supply at which the offtaking channel is desired to run full. It may be understood that if the wing extends too little into the parent channel, Fig. as the discharge of offtake is fixed, whatever discharge would have passed through F H , will have to be forced in through GH, i.e. the part of discharge, which would have passed through KG, will now be forced through GH. Naturally the lowest layers, which are easily deflected will get into the offtake. If the wing extends too much into the parent channel the conditions are reversed and more silt laden water passes on into the tail of the parent channel. If some openings are left at the foot of curved wing so that water escapes back into the parent channel more of water will have to be cut off by the curved wing to make up for the water lost through the openings.



CURVED WINGS

RIGHT ANGLED OFFTAKE WITHOUT CURVED WING

PATTERN OF FLOW FOR TOP WATERS



PATTERN OF FLOW FOR BOTTOM WATERS

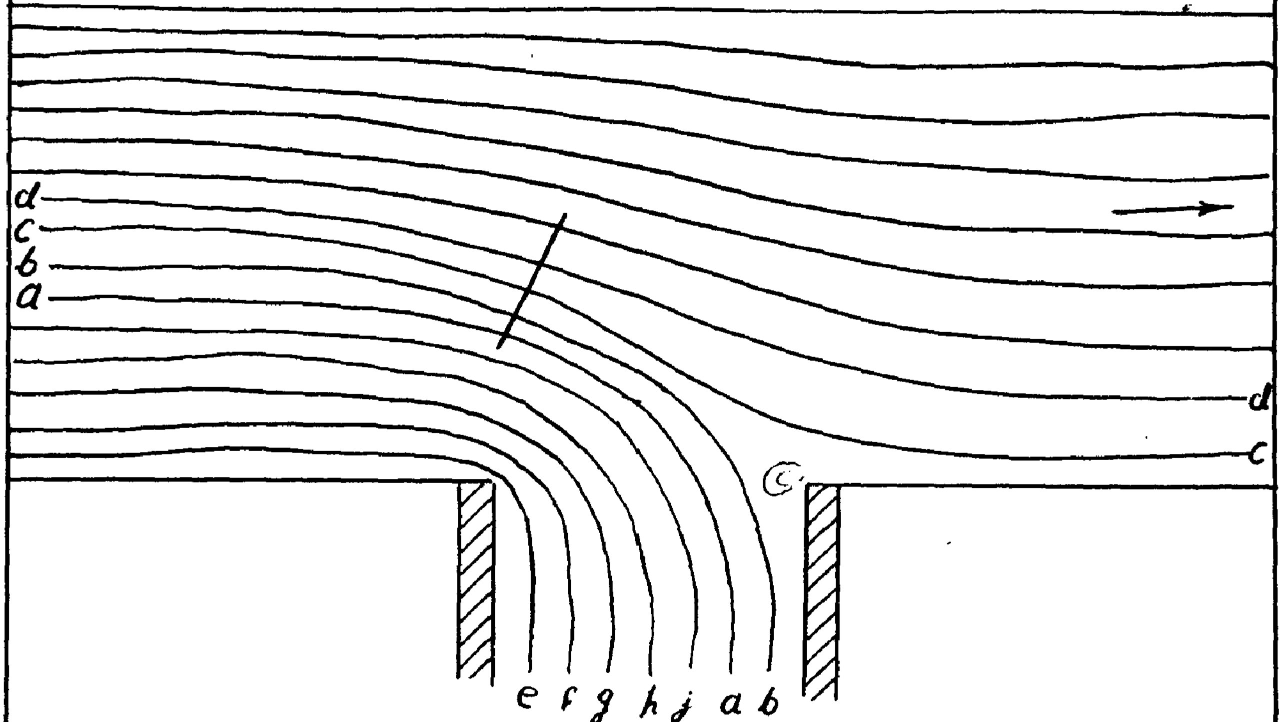


FIG. 1.9

RIGHT-ANGLED OFF TAKE WITHOUT CURVED WING:-

PATTERN OF FLOW:-

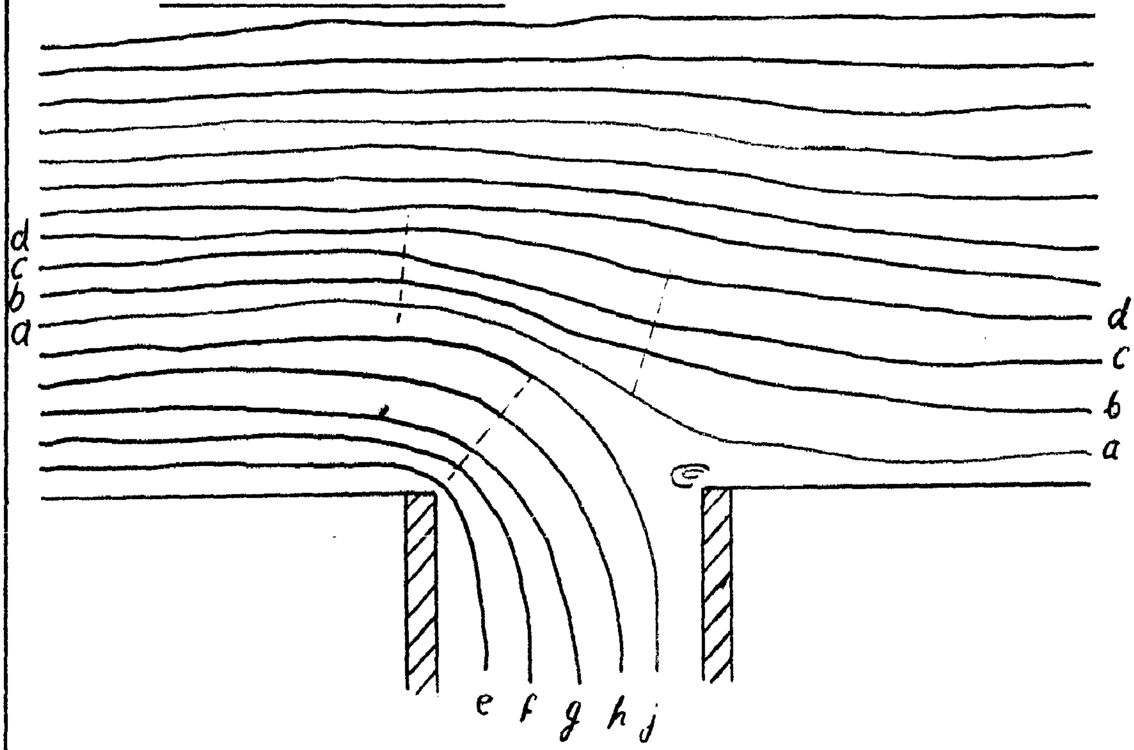
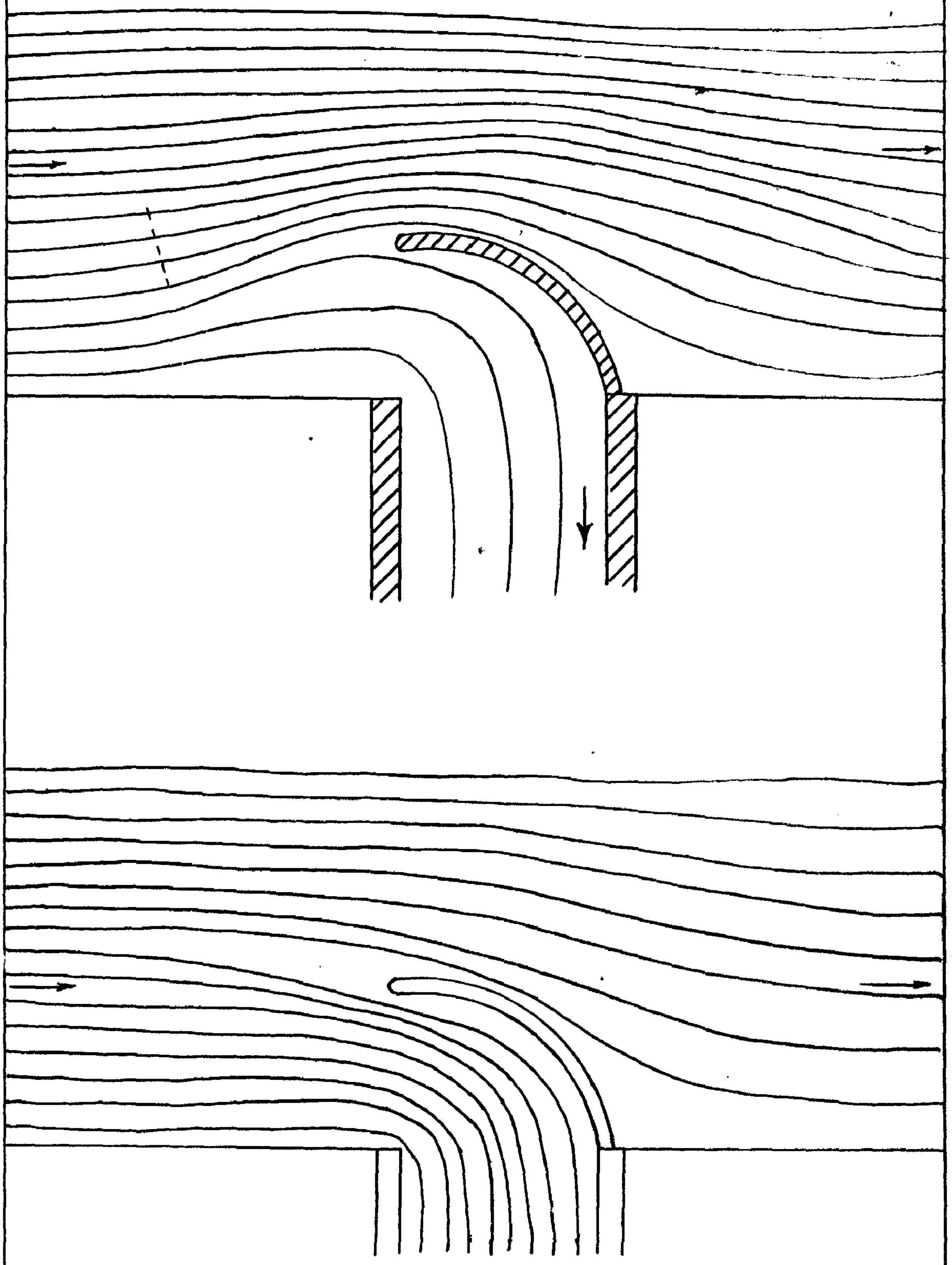


