SUMMARIES

OF

GOLD MEDAL PAPERS

(1912-2012)

PAKISTAN ENGINEERING CONGRESS

ON BEHALF OF PAKISTAN ENGINEERING CONGRESS

Pakistan Engineering Congress as a body does not hold itself responsible for the opinions expressed by the different authors in this Volume

Compiled and Edited
By
Engr. Ch. Ghulam Hussain
Vice President / Convener Publication Committee

Price Rs. 500/-
Members Free

Can be had at
PAKISTAN ENGINEERING CONGRESS
(4th Floor) Pakistan Engineering Congress Building,
97-A/D-1, Liberty Market Gulberg – III, Lahore 54660
(Near Liberty Roundabout)
Phone: 042-35784238, 042-35784235 Fax: 042-35784236
Web-site: www.pecongress.org.pk
E.mail: info@pecongress.org.pk
ISBN 978-969-603-019-5

PAKISTAN ENGINEERING CONGRESS
THE EXECUTIVE COUNCIL FOR THE 72nd SESSION

PRESIDENT
Engr. Riaz Ahmad Khan

Immediate Past President
Engr. Husnain Ahmad (President 71st Session)

VICE-PRESIDENTS

1. Engr. R. K. Anver
2. Engr. Ch. Ghulam Hussain
3. Engr. Ch. Muhammad Arif
4. Engr. Shabbir Ahmad Qureshi
5. Engr. Pir Muhammad Jamil Shah
6. Engr. Syed Mansoob Ali Zaidi
7. Engr. Dr. Izhar ul Haq
8. Engr. Syed Saleem Akhtar
9. Engr. Khalid Javed
10. Engr. Tariq Rasheed Wattoo
11. Engr. Ali Arshad Hakeem
12. Prof. Dr. Ing. Syed Ali Rizwan
13. Engr. Faqir Ahmad Paracha
14. Engr. Akhtar Abbas Khawaja
15. Engr. Muhammad Amin

OFFICE BEARERS

1. Engr. Akhtar Abbas Khawaja
   Secretary

2. Engr. Najam Waheed
   Joint Secretary

3. Engr. Khalid Javed
   Treasurer

4. Engr. Ch. Foad Hussain
   Publicity Secretary

5. Engr. Iftikhar Ahmad
   Business Manager

EXECUTIVE COUNCIL MEMBERS

1. Engr. Ch. Muhammad Rashid Khan
2. Engr. Anwar Ahmad
3. Engr. Nayyar Saeed
4. Engr. Najam Waheed
5. Engr. Iftikhar Ahmad
6. Engr. Malik Ata ur Rehman
7. Engr. Jamil Basra
8. Engr. Ahmed Nadeem
9. Engr. Ijaz Ahmad Cheema
10. Engr. Muhammad Ibrahim Malik
11. Engr. Prof. Zia ud Din Mian
12. Engr. Ch. Foad Hussain
13. Engr. Brig. Sohail Ahmad Qureshi
14. Engr. Liaqat Hussain
15. Engr. Shaukat Ali Shaheen
16. Engr. Ch. Aftab Ahmad Khan
17. Engr. Capt. (R) M. Qadir Khan
18. Engr. Iftikhar ul Haq
19. Engr. Rana M. Aslam Chohan
20. Engr. Malik Ahmad Khan
21. Engr. Syed Anwar ul Hassan
22. Engr. Zaffar Ullah Khan
23. Engr. Pervaiz Iftikhar
24. Engr. Syed Nafasat Raza
25. Engr. Muhammad Sharif Shah
26. Engr. Muhammad Sarfraz Butt
27. Engr. Sheikh Saeed Tahir
28. Engr. Syed Abdul Qadir Shah
29. Engr. Tariq Iqbal Mian
30. Engr. Faisal Shehzad
31. Engr. Ch. Sarmad Akhtar

TABLE OF CONTENTS

Sr. No.	Paper No.	Vol. No.	Subject	Years	Page No.
1.	32	V	Lining of Irrigation Channels By: J.A. Curry	1917-18	15
2.	51	VII	Experiments on Broad Crested Weirs By: F.H. Burkitt	1919-20	21
3.	65	VIII	A Few Aspects of the Punjab Road, Transport Problem By: K.G. Mitchell	1920-21	29
4.	68	IX	Water - logging from Irrigation Canals in Alluvial Soil By: F.V. Elsden	1921-22	35
5.	69	X	The Design and Construction of Light Suspension Bridges By: A.St.G. Lyster	1922-23	41
6.	81	XI	The Application of Modules in Irrigation By: E.S. Lindley	1923-24	47
7.	86	XII	Economic Railway Construction By: Major E.P. Anderson	1924-25	55
8.	94	XIII	Analysis of Partly Stiffened Suspension Bridge Type - 2F By: J. Halcro Johnston	1925-26	61
9.	102	XIV	Concrete Lining of the Gang (Bikaner) Canal By: C.F. Jefferis	1926-27	69
10.	110	XV	Report on Flume Experiments on Sirhind Canal By: A.G.C.Fane	1927-28	75
11.	125	XVII	Headless Canal Meters By : F.H. Burkitt	1929-30	81
12.	138	XVIII	Hydraulic Gradients in Subsoil Water Flow in Relation to Stability of Structures Resting on Saturated Soils By : A.N. Khosla	1930-31	87

Sr. No.	Paper No.	Vol. No.	Subject	Years	Page No.
13.	145	XIX	Construction of A Railway Bridge over the River Indus At. Kalabagh By : W.D. Cruickshank,	1931-32	93
14.	153	XX	Tunnelling in Connection with The UHL River Hydro –Electric Project By : G.H. Hunt, R.D. Kearne & N.V. Dorofeeff	1932-33	99
15.	162	XXI	Pressure Pipe Observations at Punjnad Weir By : A.N. Khosla	1933-34	107
16.	169	XXI	Silt Exclusion from Distributaries By: H.W King	1933-34	115
17.	174	XXII	Metallic Arc Welding as Applied to Bridges and Allied Structures with Special Reference to the North Western Railway. By: W.T. Everall P. S.A. Berridge	1934-35	121
18.	195	XXIV	Reconstruction of the Khanki Weir By: A.N. Khosla	1936-37	127
19.	197	XXV	Water-logging on the Upper Chenab Canal its Causes and Cure By : B.N. Singh	1937-38	135
20.	211	XXVI	Silt Excluders By : F.F. Haigh	1938-39	143
21.	215	XXVI	Reconditioning of Marala Weir By: E.O. Cox. R.B. Ganpat Rai	1938-39	151
22.	221	XXVII	Lining of the Haveli Main Canal By : R.S. Duncan	1939-40	159
23.	228	XXVII	Finances and Economics of Irrigation Projects By : Kanwar Sain	1939-40	165

Sr. No.	Paper No.	Vol. No.	Subject	Years	Page No.
24.	230	XXVIII	Remodelling Distributaries and distribution of Water to Areas Irrigated by Colony Canals By : A.W.M. Jesson	1940-41	173
25.	235	XXVIII	The Formation and the Reclamation of Thurlands in the Punjab By : M. L. Mehta	1940-41	181
26.	245	XXIX	Rainfall RunOff By : S.D. Khangar N.D. Gulhati	1941-42	189
27.	251	XXX	The Kalabagh Barrage By : S. I. Mahbub	1942-43	197
28.	260	XXXI	Lining of Channels By : S. I. Mahbub	1943-44	205
29.	261	XXXI	Construction of a Motor Road Round Simla By: W.A.R. Baker Balwant Singh	1943-44	213
30.	264	XXXII	Irrigation Outlets By : S.I. Mahbub N.D. Gulhati	1944-45	219
31.	267	XXXII	Chief Considerations affecting the Design and Usage of Railway Sleepers in India By : S.L. Kumar	1944-45	227
32.	290	XXXVI	Studies in Lysimeters By : A.G. Asghar H.S. Zaidi M.A. Qayyum	1949-51	233
33.	294	XXXVI I	Observation, Record and Analysis of Pressure Pipe Data of Weirs on Permeable Foundations By : Dr. Mushtaq Ahmad	1951-52	241
34.	300	XXXVI I	Engineering Planning for Industrial Development in Pakistan By : I.A. Zafar	1951-52	247

Sr. No.	Paper No.	Vol. No.	Subject	Years	Page No.
35.	307	XXXVI II	Construction aspects of Balloki-Suleimanki Link By : S. Allah Baksh M. Muzaffar Ahmad	1952-54	253
36.	315A 315B	XL	Studies on some Hydraulic Features of the Design of Taunsa Barrage By : Dr. Mushtaq Ahmad Abdul Latif Ch. Mohammad Ali	1956-57	259
37.	329	XLII	The Phenomena of Losses and Gains in the Indus River System By : S.S. Kirmani	1957-58	265
38.	339	XLIII	Dewatering of Foundation By : Dr. Nazir Ahmad Zia-ul-Haq	1958-59	273
39.	351	XLVI	Design of Alluvial Channels as Influenced by Sediment Charge By : Dr. Mushtaq Ahmad Ch. Abdul Rehman	1961-62	279
40.	352	XLVI	Artificial Cut-Off at Islam Headworks By: Khalid Mahmood Abdul Basit Akhtar	1961-62	285
41.	365	XLVIII	The Engineering Profession in Pakistan By: S.S. Kirmani	1963-65	291
42.	377	XLIX	A Study of the Effect of Suspension Parameters of Ride index of a Railway Vehicle and Results of Trials on the Pakistan Western Railway. By : M.Z. Mozaffar	1965-66	297
43.	390	L	Structural Investigations of Sukkur Barrage Arches By : Ch. Mazhar Ali	1967-68	303
44.	413	LIV	Panjnad Headworks after 1973 Floods By : Mohammad Aslam Chohan	1974-75	309

Sr. No.	Paper No.	Vol. No.	Subject	Years	Page No.
45.	456	LVIII	Construction of River Training Works on the Left Bank of River Ravi from Babu Sabu to Chung, Near Lahore By : Syed Mansoob Ali Zaidi	1981-82	315
46.	459	LIX	A Study on Dieselization of Sibi-Khost Section with GEU - 15 and GMU – 15 Group-IV Diesel Locomotives By : Mian Ghias-ud-Din	1983-84	321
47.	460	LIX	Improvement of Bearing Capacity for Foundations of Kotri Gas Turbines Extension Project By : Mohammad Rasheed Ch.	1983-84	327
48.	472	LX	Construction of Khairwala Drainage Project. By : Syed Mansoob Ali Zaidi	1984-85	333
49.	480	LXI	Transport Options for Developing Countries By : Mian Ghias-ud-Din	1985-86	339
50.	484	LXI	Remodelling Marala Barrage and Link Canal for Silt Control By : Mohammad Aslam Chohan	1985-86	345
51.	492	LXII	Combating High Sulphur in the Coal at Lakhra Power Plant By : Ghulam Murtaza Ilias	1986-87	351
52.	493	LXII	Estimation of Maximum Discharge for the Design of Hydraulic Structures By: Dr. Mushtaq Ahmad	1986-87	357
53.	511	LXIII	Subsurface Pipe Drainage Construction Methodology By: Engr. Javed Saleem Qamar	1987-88	363
54.	522	LXIV	Alluvial Channels Redesign Procedure By: Engr. M. Naimetullah Cheema, M. Husnain Khan, Tahir Hameed	1989-92	369

Sr. No.	Paper No.	Vol. No.	Subject	Years	Page No.
55.	541	LXV	Remodelling of Bambanwala Cross Regulator (RD 133296 U.C.C.) By: Engr. Usman Akram	1992-94	379
56.	543	LXV	Drainage of Irrigated Lands of Pakistan A Critical Review By: Dr. Nazir Ahmad	1992-94	385
57.	554	LXVI	On the Flood Frequency Analysis at Important Discharge Measuring Sites of Pakistani Rivers By: Syed Ali Rizwan & Muhammad Azam Chaudhry	1994-96	397
58.	565	LXVI	The Incidence of Rutting on Bituminous Roads By: Engr. Shaukat Ali	1994-96	403
59.	578	LXVII	Experience Gained from Interceptor Drains Installed in LBOD Stage – 1 Project By: Yawar Hamid	1996-98	413
60.	606	LXVIII	Effects Of Upstream Storages On The Present Eco-System In Areas Downstream Of Kotri Barrage By: Engr. Barkat Ali Luna Engr. Muhammad Jabbar	1998-2000	423
61.	650	LXIX	Failure of 2–Meter Dia and 54 Meters Long Piles no. 3/1, 3/2, 3/3, 4/3, 6/2, Cracks in Transom No. 6 and other Problems of West Channel Bridge Over River Chenab Near Chiniot By: Engr. Muhammad Iqbal Qureshi	2000-04	429
62.	651	LXIX	Performance of Subsurface Drains in Mirpur Khas Area of LBOD Stage – 1 Project By: Yawar Hamid, Irshad Ahmed Bohio	2000-04	437
63.	656	LXIX	Using Environment Friendly Finely Divided Materials In Brittle Matrix Composites By: Syed Ali Rizwan & Husnain Ahmad	2000-04	445

Sr. No.	Paper No.	Vol. No.	Subject	Years	Page No.
64.	662	LXX	Damages To The Right Pocket Of Sukkur Barrage And Emergency Restoration Works (2004-2005) By: Barkat Ali Luna, Malik Ahmad Khan, Ch. Muzaffar Hussain & Dr. Muhammad Salik Javed	2004-06	453

PREFACE

Ever since the inception of Pakistan Engineering Congress (established in 1912), 807-technical papers on diverse projects/ themes have been presented at its Annual Sessions and preserved in 71-volumes. In addition, 293 papers have been presented at various seminars and symposia on many diverse topics presented in 33-volumes. The Congress has the privilege of owning some of the original research and design papers which are in high demand and enjoy universal acceptance. In 1986, US Aid got reprinted 39-papers in four volumes on "Irrigation and Drainage" selected from the Proceedings of the Congress and distributed throughout Pakistan to Irrigation Engineers.

Presently, each year 20-24 papers are presented at the Annual Session and 8-10 papers at the Annual Symposium. In recognition of the contribution of the authors of the papers presented at the annual session which were adjudged best, the Congress initiated award of "Congress Medal" in 1917. Kennedy Medal was constituted in 1918 for the best paper in the field of "Irrigation and Drainage". Dr. Mubashar Hassan Medal was started in 1974 for the best paper presented at the Annual Symposium

In view of the rich technical value of these monumental papers, summaries of 52-papers were ably prepared by Engr. Ch. Mazhar Ali (now Engr. Dr. Ch. Mazhar Ali) were printed in a Book and presented at the 64th Annual Session (1992-94). The enlarged addition of this publication covering the period to date is now in your hands. Here it would not be out of place to acknowledge the valuable contribution made by Engr. Syed Mansoob Ali Zaidi, Vice-President (PEC). It is hoped that the readers would find the volume useful and interesting.

(Engr. Akhtar Abbas Khawaja)
Secretary

Paper No. 32
Year 1917

LINING OF IRRIGATION CHANNELS

By

J.A. CURRY

Paper No. 32
Year 1917

LINING OF IRRIGATION CHANNELS

By
J. A. CURRY

The object of canal lining is to substantially reduce the loss of irrigation water through absorption and percolation. The experiments conducted on Upper Bari Doab Canal, thirty years ago, indicate that the total loss of irrigation water in the main canal, branches, distributaries and water courses including wastage in the fields, is about 72% of the canal supplies. The use of administrative control to regulate irrigation supplies to the fields has resulted in reduction of losses in the watercourses and on field. The estimated percolation losses in a canal system, after some period of working, are about twenty five percent of the water entering at the head of the main channel. In the past, canal lining was aimed at preventing only a portion of total loss. However shortage of water in rivers, especially (luring winter, require that lining should aim at hundred percent efficiency in preventing all leakage from the channel. With the opening up of new canals in Punjab, which has a limited source of irrigation water, it is essential to make optimal use of available water. Another advantage of canal lining is the prevention of water logging, while the maintenance expenses would also be reduced.

There are some major construction problems related with the lining of existing operating canals. The construction activity could start only during a canal closure which lasts for ten to fourteen days. The working days get further reduced because almost one third of the closure period is lost while water is "running off'. In the case of a main canal, lining can only be done during a general canal closure whereas irrigation demand does not permit closure of the main canal every year for a period of exceeding 10 days. The non-availability of skilled labour and

short term arrangements for housing, food, fuel, transport etc., lay a substantial burden on the construction costs. Further, lining of channels is not the only effective method of controlling water logging as the supporters of tubewells assert that water logging can be better checked by pumping up the seepage water. Some engineers prefer seepage drains as a remedy for water-logging. There may be another reason to oppose lining: prevention of seepage losses would essentially require remodeling of the channel.

The cost analysis for a perfectly impervious lining and its comparison with the saving in irrigation water, crop incomes and long term benefits, show that an outlay of Rs. 20 per hundred sq.ft. of lining would be reasonable. The cost of cement cum sand plaster lining comes out to be Rs. 19.80 per hundred sq.ft. of plaster and it will be even less for established berms or by the treatment of cement and sand with patent material. This type of lining would require a mechanical mixer for a large scale project. The cost of materials for a 6 inches thick concrete lining is Rs. 12.5 per hundred square feet. This compares unfavorably with the rate for cement and sand plaster, but keeping in view the reduction in extra charges, the overall cost would be about the same. Slab lining has an advantage of rapidity of manufacture, cheapness combined with efficacy, but it requires specially designed watertight joints to be fully impervious. The cost of slab lining when employed on large scale is estimated to be Rs. 20 per hundred square feet. One of the cheapest lining materials is clay puddle costing Rs. 17.80 per hundred square feet. The execution of this type of lining is slow and needs careful supervision to ensure a uniform quality waterproof material. It cannot be applied in areas where haulage of the right kind of clay is expensive. The cost analysis of various types of lining has been based on channel water depths varying from 8 to 10 feet.

Lining for canals is quite expensive if they are to be kept 100 percent watertight. Depending upon site conditions a suitable lining can be selected keeping in view financial aspect of the project. If a lining is tobe laid below spring level, composite linings are ruled out, while a clay puddle lining is also objectionable. The only likely choice would be slab lining and it would be profitable for a channel whose controlling water depth is atleast ten feet. If a lining is to be laid above spring level, clay puddle lining would appear to be the most feasible provided that

suitable clay is available. On large canals, a slab lining or a cement and sand plaster would justify their adoption. In other cases if lining cost is somewhat more than the benefits of saving in irrigation water, even then it may be justified as a remedial measure against water logging provided tubewell pumping or seepage drains are otherwise uneconomical. The lining of new canals can be carried out at less cost and with less difficulty than for a developed canal. At the present moment, it is sufficient to remark that though experiments now show that truly successful lining operation may be expensive, yet when the value of water lost is considered, the expenditure of large sums of money to save such loss would in certain cases appear to be justifiable.