FIG. 20

CURVED WING

a) PROJECTING WELL INTO MAIN CANAL PATTERN OF FLOW SHOWING FAVOURABLE CURVATURE



b) PROJECTING TOO LITTLE INTO MAIN CANAL UNFAVOURABLE CURVATURE DUE TO SUCTION

FIG. 2[illegible]

In case the curved wing, pitching in the bed is generally not required.

The curved wing, can be used in cases where the offtaking channel on account of its gradient has the same silt carrying capacity as the parent channel.

[redacted]. Silt vanes-cum-curved wing. [redacted]. Though silt vanes have been used with success where the offtaking channel has the discharge carrying capacity more than one-third of the parent channel yet silt vanes-cum-curved wing is the right type of device for such a case.

Silt platforms & Skimmer Heads:

Skimmer platforms have been constructed at the head of certain distributaries taking off from the main line canal which brings in heavy sediment load. The platforms may be wooden or cement concrete. The height of the platforms may be according to the conditions of the main line and the distributary. Generally platforms are two to three ft. high from the bed of the main line and in level with the sill of the distributary of the regulator. Water charged with heavy sediment load flows straight while the top water goes into the distributary.

Skimmer platforms work quite efficiently in checking material entering the distributary to start with but later on the bed of the canal becomes silted up and the platforms more or less get buried into the sand deposit which finds its way into the canal. In the Punjab such skimmer platforms have been constructed on several canals and the experience is that their working is very satisfactory in the beginning but in course of time the efficiency diminishes considerably. The working of one such skimmer platform is given here to illustrate the point.

Skimmer platform on the Bubehali Distributary, Upper Bari Doab Canal, Main Line Upper:- This is one of the channels on the Upper Bari Doab Canal which Kennedy kept under observation for his flow formulae. A large quantity of silt was drawn in by this distributary from the parent channel. Its discharging capacity decreased considerably. In 1935 a skimmer platform was constructed at the head of the distributary which silted up badly. A skimmer platform at a level higher than the previous one was constructed in February 1949. Since the construction of the platform the distributary has started working very efficiently. The head reach has scoured out considerably. For the same gauge in the distributary the discharge has increased from 52 to 82 causec, for the same gauge in the distrib utary.

Silt Platforms consist of a horizontal slab usually of reinforced concrete supported on piers at such a level as to exclude heavily loaded bottom water, [redacted]. Because the piers divide the space below the platform into several channels, this arrangement is also known by the name of silt tunnels.

Design. Level of the platform. The level should be suchnas to

(a) exclude maximum of bottom water,

(b) leave such quantity of top water that should fill the offtake

and (c) the depth of tunnels should be sufficient to allow stray debris to pass through the tunnels without choking them.

From this last consideration the tunnels should not less than 2 ft. in height. The deeper they are the better it would be. The choked up tunnels are worse than no tunnels because then all the bottom water will roll upto the platform and pass into the offtake.

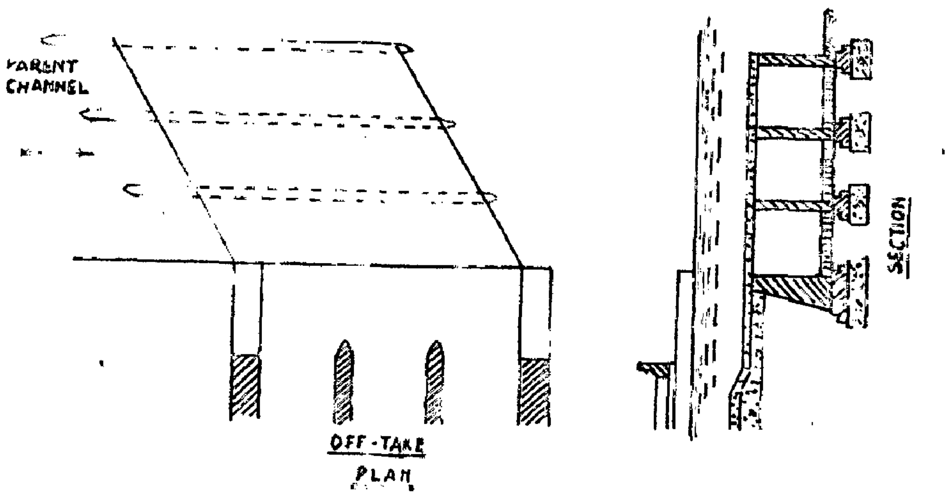
It will be seen that a compromise has got to be effected between conditions (a) and (c) on one hand and (b) on the other.

Width. Width of the platform should be sufficient so that enough water passes over it to fill the offtake at all times with some to the extent of 20 to 25% as spare.

Shape. As water approaches the offtakes in the form of a curve, the platform should be of lozenge shape [redacted]. but for facility of construction either its upstream or both upstream and downstream ends may be made square with the offtake, [redacted]. [redacted]

A smooth pitching should be constructed for a short distance upstream of the tunnels and in their bed to make the silt drop into the lower layers of water.

Piers. The number of piers to be provided should be decided from structural consideration. One thing is essential, that the piers



SILT PLATFORMS

should terminate with the noses sloping at 1 in 3. The top of these noses should have good cut water so that no rubbish gets collected.

Silt tunnels are useful in the case of small channels taking off from deep channels and where curved wing cannot be constructed for one or the other reason. The disandvantage of silt tunnels as compared with silt vanes is that as there is heavy silt charge passing at the edge of the parent channel just below the offtake, there is possibility of some deposit occurring which on accimulation may from a heap or block at the outfall of the tunnels. It is for this reason that this device is suitable for comparatively small offtakes ex-deep and large channels. Should the deposit render the excluder imoperative the matter may be improved by providing a few silt vanes to lead away silt to the mid stream.

G. OTHER MODIFICATIONS

Silt Platform-cum-Guide Wing.

The curved wing is added above the platform to guide top water into the offtake. The principles of design are the same as for tunnels except that as the quantity of water entrapped by curved wing for diverting into the offtake is not likely to slip off as with open platform provision need only be made for passing full supply discharge into the offtake at the lowest working level of the parent channel with no more excess than 10 to 15 p.c.

The advantage of the platform with guide wing over the simple platform is that on account of slight heading up caused by the curved wing a head of 0.1 to 0.15 is created at the offtake which increase velocity and prevents silt being deposited in the head reach of the offtaking channel.

Curved Wing with adjustable divider.