Paper No. 51
Year 1919

EXPERIMENTS ON BROAD CRESTED WEIRS

By

F. H. BURKITT

Paper No. 51
Year 1919

EXPERIMENTS ON BROAD CRESTED WEIRS

By

F. H. BURKITT

The author was calculating the afflux due to a proposed weir at Abazai on river Swat when it was suggested to him by Mr. Harvey that the weir would cause probably no afflux in high floods. Mr. Harvey's opinion was later confirmed by the author by observations during the construction of this weir. River Swat being very narrow at Abazai was further constricted by a ring bund round half the weir already constructed. The bund had to be made smooth and strong enough to withstand the high afflux expected during a freshet passing through the constriction. Considerable afflux was noted at the bund during low winter discharges but a large freshet twice as big as the maximum recorded winter flood passed the site without causing any significant afflux. The estimated afflux had indicated, however, that water would overtop the ring bund. Also on many subsequent occasions the author observed that a smooth obstruction in the bed or on sides of a stream causes a smooth depression in water surface downstream of the obstruction and since the loss of head is negligible in such cases, the water levels upstream and downstream are the same.

The proposal in 1917 of raising the crest level of Kalabagh weir to feed the new left bank Indus canal led to a difference of opinion over the possibility of increased afflux. The author's view that broad crested weir would probably cause no afflux (except for a small loss of head due to friction) in a big flood was not accepted. This prompted the author to conduct some experiments to establish the truth of his opinion. A minor near Mardan with a well fall of about 26 ft was selected for the experimental study. The earthen channel from head to wellfall was converted into a 2.5 feet wide flume. A six inches high weir having

upstream and downstream glacis slope of 1:5 and 1:15 and crest width of one foot was made to fit the full width of the flume. One side of the flume was finished with cement plaster to make a sharp edge which could be used for recording water levels with the help of a scale graduated to decimals. On the downstream end of the flume a provision was made to facilitate the regulation of water level for changing the depth of flow downstream of the weir. The channel below the wellfall in a length of 120 feet was properly shaped and closed at the end to serve as a measuring tank. The gate of the head regulator was suddenly opened and the depth of water at downstream edge of weir observed at regular intervals. The time taken to fill the tank for various depths was noted to compute the discharge. The analysis showed that the discharge could be represented by the relation;

$$q = y^{2/3} g^{1/2}$$

where g is the discharge per foot run and y is the depth at downstream edge of the crest: The observed discharges and those calculated from the equation were in close agreement. For a fixed discharge passing over the weir the downstream water level was raised gradually by means of karries at the lower end of the flume to form a standing wave which shifted up the glacis and then to crest as the downstream water was further raised till undulations replaced the standing wave, and the corresponding upstream and downstream water levels were observed to be the same. The observations revealed that a weir caused no afflux when flow attained a certain depth. A smooth depression in water surface replaced undulations with further raising of downstream water level.

These experiments mainly aimed at studying the effect of variation in water levels downstream of the weir, on water levels at the crest and upstream of the weir. The tail water levels were maintained so as to cover the range of flow conditions characterized by the following.

(i) Free overfall

(ii) Formation of standing wave

(iii) Undulations appearing in place of standing wave

(iv) Undulation replaced by a depression in water surface at the crest.

It was invariably observed in all the experiments that if standing wave was below the downstream edge of the crest, depth of flow at this edge was 2/3 (H + U^2 2g) where H is the head at crest and u is velocity of approach, and the velocity at the edge is $g^{1/2}$ $y^{1/2}$ The energy generated with increase in velocity over the weir was not entirely lost if water rose in smooth curve downstream of the weir to the water level upstream. Loss of energy however, was considerable with formation of a standing wave. Gradual expansion below the exit ends of siphons, culverts and bridges can be very helpful in recovery of head.

The experiments on broad crested weir led to following conclusions:

(i) when the standing wave forms clear of downstream edge of crest the depth of flow at the edge is given by relation 2/3(H + U^2/2g) and the discharge over the crest is maximum for a given upstream water level.

(ii) With depth of flow equal to 2/3 (H + U^2/2g) at the downstream edge of crest, the water surface after depression over the crest rises in smooth curve to the upstream water level.

(iii) There will be no afflux if the depth at the downstream edge of the crest is slightly more than 2/3 (H + U^2/2g).

When conditions of no afflux begin the critical depth at downstream edge of crest is given by equation.

$$y = c \times 2/3 (H + U^2/2g) \quad (1)$$

The value of c is 1.06 and remains constant except for very low velocity at crest such as 1.5 ft./sec for which value of x approaches 1.20. The maximum height x of weir which causes no afflux for given depth and velocity in a stream can be determined from the equation:

$$x = D (U^2 / 2g) = K. g^{2/3} \quad (2)$$

where D is natural depth for given discharge in a stream prior to construction of weir, x is the maximum height of weir over the bed, u is velocity of approach and k is a coefficient. For e equal to 1.06, 1.10. 1.20 the corresponding values of k are 0.473, 0.476, and 0.495.

If height of a weir is more than that given by eq. 2, height of afflux

above the weir crest is obtained by equation:

$$H^3:+ H^2 (2x-3/2. q_1^2/ g^{1/3}) + Hx\{x-3 q_1^2/ g^{1/3}\}-/3. x^2. q_1^2/ g^{1/3}+ q^2/2g=0$$

(3)

Where x is the height of weir, qi and q are the discharge intensities over the weir and in river upstream. In case of a weir with undersluices, discharge through normal bays is calculated to find q1 over the weir. If height of the weir x is greater than the value obtained from eq. (2) it will cause afflux. Both the undersluice and normal bays will discharge as broad-crested weir with free overall and depth of flow at the downstream edge of openings equal to their respective "two thirds", the afflux will be given by the following equation :

$$L1 (H - U^2/2g)^{3/2} =L2 (H + x + U^2/2g)^{3/2} + 1.8370/9 \quad (4)$$

where Q is river discharge, L1 and L2 are widths of weir and undersluices openings respectively, x is height of weir, H is height of afflux level above weir crest and u is velocity of approach.

Velocity being super-critical at glacis and sub-critical in the stream lower down, the standing wave forms at some point on the glacis or below it. Unwin's formula can be applied to find position of standing wave on horizontal bed but the author had to modify the relation for using it to determine the position of standing wave on the glacis. The depth of flow Ho just upstream of standing wave can be determined from following equation:

$$Ho^3 - Ho \{ (H1 -a) + 2q^2/gH\} + 2q^2/g = O \quad (5)$$

Where H1 is the depth downstream of standing wave, q is the discharge intensity and 'a' is the height form floor (where in is measured) to point on glacis where Ho is measured. Taking a few values of 'a' a graph for Ho can be developed. The standing wave forms where this graph intersects the water surface curve. It was found in the course of these experiments that water surface plotted by using Manning's formula is very close to the observed one. Sometimes instead of a standing wave of alone there appears in water surface a smooth wave of height (U^2

V^2)/2g followed by a standing wave. The author observed this phenomenon in his experiments but it is difficult to define conditions necessary to produce it.

Paper No. 65
Year 1920

A FEW ASPECTS OF THE PUNJAB ROAD TRANSPORT PROBLEM

By

K. G. MITCHELL

Paper No. 65
Year 1920

A FEW ASPECTS OF THE PUNJAB ROAD TRANSPORT PROBLEM

By

K. G. MITCHELL

Mechanical transport is the need of the time and more and better roads are necessary for the fulfillment of transport requirements. In addition to this, careful selection of road vehicle compatible to the quality of local roads will pay in the long run. The obvious advantage of road over railway is the free accessibility of the former.

In the assessment of the success of any proposed road, the appreciation of land values and revenues, the increased efficiency of administration, the reduction of crime and political enlightenment are the vital plus factors and must be considered. Commercial possibilities, probable effect on roads and possible future road policy are the subjects which are discussed in this paper.

Mechanical transport may be defined as the carriage of passengers and goods by mechanically propelled vehicles which can be steered unlike vehicles which run on fixed tracks. Bullock carts ruin the roads and need to be discouraged. The discussion in this paper is only on the commercial goods or passenger's vehicles capable of carrying one ton or more in the first case and ten or more passengers in the second. The petrol motor lorry, the steam truck and the steam train are the various types of mechanical transport vehicles. The steam truck and steam train tried around 1873 are now ruled out as unsuitable because of the heavy destruction caused to roads by steel tyres. Their design is based on wrong assumptions and ignores the well-known theory of three point contact. Of the remaining two choices, steam lorry may also be

excluded because of the delicacy of its machinery. Dirty or silt laden water can destroy or choke the boiler of the engine. The choice considered in the paper is, therefore, restricted to motor lorry only.

Good level roads, no interruptions, no competition from other modes of transport, steady and regular traffic in both directions, and lead of such a length that the lorry can do a single trip in a day, are the ideal conditions for the commercial success of mechanical transport. A combination of all such favourable factors is never found in actual practice. Running cost curves showing unit costs against capacities of lorry and against annual mileages are useful in the determination of the commercial possibilities. It is deduced from these curves that the optimal benefits can be obtained by running a 4-ton lorry, for 33 miles per day, for a 300 day year. These deductions may be termed as unduly optimistic as these ignore useless journeys and partial loads. However additional benefits, such as fixation of a trailer with the lorry etc., are also ignored. Thus these conclusions can be considered more or less realistic.

A specific case of Lyallpur and Jaranwala 25 miles apart, is considered in detail for the comparison of different available options of communications. This distance can be currently covered by railway via Chichoki Mallian (one train a day, fare Rs. 1-13-6. distance of 118 miles and time of 7 hours), or rather uncomfortably in two stages by turn-turn (direct distance of 25 miles, fare Rs. 1 and time of about 5 hours). These options were compared with a 4-ton motor lorry service which can carry 18 to 50 passengers charging about Rs. 1 per passenger. With one as stand-by, a daily service in each direction of three trips or 75 miles per lorry could be kept up. The total cost of running a lorry at about 2.5 annas per ton-mile would be Rs. 32,000 a year inclusive of stand-by lorry charges. The total earnings will be Rs. 40,500 a year at 75 percent of full load earning, which shows good profit. This example manifests the clear advantage and superiority of mechanical road transport over other alternatives.