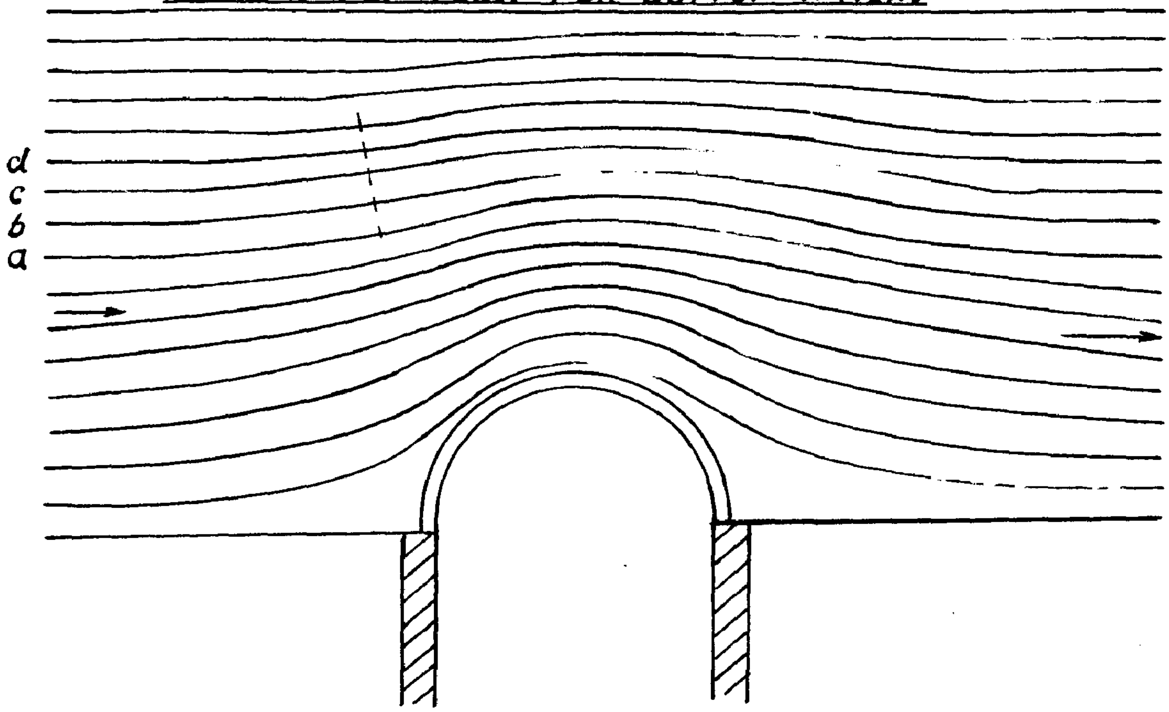
It is provided to control the quantity of silt entering the offtake.

It the divider DF is moved to the position DF' and as there are regulators across the parent and offtaking channels the same quantity of water passing through HF will be forced through HF' but more of bottom water will be deflected towards the offtake and if the divider is changed to position DF" more of top water will get into the offtake. It is specially useful in case where there is considerable variation in the discharge of the parent channel.

....2.

GIBB'S SEMI-CIRCULAR WALL :-

PATTERN OF FLOW FOR BOTTOM WATERS



UNFAVOURABLE CURVATURE

Top WATERS

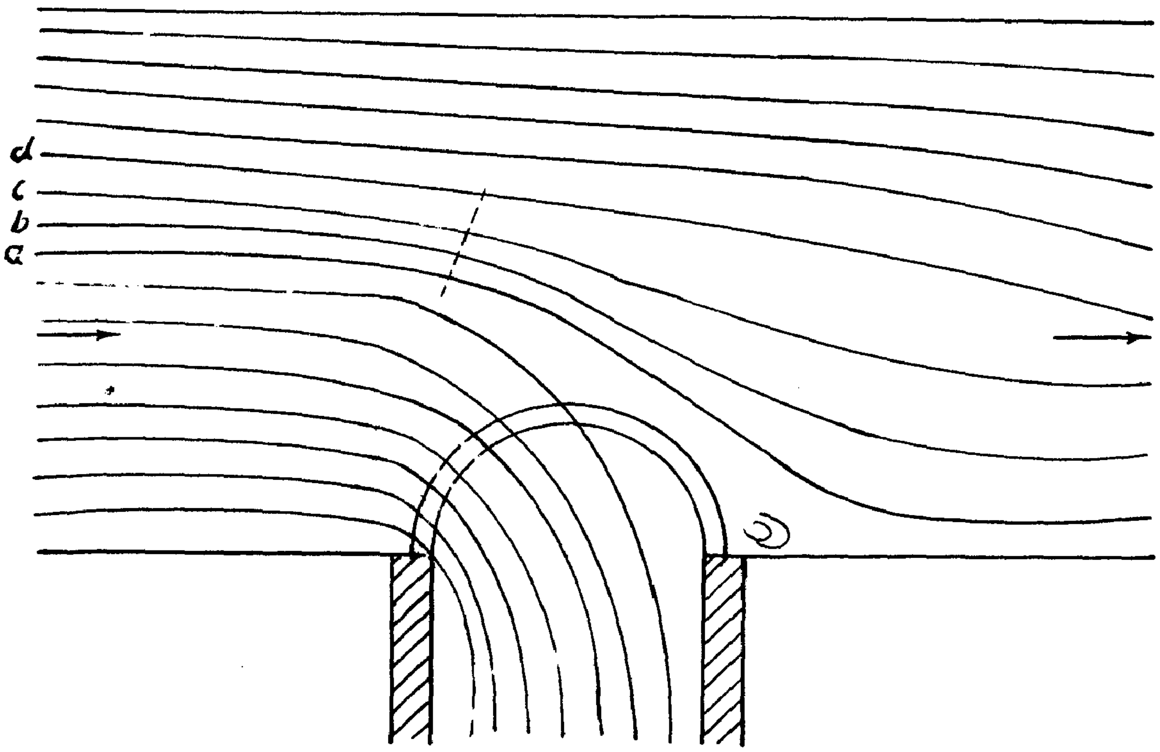


FIG. 23

Gibbs Semi-circular Wall:

This is a variation of raised sill with an additional disadvantage that as the wall is protruding into the parent channel it will obstruct the passage and throw up bottom water into the surface from where it will be drawn into the offtake.

Reverse Vanes.

If there is a channel bifurcating in two offtakes one of which silts badly and if there is not enough room to accommodate vanes, a simple way will be to construct reverse vanes in such a way that more silt goes into the offtake which does not silt.

CONCLUSION

It is seen that channels carry huge quantities of sediment load. In irrigation channels, these get deposited in the fields effecting their fertility or deposit on the canal bed reducing the cross sectional area. Suspended silt makes water turbid giving rise to weed growth. Fine silt, on the other hand, is beneficial for irrigation. The problem confronting the canal engineer is not simply exclusion of silt but its balanced control. The various steps to achieve this end have been enumerated in this paper. Starting from the control of silt at the origin and at Headworks down to measures of excluding and ejecting silt in canals and distributaries have been discussed. The basic principles of design and short reviews of work done on sediment transportation have been given. The procedure should be to exclude as much coarse silt as possible from entering the canal by measures at head of canal and to eject that silt which has escaped in by means of ejecting devices in lower reaches or in distributaries.

NEED FOR FURTHER RESEARCH

The designs of silt excluding & ejecting devices now employed in India are based on emperical formulae and experience. The rationalisation of these designs with the help of present day knowledge about sediment transportation is a ripe field for Research Engineers. Work on

Contd.

these lines is being carried out in the United States. In India this problem is of more significant and immediate nature. The design of works at Nangal is an example. Research can be based on the various lining materials used in canals in India viz; Concrete, Pre-cast slabs, stones, brick and earth. Work on setting time and setting velocities in channels with different linings and flow conditions and for different grades of silt can be carried out to be of practical use to a canal engineer. Sampling of silt in existing channels to study their behaviour can be made more useful by developing accurate and cheap methods of sediment samplers.

A series of laboratory experiments can be carried out to determine critical tractive force or critical velocity at which sheet movement starts. It is worthwhile investigating the quantity of material carried in suspension at various depths starting from as near the surface of water and going down as near the bottom as possible, the observations being taken while the water is in motion. The point where the detritus load starts should be noted. By plotting curves of Total (suspended and bed) load against depth for different grades of materials, it willbe possible to demarcate the boundary layer of the bed load. In addition it might also be worthwhile determining the viscosity of the samples taken at different depths viscosity plays an important part in the modern con-

Contd.

ception of sediment movement and it varies with the concentration of suspended sediment. The effect of the sides on movement of sediment load can be investigated by taking different widths of channels for above experiments.

The strength of a vortex required to uproot given category of bed load can be investigated. Little has been done in this regard.

The technique of model experimentation has made great strides and in India a large number of model investigations have been carried out but mostly on solution of specific problems. This has brought out broad principles involved, but an exact theoritical approach is still necessary to reach greater perfection.