For the comparison of different means of goods transportation, Lahore Amritsar route was considered. The results showed very little advantage of lorry service over results showed very little advantage of lorry service over railway. However, if there is a demand of rapid transit, the

lorry rates could be increased to get handsome profit.

The carriage of exportable agricultural produce to rail either by light railways or by mechanical transport was also examined. Mechanical transport has clearly more advantages as compared to light railway because of less initial investment. Money already invested in Mandis will be rather wasted if light railway is introduced because it would have to transship direct to the broad gauge line.

The wear on an ordinary water-bound macadam road is largely, about 83% due to attrition. Wear due to traffic is only 10 percent and due to weather 5 percent. Attrition is due to mutual rubbing between the component stones of the crust. Traffic causes direct wear by rubbing stone particles into dust. In dry climate, because of less moisture, water bound macadam is not good for mechanical transport. Therefore stone of good binding properties sprayed with tar increases the life of the road. Excessive camber in less rainy areas of Punjab also causes enormous wear. Bullock cart damages the road beyond repair with crown and shoulders practically unworn because the bullocks straddle the camber more comfortably.

A 40-maund bullock cart exerts a load much in excess than that allowed in England and the larger diameter of the wheel of the local country cart does not materially reduce the intensity of pressure. Furthermore, bullock carts are frequently out of plumb and cutting action of the edge of the tyre and wrenching action are highly destructive. This wear is greatly aggravated by its concentration on two narrow strips, resulting in ruts.

Abrasion, simple attrition, impact, compound attrition, surface shear and tyre wear are the general types of wear caused y mechanical transport. High speeds can enhance the effect of compound attrition. Sudden acceleration, deceleration Jr slipping of the driving wheels owing to road jumps can increase surface wear. Although speed is a ruling factor in road destruction, it is possible that the motor lorry can be made less destructive than the bullock cart without adversely affecting commercial efficiency. However as it is not always possible to limit the speed and axle weights of vehicles, therefore research is necessary for improving the construction of roads which can better

withstand the rigors of traffic.

Some of the possible types of future roads have also been discussed here. Presently, it is possible to reduce attrition by application of tar which makes a more permanent mortar with dust as compared to water. However further improvements are essential. An evolution from tar spraying through bituminous concrete to cement concrete is the cement grouting of water-bound macadam. But cement grouting is not much effective because the treatment can at best check attrition in the surface skin into which the grout may penetrate. The concrete road has proved a success in America. It has longer life and is cheaper to maintain. It offers a smooth surface for fast traffic, and can be given gritty finish by belting which gives sufficient foothold for horses. It requires less frequent closures for repairs and needs reduced annual demand for renewal metal. The comparison of water bound macadam roads whose normal life is 3 years, and concrete roads having a life of 15 years has shown that concrete roads are a better financial proposition in addition to other advantages.

Lastly it is stressed that there is a great need for the collaboration of local governments for the framing of specification of roads and mechanical vehicles. The continuous import of military vehicles for commercial use will lead us to destruction. It can be safely stated that there are many openings in Punjab for the mechanical transport provided good, economical and sufficient number of roads can be constructed.

Note:-

Paper No. 65 appeared at pages 101 to 132 of the Proceedings of Punjab Engineering Congress 1920, Vol. VIII. There are 2 Plates and further 10 pages of discussions.

Paper No. 68
Year 1921

WATER-LOGGING FROM IRRIGATION CANALS IN ALLUVIAL SOIL

By

F. V. ELSDEN

Paper No. 68
Year 1921

WATER-LOGGING FROM IRRIGATION CANALS IN ALLUVIAL SOIL

By

F. V. ELSDEN

The irrigation of land by means of canals has caused water logging in some of the areas which has made it exceedingly important to combat this hazard. In approaching this problem an investigator is handicapped by the lack of appropriate data reflecting the conditions of water table prevailing before the construction of canal. On the other hand a really effective and financially practicable remedy has yet to be evolved to prevent water logging.

The study of water table between confluence of two rivers shows that the spring level was, and in some places still is, below the level of rivers bounding the tract. Evidence indicates that in Punjab water level remained below ground level prior to construction of canals in spite of recharging from the rivers which suggests existence of outlet for subsoil flow in the neighbourhood of confluence point. Recharge from the rivers and rain contribute to the ground water table. During the rainy period spring level rises, but surplus of water will drain off by subsoil flow to restore the water table to the normal level during dry period. In some places where spring level is close to the ground surface, rainfall may cause water-logging temporarily and disappear after a dry period. The surface water is removed by surface drainage or by means of subsoil drainage into the rivers which form the boundaries of the affected area. The above two processes of drainage, assisted by evaporation and transpiration would remove the whole of the water added to the soil from all sources so that water table is kept at a depth-not injurious to crops.

The direction of flow of subsoil drainage is independent of direction of flow along surface drainage. Permanent surface water in any considerable quantity is likely to influence the direction of subsoil, but even then the direction of subsoil flow may be different from that of the flow in the surface channel. Therefore a canal crossing a surface drainage line has no influence on subsoil flow, provided accumulation of surface water is prevented. The alignment of canal is selected in such a way that it passes along the highest ridge of the central plateau and part of water adds to the subsoil water table. Similarly distributary channels, water courses, and field irrigation also contribute to the water table.

The introduction of canal system disturbs the equilibrium of subsoil water table resulting in rise in water table. Seasonal fluctuations due to rain fall may cause water logging in low-lying areas. It may not be overlooked that the canal would cause permanent rise in spring level resulting in water logging. The permeability of sub-soil beneath a canal bed vary from place to place and from stratum to stratum. The presence of impermeable stratum below a canal bed would prevent downward percolation resulting in a considerable rise in spring levels to constitute water-logging. In Punjab on Upper Jhelum Canal serious water-logging has occurred in lands adjoining some reaches of the canal due to this reason.

In earthen channels water escapes from the wetted perimeter by percolation through minute interstices of the soil. A percolation cone is formed in which water can flow under the action of gravity and capillary attraction. Flow takes place in a horizontal as well as vertical direction so that water fans out. Solid cones are formed beneath large canals and dispersed cones are formed under smaller channels or due to presence of impervious layer. Solid cone will obstruct the subsoil flow since it creates an adverse hydraulic gradient whereas dispersed cone cannot do so. Formation of solid or dispersed cone will depend upon the size of canal. The mathematical relationship

$$D = \frac{W(V-V')}{2V, \tan Q'}$$

where D,W,V,V' and Q are depth of soiled

Cone, width of channel, percolation velocities and divergence angle respectively, indicates that there is for every soil and for every depth of spring level, a limit to the size of canal which can be constructed without the danger of obstruction of subsoil flow. This limit will depend not only on the spring level which existed before the canal was constructed, but also on that which is likely to prevail after the canal has been in flow for a long period.

The spring level observations are recorded in Punjab at regular basis in the months of June and October. The study of this record for the years 1905 and 1919 in Lower Chenab Canal system shows that generally the spring levels have risen. Water logging has appeared between main canal and Kot Nikka Branch, but there has been no complaint in the area situated between the main canal and Gugera Branch. The spring levels between the Rakh and Jhang Branches may rise dangerously to cause water logging. Water logging will inevitably appear between Jhang Branch and the river on the right and between, Burala Branch and the Deg Nala on the left. The spring level lines observations are likely to be misleading close to large canals owing to the absence of close observations and the spring level profile would not clearly indicate the presence of well-developed percolation cones. Effect of percolation cones on subsoil drainage can be better judged by careful planning of water-table observations through sinking of bore holes or wells especially for this purpose at suitable locations. Generally smaller channels and large distributaries have little effect on obstruction to subsoil flow whereas percolation from large canals has a predominant role in causing water logging. Spring level and geological observations, prior to construction of canal, help in analyzing the real cause of water logging. The study of cross-sections of Ground surface cross sections may indicate places liable to water logging.

The experience of irrigation in Lower Chenab Canal system shows that volume of water contributed to the soil expressed in feet depth spread over the whole area from two distinct sources, rainfall and canal irrigation system, is estimated as 0.28 ft. and 0.78 ft respectively. After canal irrigation, an increase in the quantity of water added to the subsoil is of the order of 200 to 600 percent based on rainfall and irrigation intensity. Greater part of contribution to the subsoil is by main canals, and also significantly by water courses. Other parts of the canal system have relatively little effect.

The earlier methods of preventing water logging were to restrict irrigation in the area or by means of construction of surface drains. Reduction of intensity of irrigation or restriction of irrigation during Kharif period are to some extent useful checks against water logging, but the reduction in quantity of water added to the subsoil would not be sufficient to check water logging. It is also difficult to put it into practice due to the existing conventional system of irrigating the fields. The drainage works so far carried, out in Punjab have not been found to be effective as a remedy for water logging because the surface drainage system drains rainfall runoff only. An extensive network of surface as well as seepage drains, not so for attempted in Punjab is needed to combat water logging. Such a system of drains would be very costly, most troublesome to maintain, and even then it may not offer a complete solution to the problem.

The main cause of water logging is the canal irrigation system and can be controlled only by waterproofing the bed and sides of these channels, probably in suitable selected reaches, mainly at and near bifurcations of large canals. This may be the best remedy to check water logging, on the main line of Lower Chenab Canal. At present various lining material such as clay puddle, cement, bitumen are on trial, but an entirely satisfactory means of lining has yet to be evolved. Further research in this field is essential to find an economical and effective water proof lining. Lining of canals and water courses may also prove to be actually remunerative owing to the revenue and increased agriculture production which will be obtained from the water saved.

Paper No. 69
Year 1922

THE DESIGN AND CONSTRUCTION OF LIGHT SUSPENSION BRIDGES

By

A. ST. G. LYSTER

Paper No. 69
Year 1922

THE DESIGN AND CONSTRUCTION OF LIGHT SUSPENSION BRIDGES

By

A. ST. G. LYSTER

In the hilly areas of India, where the depth and velocity of the water in the rivers prevent the erection of centering, the importance of light suspension bridges is evident. The light nature of hill traffic governs the design considerations. Some of the design aspects in relation to construction of light suspension bridges which are not intended to carry wheeled traffic have been discussed in this paper.

Light suspension bridges are designed for live loads by pack animals and foot passengers. Maximum possible crowd load can be taken from the standards in Military Engineering, Part III A. The live loads depend directly on the width of the roadway which in turn depends directly on the density of traffic. In the Sutlej Valley, roadway is commonly fixed by considering that a single file of laden mules blocks it for all other traffic. This consideration is valid for such places where converging lines of traffic are not likely to meet at the bridge or where delay is of small importance. Live load assumption of 50 and 100 lbs per sq.ft. of roadway is reasonable for bridges of 3 ft and 6 ft roadway. Impact is accounted for by a factor of safety. The existing bridges, where no impact factor was provided, have a reduced factor of safety than intended by the designer.

Suspension Bridges are either unstiffened or stiffened. In the case of unstiffened suspension bridges, moving load is transferred to the cables by each suspender in turn. Stiffened suspension bridges have a mechanism of trusses through which moving load is transferred to the cables. This type of bridge may either have stiffened roadways or stiffened cables.

Main parts of the unstiffened bridge are: the main cables, the towers, the anchorages, the suspenders fastened to the main cables by strong clips which support the transoms, the road beams on the transoms connecting them to one another, floor of the bridge on the road beams, a hand rail on each side of the road and wind ties. The road beams are jointed at each transom in such a way that roadway becomes more or less a flexible chain. When the hanging main cables are loaded by a uniformly distributed horizontal load greater than the self-weight of the cable, it attains a parabolic shape. The analysis shows that the induced tension is minimum at dip-span ratio of 1/3, while dip is the vertical distance between points of support of cable and lowest point of cable. A lower dip span ratio of 1/10 to 1/15 is economical as well as safe against swing under wind pressure and deformation under concentrated loads.

Wire ropes commonly used for main cables are available under a variety of names in the market. The basic grade ropes are not good for shocks and severe bending stresses. For steel ropes with a tensile strength of over 90 tons per square inch, acid grade or special grade is recommended. The ropes with a tensile strength of 100-110 tons per square inch should be made of special acid grade steel for winding purposes and this grade may also be used for more or less stationary ropes. Locked coil or spiral construction are the best type of ropes but they are not quite flexible. The use of hemp core instead of wire core causes the rope to flatten at the saddles which opens up strands and increases the risk of rusting by contact with dampness; the towers are usually constructed of easily available bricks. The backstays should be preferably so arranged that the resultant pressure on the towers is vertical. A bearing plate for the main cables called saddle is bolted to the top of the tower. Saddle is generally designed to keep the resultant cable pressure on the towers at 90 degrees. To achieve this either the backstay or the tangent to the curve of the span must make equal angles with the horizontal or the tension on both sides of the saddles must be equal. There are various types of saddles of which the C. I. Rocker is the best because of its flexibility to move so as to equalize tensions. The use of C.I. Rocker also keeps verticality of the resultant pressure of the cables on the towers. Suspenders are fastened to the main cables by clips which may either be wire rope or steel rods. Roadway is usually

constructed of wooden beams and planks. The roadway of an unstiffened suspension bridge must be capable of transmitting bending stresses from one bay to the next. Camber in the roadway is provided to cater for its deflection under live load and the amount of camber must equal the expected deflection.

Unstiffened bridges allow vertical as well as horizontal movement of the roadway. The concentrated or partial live loads cause vertical movement whereas wind pressures cause swaying or lateral movement. Vertical movements can be controlled by stiffening trusses which are described later in this paper. Cradling, wind ties and horizontal stiffening are effective devices against lateral movements. By cradling the cables to an inclination of 10" to the vertical, the lateral oscillation due to wind pressure can be reduced to about one half but it also increases the tension in the suspenders and 'main cables by about 3.5%. Wind tie is used to fasten some point on the roadway (e.g., the end of one of the transoms) to a fixed point on the bank of the river. Wind ties can be either simple or continuous but both have some disadvantages. In the case of simple wind ties, the vertical component cannot effectively resist the lateral movement whereas horizontal component introduces a compressive stress into the roadway which is not desirable for an unstiffened bridge. This compressive stress can be eliminated by making the wind the continuous form of a parabola and placed in a horizontal plane. A wind tie having no component in the vertical plan offers no resistance to undulations of the roadway. A gust of wind can cause deformation of the parabola because the load is not uniformly distributed along the catenary. A horizontal stiffening truss designed to distribute partial loading can be provided as a remedy. In some cases inverted catenary guy ropes are used for stiffening of the structure. Inverted catenary is always cradled. It is also ineffective for the simultaneous resistance against lateral as well as vertical movement and hence must be regarded as dangerous.

Roadway can be stiffened either by inclined rods called stays or by trusses. Partly because of difficulty in a rational design of the stays and partly because they do not act in unison with suspenders, the stays are inferior as stiffeners as compared to the trusses. Stiffening trusses are designed to distribute loads uniformly along the cable thus obviating any deformation of the cable or roadway under partial loading. Truss is

usually fastened to the abutments at each end so that the reactions at the ends may be either positive or negative. The fastening allows the truss to expand longitudinally under temperature changes. Stiffening trusses can be analyzed by using Rankin theory. Some advantages can be achieved by providing a hinge in the centre of the truss. The truss is normally made of uniform section throughout. PWD paper No 51 recommends that the depth of the truss should not be less than 1/24th of the length. The design commonly adopted being panels with cross bracing is ineffective in case of timber where a lattice girder, which can be constructed easily, is quite useful.

The introduction of stiffness or rigidity into the able itself is relatively a new method. Some designers have focused on the origin of deformations and have preferred to induce stiffness in the cables so that the suspenders transmit the load directly to a rigid structure. In the stiffened cables the most suitable place for horizontal stiffening against wind pressure is in the plane of the cables. There are different options available for the stiffening of cables. Skilled labour is needed for the construction of such bridges. The stiffened cables do not seem to be advantageous for light suspension bridges.

Paper No. 81
Year 1923

THE APPLICATION OF MODULES IN IRRIGATION

E. S. LINDLEY

Paper No. 81
Year 1923

THE APPLICATION OF MODULES IN IRRIGATION

By

E. S. LINDLEY

In Punjab Irrigation System, the outlet discharge is generally influenced by the water levels in distributary and the water course. The heads of the distributary are generally manually controlled under the supervision of the irrigation engineer and the effort is to maintain a constant supply level in the offtakes fed through manually controlled regulator gates. The discharge of the offtake is calculated by using a discharge table or a curve. Considering the average regulating condition of Punjab Canals, the reported discharges of channels are generally incorrect and doubtful because of the errors in discharge measurements and changing bed levels due to silt movement.

The irrigation engineers are divided mainly in two schools of thought in regard to application of modules. One prefers the positive module for maintaining a constant discharge to the irrigators whereas the other favours the use of proportional semi-modules as the remedy for unsteady flow in the distributary. Experience has, however, shown that one system of moduling cannot be assumed to have a universal application. There are three classes of devices available for moduling of outlets, namely modules, orifice-semi module and flume semi-modules. The semi modules can be used for proportional distribution of irrigation water. Mr. Crump introduced the idea' of flexibility. Mathematical treatment shows that an orifice semi module with a setting of 0.3 and flume semi-module with a setting of 0.9 would have a flexibility of 1.0 which means that the outlets with these settings would serve as proportional modules. Flume semi-module is proportional over a wide range of fluctuation in discharge whereas orifice semi module is proportional only for a limited range. Flume semi-module because of its

sensitiveness to rise in water level in the channel due to bed silt or temporary obstruction is not at all a module. The orifice semi module set for low flexibility works somewhat like a module and it is nearly consistent in its performance.

In a canal system, excess supply may enter the head of a distributary resulting in rise of water levels at tail. Mr. Crump derived a formula to determine the excess supply at the tail based on flexibility of outlets. It is easy to realize that if all the offtakes were proportional modules, the percentage increase at head and the tail would be the same. Considering the efficiency of our canal system, regulation practice and cost of construction of modules, the combination of positive-modules and semi-modules works effectively for automatic control of supplies in the distribution system. Policy of providing suitable outlets must include the following points;

1. Determine the flexibility that will suffice if applied to the bulk of outlets.
2. Decide if special conditions make positive module essential for any of the outlets.
3. Provide proportional tail-clusters and, if required, flexible outlets in the tail reaches to act as safety valves for fluctuations finally ending up in the tail portion.

This policy was adopted on the Lower Chenab Canal of Jhang Division and it showed that orifice semi-modules traditionally set at bed level for effective silt distribution would require no more safety valve than proportional tail clusters. A good deal of outlets on which tempering was most profitable were replaced by tamper-proof outlets to achieve a fair degree of automatic control for rigid behaviour of outlets. The efficacy of that degree of flexibility could not be judged in other divisions, because the outlets were tempered with by the irrigators. The experience of moduling in Deccan brought out a successful use of positive module at head due to enormous fluctuation of supply level because of weed growth. This requires a considerable provision of flexibility in tail reaches and, therefore, exact control of head supplies. In case of a inundation canal, flume semi module as proportional distributor would satisfy the regulation demands.