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APPENDIX

TABLE SHOWING CANAL CHARECTERSTICS IN INDIA

Old Canal Systems

	Name.	Discharge cusecs.	Bredth feet.	Depth feet.	Slope	Remarks.
North:	1.Jammna Canal(West)	9,000	200	7.25	1/1000	Unlined.
	2.Sarda Canal	5,800	220	7.4	1/500	"
	3.Upper Ganges	7,180	140	10.0	1/2640	"
	4.Sone Canal main western	1,980	73	7.5	1/7040	"
	5.Upper Bari Doab	6,750	150	7.0	-	"
South:	Krishna Delta:					
	a)Western Delta main canal	3,317	230	8.5	1/19555	"
	b)Eastern Delta main canal	1,913	110	8.0	1/16000	"
	Godavari Delta:					
	a) East Delta Canal	3,205	184	8.15	1/16000	"
	b) Central Delta Canal	2,573	96	9.5	1/16000	"
	c) Western Delta Canal	6,266	170	10.7	1/16000	"

New Canal Systems

	Name.	Discharge cusecs.	Bredth feet.	Depth feet.	Slope.	Remarks.
North:	1. Bhakra-Nangal:					
	a) Nangal Hydel Canal	12,500	80	20.6	1/10,000	Lined
	b) Bhakra Canal	12,455	37	19.0	1/6,667	"
	2. Gandak (Bihar):					
	a) Main Western Canal	16,230	138	16.0	1/6,250	"
	3. Rajasthan canal:					
	a) Upto 3 miles	18,500	330	14.8	1/10,000	Unlined
	b) 3 - 134 miles	18,500	162	19.0	1/11,500	Lined
	c) Below 134 miles	16,065	307	13.6	1/12,500	Unlined
	4. Ramganga River Project:					
	Feeder Channel.	5,000	170	9.5	1/7,813	"
	5. Kosi East Canal:					
	a) Upto 2½ miles	15,000	380	12.0	1/18,000	"
	b) Below 2½ miles	10,500	260	11.0	1/19,000	"
	6. Chambal:					
	a) R.B.(Rajasthan)	6,880	52	18.0	1/3,800	Lined
	b) R.B.Canal (M.P.)	3,900	121	9.4	1/10,000	Unlined
	7. Tungabhadra Project:					
	a) High level Canal	4,000	65	12.0	1/10,300	Lined
	b) Low level Canal	2,500	72	10.5	1/10,000	"
	8. Hirakud Dam:					
	a) Baragarh Canal	3,992	145	10.0	1/10,520	Unlined
	9. Kakrapur Project:					
	Left Bank Canal	3,006	86	10.0	1/9,000	"
	10. Nagarjun Sagar Project:					
	a) Right Bank Canal	21,000	250	15.0	1/8,000	Lined
	b) Left Bank Canal	15,000	95	22.0	1/10,000	"

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PERMISSSBLE CANAL VELOCITIES, IN FEET PER SECOND (AFTER FORTIER AND SCOBEY)

Original Material Excavated for Canal.	Velocity, after Aging, for Canals Carrying.			Velocities According to Eqs. (23) and (25) for Clear Water	
	Clear Water, no detritus	Sater transporting colloidal clay.	Water transporting non-colloidal silt, sand, gravel, or rock fragments	Depth 1 ft.	Depth 10 ft.
Fine sand (non-colloidal)	1.50	2.50	1.50	1.00	1.50
Sandy loam "	1.75	2.50	2.00		
Silt loam "	2.00	3.00	2.00		
Alluvial Silts "	2.00	3.50	2.00	1.00	1.50
Ordinary firm loam	2.50	3.50	2.25		
Volcanic ash	2.50	3.50	2.00		
Fine gravel	2.50	5.00	3.75	2.50	3.50
Stiff clay (Very colloidal)	3.75	5.00	3.00		
Graded, loam to cobbles (non-colloidal)	3.75	5.00	5.00	3.00	4.50
Alluvial silts (colloidal)	3.75	5.00	3.00		
Graded, silt to cobbles (colloidal)	4.00	5.50	5.00		
Coarse gravel (non-colloidal)	4.00	6.00	6.50	3.50	5.00
Cobbles and shingles	5.00	5.50	6.50	5.00	7.50
Shales and hardpans	6.00	6.00	5.00		