On some Punjab Canals, traditional system of rotations and two different rates of design water allowances and discharges for summer and winter require rate able semi-module with low flexibility. In Deccan where each individual watering is fixed, rate able positive modules appear to be the only choice. A positive-module has to be rate able, otherwise a great number of sizes have to be built. For proportional as well as rate able semi-modules there is a need for changing the setting without discentaling the whole outlet. Various types of modules have been developed and applied in the irrigation system. Mr. R.G. Kennedy devised Gate Module with an adjustable orifice to give any desired discharge within a fairly wide range. These modules were replaced by other modules due to complicated operation for their satisfactory working. Wilkins module and Reflux module were manufactured but could not be put to use for practical reasons. Gibb Module consists of a semicircular open flume with a set of vanes. The trial of Gibb module on the Shahkot distributary was not satisfactory due to setting of the outlet close to bed level and provision of small modular range. The working of Kent "0" type module unlike the Gibb module is independent of the level at which it is set and is quite accurate. This module has been applied in water works, but its high cost would not justify its use in irrigation works. Venturi Module Sluice works -through gearing by a small reversible turbine. It requires mechanical knowledge for its operation as compared to operation of gate and its crab. This type of module would be useful for eliminating manual control.

The simplest orifice semi module is a planning pipe or outlet with a free fall, but it has limited field application. The Kennedy Gauge outlet is the semi-module which has been recently abandoned after haivng been extensively used in practice. This type of outlet failed due to incorrect setting frequent tampering by irrigators and imperfect hydraulic design. Discharge of the standing wave semi-module is independent of the water level in the water course due to formation of hydraulic jump. Mr. E. S. Crump developed a rateable standing wave outlet for proportional distribution on Upper Bari Doab Canal, but it requires further trial on other canals to prove its usefulness.

Board crested weir or standing wave flume also named as "Harvey Outlet" and "Khanewal Flume" essentially consists of a rectangular

throat section to control and measure the discharge, a bell mouth to cause flow to fill the throat without contraction or eddies and an expanding section in which-velocity head is recovered so as to make the minimum modular head as small as possible. The theoretical discharge is given by the formula $D = 3.09\ W\ (d + ha)^{1.5}$ where D is the discharge in cusecs and w, d, ha the throat width, the depression and velocity head. Further investigations are required to establish validity of this formula for the geometric profile of flume which includes surface slope length of expansion flume and the flume curves. Standing wave flume having rate able discharge with desirable accuracy are being manufactured in masonry.

It is essential to maintain records of the rate of flow to regulate the supplies in a canal system. Several types of water works meters such as positive piston type, semi positive type, and inferential type are not suitable for rating purpose because they measure volume only. Current or velocity meters which include Grant-Mitchell and Dethbridge meter have been successfully used in California and Australia. Rate of flow meters are suitable for Irrigation Works. Any gauge outlet, semi-module or broad crested weir can be used as measuring devices for the known water levels and the coefficients. For a wide range of flow, venturi meter gives a fairly accurate measurement of discharge.

The process of selecting an appropriate module for an outlet depends upon the availability of minimum working head and the upstream head. The chosen outlets are set to correct rating with the correct gauge reading at the average level of design discharge between silted and scoured bed conditions. The irrigators' complaints regarding outlet discharges are disposed off by checking the water level marks upstream and downstream of the module and to see that the accessories are functioning in accordance with design parameters. In actual operation the modules are tampered with, but it has been observed that Gibb Module, free fall pipe and standing wave are not easy to temper. However irrigators resort to smashing of these modules or tunneling through the masonry part. Vigilance on the part of a canal engineer can prevent loss of precious irrigation water.

The moduling of outlets by various types of modules have been practiced in Punjab. The proportional distribution with the use of semi-modules has certain disadvantages considering the convenience of canal

engineer than the needs of the irrigator. Positive moduling is the ideal from irrigator's point of view, but it has some disadvantages under practical conditions. Semi-modules used with low flexibility could satisfy the need of the irrigators. Moduling of outlets can be extended to the moduling of heads of minors by constructing meter flumes. The experience at Dhaular distributary system at Darshan head, Budduana head and Lakhbadar head shows that excess supplies can be distributed to the offtakes in a controlled manner to keep the variation factor within permissible limits. It is, however, essential to have meters at the head of a channel and at suitable intervals down the channel to achieve operational control of the system.

Paper No. 86
Year 1924

ECONOMIC RAILWAY CONSTRUCTION

By

MAJOR E.P. ANDERSON

Paper No. 86
Year 1924

ECONOMIC RAILWAY CONSTRUCTION

By

MAJOR E.P. ANDERSON

INCHARGE Committee have laid down in their recent report that new railway projects should be undertaken only if they could earn an average annual interest of 5.5%. The object stated for this policy is to provide cheap and efficient transportation throughout India. The present construction standards have to be improved to comply with "5.5% test" on new railway lines. Construction methods that can help to achieve economy along with required standards form the main subject of this paper. Economic construction is essential in view of existing high fares for the carriage of passengers and goods which cannot be raised further. The engineers must find ways to construct serviceable lines under present day conditions approximately at the cost level of first class lines applicable 15 years ago. The new construction approach is of immediate importance in many parts of India and nowhere more than in Punjab.

Optimum benefits with minimal expenditure are the fundamental requirements of economical design of the railway track. Speeds should be kept as low as 25 miles per hour which especially suite agricultural and mainly irrigated country side of Punjab plains. For an average train load of 850 tons, 14 or 15 sleepers per 36' rail length for a 60 lbs. rail provide adequate spacing. In case 75 lbs rails can be arranged from the main line renewals, the number of sleepers can be reduced to 13 per 36 feet rail in order to achieve economy. Ballast quantity of 10 cubic feet per foot run is enough for proper functioning of the railway track.

A relatively less expensive construction at serviceable standards can be achieved by constructing, as a first step, only what is absolutely

necessary. The remaining portions can be gradually constructed from the actual operational earnings. Specifications of bridges and buildings must be relaxed and modified to suit the needs of the time. Bridges on a railway line contribute substantially to the overall cost of the track. Need for a bridge must thoroughly be evaluated and should only be constructed if absolutely necessary. It may be wise to accept the possibility of holding up a certain amount of water for short periods or even a washout once in ten or fifteen years in the absence of a bridge. For essential bridges, cheap methods of construction may be employed even at the expense of durability. Cost of bridges can be further reduced by building them for a life of about 15 years. Heavy construction for a life span of 30, 50 or 100 year turns out to be uneconomical because the lines are always subject to changes for meeting the requirements of doubling the line or increased loads long before the end of this useful life. Later investment is always possible by saving money on compound interests. Enormous advantage of compound over simple interest is obvious. Present methods must be modified to follow the examples set by less developed parts of Canada, the United States and parts of Africa. The principle of sacrificing durability for the sake of economy also applies to buildings. The present expensive construction of permanent buildings should be replaced with the absolutely necessary structures and those too not in everlasting fashion, keeping the possibility open fur improvement when necessary in the light of modern scientific knowledge. The maintenance of such structures may prove to be a problem for engineers but better education of Indian Staff can solve this problem successfully.

The Author has worked out an example of an imaginary branch railway line; 50 miles long, with a 5'-6" gauge to practically demonstrate the cost comparisons associated with his different proposals already discussed. A total of five stations at an average spacing of 10 mile have been considered in addition to the junction. Two of the stations will at first be crossing stations, one the terminus and two will be flag stations with small goods sidings. The sharpest curve is 3" and gradient is 1/500 or easier. Bridges are required for air average 100 feet of waterway per mile at most.

For the purpose of cost estimation the preliminary expenses have been divided into sub-heads of land, formation, bridge-work, fencing, electric

telegraph, permanent way and ballast, stations, plant and general charges. Land should preferably be acquired in abundance which can later on be sold on higher rates thus getting enough revenues for future improvements. Formation should be made in such a way that it requires the least possible cutting or filling. For the construction of bridge-work, the old girders removed from the older lines in the course of strengthening may be preferred over timber. Well-seasoned timber resistant to white ants should serve the purpose and may prove to be economical. Fencing will not be required for a bench line like the one considered in the example. For the electric telegraph facility, wires and the telegraph instruments are all leased from Government Telegraph Department. Therefore, no money is required under the two heads of fencing and electric telegraph. Smallness of the siding accommodation at English roadside stations on lines with light traffic is a good example which should be adopted for Punjab country-side lines. Building work economy can be achieved by reducing the number of men to be accommodated, adopting cheap types of construction and reducing the present scale of accommodation per man. The last option is undesirable in view of the importance of having a healthy working environment and contented staff.

The economy in employing fewer men can be justified to achieve the object of earning an average rate of interest of 5.5%. For this purpose a graph between capital expenditures and rate of pay per month, successfully adopted in France during critical situation of man-power in summer of 1918, may also be adopted in this country. As evident from figures included in the paper it is possible to cheaply group all the points and the block instruments under the hand of one man without causing any delay to traffic as long as two trains at a time have to be dealt with. Sweepers and Bhisties do not perform whole time duties and live in villages usually situated close to the stations, therefore, no accommodation may be provided for them at stations. Construction cost of buildings may be reduced by the use of mud concrete or sun- 'dried bricks treated with silicate of soda. Possibility of timber building construction should also be explored for finding new cheap and efficient ways of construction. Platforms and signals are expensive luxuries and should be avoided for light traffic. Simple furniture and weighing machines should be provided at the .stations. Adding costs under these heads gives a total annual charge of Rs. 5285 in the

example. If weekly working expenses are Rs. 175 per mile the minimum gross earnings per mile per annum for meeting all the charges will be Rs. 5285+52x175 = Rs. 14385 or Rs. 288 per mile per. week. If old 75-lb rails are used, the estimated expenditure slightly reduces to Rs. 286 per mile per week.

Paper No. 94
Year 1925

ANALYSIS OF PARTLY STIFFENED SUSPENSION BRIDGE TYPE – 2F

J. HALCRO JOHNSTON

Paper No. 94
Year 1925

ANALYSIS OF PARTLY STIFFENED SUSPENSION BRIDGE TYPE – 2F

By

J. HALCRO JOHNSTON

An opinion was expressed in the annual session of Punjab Engineering Congress 1922 that bridges must either be stiffened in the orthodox Way or left unstiffened altogether. The author disagreed with this view and showed how stiffness of most of the bridges in the Punjab Hills depended on the rigidity of their floors. This paper attempts to give a solution of the general case where the moment of inertia of the stiffening system may vary from zero to infinity.

The method used in this paper is not new. According to the author of the "Framed Structures" the bending moment is not proportional to the load and the use of influence lines is, therefore, inadmissible. This objection is not practical. The bridge designers are relying more and more on influence lines. The author has restricted himself to the methods of drawing these. The special case of small Moment of Inertia or what is sometimes referred to as the stiffened floor has also been dealt with. No attempt has been made to treat the general case of continuous girder and suspended side spans. The practical example taken up in the paper is of a bridge with free ends at the towers and straight back stays.

The basic assumptions are: (i) the moment of inertia is uniform for all parts of the span, (ii) there is no bending moment and (iii) no end reactions are produced under dead load. Mean temperature has been assumed for the analysis. Thrust computation is based on the assumption that deflection is negligible which is same as assuming a large moment of inertia. Bending moment has then been worked out for

a single load as a function of thrust and deflection. Deflection has been eliminated from the moment deflection relation to yield and equation expressing moment as a function of the thrust, the position of the load and the position of the section at which the moment is required.

Thrust has been found by the method of least work. It depends on the work done in bending the stiffening girders, stretching the cables including backstays and that due to temperature strains. It must therefore be worked out independently for each case. The following solution allows for the work (lone in bending only and will generally give results within 5%.

Work done in bending the girder = $F = \frac{1}{2EI} M^2 dx$

Where

M is bending moment at a point clue to load W.
E is modulus of elasticity
and I is moment of inertia of floor
differentiating the above equation w.r.t thrust H, equating it to zero and putting $M = M^0 - Hay$

$$H = \frac{M^0 y dx}{a \; y^2 dx}$$

where

H = thrust due to load W

M^0 = bending moment in girder assuming no cable.

To find the influence line for H, assume a single load W and let $M^0 = Wam^0$

where

$m^0 = 1/2(1 - z)(1 + x)$ X < Z

and $M^0 - = 1/2 (1 + z)(1 - x) = 1.2 (1 - 2)(1 + x) - (v z)$; X > Z

and y = (1 - x2)/r

hence $h H/Wr = M^0 (1-X^2) dx$

$$(1- X)^2 dx$$

Solving the above equation we get

$$h = (5 - Z^2)(1 - Z^2) \times 5/64$$

This is the influence line of H.

In the case of a uniform load W over the whole span H = 1/2 Wra
the bending moment due to a single live load W is more accurately expressed as;

$$M = M - Hay - Hau$$

Where "au" is the deflection at P, H the total thrust due to all Loads dead and live. H is that due only to load W the terms due to dead load are left out as they cancel each other. The equation is rewritten as;

$$Wam = Wam- - rhayW - Hau$$

or $$m = m - rhy - Hu/W$$

From the above equation, the equation for bending moment influence is derived which is;

$$-m = A \sinh cx + B\text{Cosh}cx + D$$

A and B are constraints of integration and D = 2h/c
where $c^2 = a^2 H/El$

$$m= \frac{\text{Sinhe}(1-z)}{2c} \frac{\text{Cosh } cx}{\text{Cosh } c} + \frac{\text{Sinh } cx}{\text{Sinh } c} \frac{2h}{C^2} i- \frac{\text{Cosh } cx}{\text{Cos } hc}$$

$$\text{and} \quad m= \frac{\text{Sinhe}(1+z)}{2c} \frac{\text{Cosh } cx}{\text{Cosh } c} + \frac{\text{Sinh } cx}{\text{Sinh } c} \frac{2h}{C^2} i- \frac{\text{Cosh } cx}{\text{Cos } hc}$$

When c is very small, moment of intertia I tends to be infinite and the above equations become those ordinarily used for stiffened bridges. When c is large, I is small and the equations are those of the unstiffened bridge. In the former case influence lines for bending moment 'm' and shear 's' are given respectively by the following equations;

$$m = 1/2\ (1 - z)\ (1 + x) - h\ (i - x^2) \text{ i.e. } m^0 - rhy$$

and

$$m' = 1/2\ (1 + z)\ (1 - x) - h\ (i - x^2) \text{ i.e. } m^0 - rhy$$

$$s = 2hx + 1/2\ (1 - z)\ x < z$$

$$s' = 2hx - 1/2\ (1 + z)\ x > z$$

In the latter case influence lines for bending moment and shear are given respectively by the following equations.

$$m = a/2c\ [e^{-c(z - x)} - 4h/c]\ ;\ x < z$$

$$m' = 1/2c\ [e^{-c(x-z)} - 4h/c]\ ;\ x > z$$

and

$$s = -1/2\ e^{-c(z - x)}\ ;\ x < z$$

$$s' = 1/2\ e^{-c(x - z)}\ ;\ x > z$$

The suspension bridges have so far been divided into two distinct classes, stiffened and unstiffened. The latter have been used wherever cheapness was the primary consideration and the former where stiffness was essential. By the use of the preceding formulae it has become possible to design a bridge for any specified stiffness or depth of girder. To determine the economic degree of stiffness to be adopted we must

have a much clearer conception of the relative advantages and disadvantages of this property. The principal considerations will probably be the cost of stiffening girders when these are used and the excessive gradients and dangerous oscillations without them.

Paper No. 102
Year 1926

CONCRETE LINING OF THE GANG (BIKANER) CANAL

By

C.F. JEFFERIS

Paper No. 102
Year 1926

CONCRETE LINING OF THE GANG (BIKANER) CANAL

By

C.F. JEFFERIS

The "Gang" canal is being constructed from Ferozepure weir to irrigate an area of 755,000 acres of Bikaner State land, and would be concrete lined throughout its whole 84 miles length. It would be the biggest lined irrigation canal and the first of its kind in India. Its authorized full supply discharge is 2,144 cs. It has been designed with Kutters "N" of 0.013, and a bed slope of 0.13 ft per thousand feet resulting in mean velocity of 4.51 ft/sec. The designed bed width and depth are 32 ft. and 8.0 ft respectively with a side slope of 1:1. The quantity of concrete lining along the whole length has been estimated as 33.5 million square feet. Prevention of seepage and water logging was the main reason to adopt concrete lining. An estimated saving of about 400 cs has made the project economically feasible.

The possible ingredients of concrete included Darbari kankar, cement, lime and surkhie. Keeping in view the haulage, manufacture cost and quality of materials it was decided to use Darbari kankar or nodular lime-stone as the chief source for fine and coarse materials for concrete. Preliminary investigations were carried out on samples of Darbari kankar to determine the percentage voids in the kankar ballast and grit, the thickness of concrete for a watertight and cheaper lining, the density of concrete and the proportions of different ingredients. As a result of the investigations a 6-inch thick concrete lining composed of 1:1:6 mixtures of lime, grit" and ballast was adopted. Lime was preferred over cement because the latter was considered to the expensive and also the capability of the factories in the area to produce cement in the

required quantity was doubtful.

A plot of approximately one square mile was selected from the extensive kankar fields at Darbari in Bikaner for procurement of kanakar A network of 2-feet gauge tramway track along with specially designed binchutes was constructed for gravity filling of wagons with kankar resulting in large savings in cost. Two trains, each are containing about 16,000 cft. Kankar were dispatched daily. Mesh screening of kankar before loading to exclude dust resulted in 10% saving in carriage.

Manufacture of concrete and its carriage at site was planned in such a manner that machinery, lime kilns, godowns and stock of grit are placed in a limited space to save excessive carriage or handling. The whole length of the canal was divided into sections of about five miles long. In the centre of each section a "lining dump" was erected for concrete manufacture: The mixed concrete would then be railed out along the canal bed to the ramming face. There would be four such dumps in operation at one time. The kankar grit and ballast material would pass through a system of mesh and hopper for required grading and to mix in the correct quantity in the concrete mixers. The mixers are steam driven, with the mixing drum size of about 21 cft. concrete. This would be a convenient size considering the size of concrete Wagons of one cubic yard capacity. The kankar is burnt in the kilns -with railway refuse ashes. The lime kilns used are of the usual cylindrical continuous burning type, coned at the base and with two openings at the bottom from which the burned kankar can be withdrawn. All the machinery at the dump site is operated by steam 4NHP, 5NHP and 6NHP type engines. The haulage of the wagons in the dump area is done manually while the train loads of mixed concrete are propelled by light modern locos.

The excavation of the canal and dressing of the sides is done manually with care to attain a true level surface before concreting. Concrete is placed in position, is roughly dressed to a thickness of about 6.75 inches, and compacted by means of pneumatic rammers operated by three portable air compressors at each ramming face. The pneumatic types of rammers have been used for concrete compaction probably for the first time in India. Hand ramming was ruled out because of rather

restricted area of consolidation and difficulty in ramming side slopes. Pneumatic ramming has turned out to be more effective and economical, and the labour was suitably trained A normal lady output at the ramming face is about 130 rft. of channel.

Shape and spacing of the expansion and contraction joints was given due consideration. The most satisfactory and economical material for sealing the joints was found to be bitumen. The width of each joint and the number of joints was kept as small as possible. At first, "v" shaped wood form joints appeared to be suitable but at the' latter' stage "Y" form was found appropriate clue to its ability to be withdrawn easily from the consolidated concrete with the added advantage of less space left to be sealed compared with the "v" type of form. The type of work was entirely new, with no past experience as a guide to decide on appropriate spacing of contraction and expansion joints. The first arrangement was to provide two longitudinal joints on the canal bed, running parallel, and at a distance of 4 ft. from top of the side slope. Main cross joints were to be spaced 44 ft apart to run right across from top of the one side slope to the top of the other. Length of the joint per running foot of canal with this arrangement was approximately 4.6 linear feet. Provision of midway joints in the side slope was discontinued because appreciable cracks were not noticed after laying of the concrete. During the winter of 1924-1925 hail' cracks were observed in the bed slab and more serious cracks were noticed in the lining laid at the commencement of operation. In the work done since October or November 1924, practically no cracks have appeared either in the bed or side slope. The cracks on the side slopes of the earlier work were not found to be from contraction of concrete but clue to subsidence or other movement of the made up earth behind the concrete lining.

Due to a high content of lime, concrete made form Darbari kankar has different contraction and expansion characteristics from those of cement concrete. It takes three months to attain the maximum strength. Slow setting of lime concrete tends to relieve internal stresses and helps to adjust to the local severe weather conditions during the hardening process. Observation of completed concrete work has revealed no signs of expansion or contraction cracks. Friction between earth and concrete is sufficient to overcome any stresses near the bottom of concrete due to

temperature variation. In view of relatively low heat conducting properties of 6 inches thick concrete, expansion joints apppear to serve no useful purpose in preventing cracks in concrete.

After frequent observations and discussion it was decided to continue to provide joints because they help in localizing any failure in concrete that might occur from other causes. As a trial, it was decided to construct two long reaches of lining without any joints at all. Two other 1000 ft. long reaches were provided with the two longitudinal joints without any cross joints. Another short reach of about 250 ft. length was laid without any joints to serve as a reservoir of water for one of the dumps. Complete absence of cracks from the sides of reservoir even with the change of temperature from 80^0 F at water edge to 180^0 F at the exposed surface indicates that this type of concrete is not affected by variations in temperature. It is still more satisfactory that no cracks were observed on the weaker areas where natural earth joints should be abolished and the cross joints should be made at the end of half or full day's work near a bridge or a hydraulic structure, The question of sealing of joints if still under consideration and would be settled after watching the behaviour of concrete during current winter. If no harmful cracks are observed during the cold weather on the long lengths of concrete, it would be safe to simply grout the joints; otherwise bitumen would have to be used for sealing.

Paper No. 110
Year 1927

REPORT ON FLUME EXPERIMENTS ON SIRHIND CANAL

By

A.G.C. FANE

Paper No. 110
Year 1927

REPORT ON FLUME EXPERIMENTS ON SIRHIND CANAL

By

A.G.C. FANE

Flume experiments were conducted on Sirhind Canal with the objectives of finding out how far the modular limit of a flume is dependent on its geometric parameters, the coefficient of discharge above modular limit and whether the efficiency of flumes can be increased by altering the length of throat and shape of upstream wing walls without much increasing the cost. The efficient flume is one in which the modular limit is the highest, the coefficient of discharge below the modular limit is constant and above the modular limit is mostly uniform for varying heads.

Crumps developed a flume model "L" which has constant modular coefficient and high modular limit. This model is better than other models due to its higher throat length of about 2 times the head. A number of flumes based on this model were built on the Sirhind Canal, but Crumps dimensions were not adhered to.

Experimental flumes were built at Gill and at Bhowani. The flumes were 4' wide, with H equal to 2' giving a discharge of about 35 cusecs. The flumes had a throat length and glacis length of 2.5 H to test the flumes with a depth of water greater than 2'. Kari stop dams were provided above and below the flumes to control the discharge and water level downstream as also to achieve the desired drowning ratio. About 50 feet downstream of stop dam a tail flume with a free fall having the same dimension as that of test flume was installed to act as a meter fall. Calibration tests indicated that the coefficient of discharge Co below modular limit is 3.1. The original adopted procedure was:

a. Adjusting the upper stop dam for H = 2 feet over test flume with a drowning ratio less than 2/3 so that flume is running below its modular limit.

b. Recording of water levers at installed gauges.

c. Adding a Kari to the lower stop-dam so as to slightly increase the drowning ratio until the modular limit is reached.

During the experimentation stage it was observed that there was variation in results due to the fact that modular limit varies considerably for different values of heads. Therefore it was decided to adjust the upper stop-dam to make H = 2 feet and as soon as the modular limit was reached and H began to increase, karries were added to the upper stop dam and thus the discharge was reduced in order to keep H = 2 feet throughout the experiment. From the experimental results, flumes were divided into the following types:

1. Flume type A, glacis slope 1 in 10, wing walls diverging 1 in 5 or vice versa.

2. Flume type B, glacis slope 1 in 10, wing walls diverging 1 in 10.

3. Flume type D, glacis slope 1 in 10, wing walls diverging at 1 in 15.

4. Flume type E, glacis slope 1 in 15, wing walls diverging 1 in 10.

Other experiments were conducted to find the results of extending the glacis from 2.5 H to 3.75 H. It was found that in flumes of these comparative dimensions a glacis length of 2.5H is enough. Experiments were also carried out to observe the effect of increasing the throat length from 2.5H to 3.75H. The results show that an increase in length of throat upto 3.5H does not reduce the modular limit. This amended E type flume is called U type. The object was to have a flume which has a constant graph of C above the modular limit, for various values of H so

that it could be used as a meter. Further experiments on U type flume showed that an increase in length of throat makes the graph of C less variable and is independent of head `H'. In this flume the higher value of H gives slightly higher modular limit while in all flumes with shorter throat length including Crumps model "L" the reverse was the case. Calibration test for U flume 2 feet wide showed that for H = 2 feet the correct value of Co is 2.96. In this type of flume we get uniformity and approximate modularity with H up to 2.5 feet. The flume with a throat length of 2 H gave poor results as regards uniformity than with a flume with a throat length of 2.5 H.

Amended U flume type UF has wing walls with a combined radius of 3H and 2H. As regards modularity there is an improvement on U type. Further improvement in geometric shape of flume leads to P type flume. It has a crest length of 9 feet and upstream wing walls 4 feet in radius. Thus the length of throat and upstream wing walls in this flume is about 1.5' less than in the case of UF flume and it shows better results as regards modularity and uniformity. Practically speaking it is a perfect meter flume.

Crumps L flume with crest 4' long and upstream wing walls about 12 ft in radius had a throat 4.8' long and its modularity was distinctly better than 6" flume with throat length ranging from 4.8 feet to 7.5 feet. Therefore results which Crumps obtained without raised crest, do not apply to a raised crest flume. From this it appears that if flumes were made with high raised crest and more bottom contraction it would be more difficult to obtain modularity. It is, therefore advisable to adopt a setting of 9/10 which gives a flexibility of 1.0.

Finally these experiments on flumes 2' to 4' wide with H up to 2.5 and with a setting of 0.9 show that:

a. If the crest length is 2H the flume performs unsatisfactorily as regard modularity and uniformity and its modular limit varies with H.

b. For flumes of this type the modular limit is raised considerably by adopting a flatter glacis slope (1 in 15) and by diverging the downstream wing walls gradually (1 in 10).

c. A glacis length of 5' is enough,

d. Uniformity can be achieved by a throat length of 3 H.

e. A flume with a throat length of 3.5 H is practically speaking modular and behaves as a perfect meter.

f. It is better to have a longer throat length and sharp upstream wing walls.

g. Results obtained without a raised crest are not applicable to flumes with raised crests.

However there is a need for further research to obtain the relationship between geometric parameter of large flumes with the flow condition.

Note:-

Paper No. 110 appears in the Proceedings of Engineering Congress 1927, Vol. 15 at pages 37 to 51. It has 15 plates.

Paper No. 125
Year 1929

HEADLESS CANAL METERS

By

F. H. BURKITT

Paper No. 125
Year 1929

HEADLESS CANAL METERS

By

F. H. BURKITT

Majority of large channels in Punjab don't have a fall with a broad crested weir near the head thus making it imperative to measure discharge through current meters. The object of this paper is to show that the discharge can be obtained nearly as accurately as from a free fall, even without any appreciable fall, with the help of one or more pairs of gauge readings at a suitably designed masonry work. The theory is simple and based on the fact that in a stream with stream line flow, the difference of the squares of velocities at two sections close together is Zgh when h is the depression in water surface between the two sections, and g is gravity. Thus if the breadths are b_o and b_l and depths are y_o and y_l, then the discharge Q is given by the equation:

$$Q = 8.025\, b_o\, y_o\, b_1 y_1 \qquad \frac{h}{b_o^2 y_0^2 - b_1^2 y_1^2}$$

Such a meter was first built near the head of the Dipalpur Canal. In its design following requirements had to be satisfied:

i. Reasonable accuracy i.e. sufficient depression in water surface between the upstream and downstream gauges, for discharges ranging from 2357 to 7071 cusecs.

ii. The loss of head through the meter was limited to 0.2 ft.

iii. The velocity at the upstream gauge had to be well above the assumed critical velocity ratio to ensure that the area here would always be constant.

iv. The two sections where the gauge readings are taken were not to be so far apart that the effect of friction would appreciably affect the accuracy.

v. The contraction in plan was not to be as great as to make the convergence and divergence, upstream and downstream too expensive.

vi. On closing the canal, it would not be necessary to unwater the canal upstream.

vii. The spans were not to be too great for a reinforced concrete bridge.

viii. The intensity of discharge upstream and downstream of any span was not to be much different than that of its neighbours so as to avoid excessive eddies.

In a design the first four requirements have to be essentially met, whereas those listed at No. 5 and 8 are not so important.
The design of the meter consisted of 5 spans of 35' each, with 4 side spans having a raised crest 2.5' above bed, without any side contractions. The central span had its floor at bed level, its contraction in area being entirely in plan. In the side spans the length of the crest was made 15'. Gauge holes were situated upstream and downstream in each span, and their siting was so determined as to cause no interference from bending of streamlines. Gauges consisted of plain length of wood with brass tops were fixed in the gauge wells, and water surface levels were measured down from the brass tops with a boxwood scale.

To check the accuracy of the method, the results were compared with current meter observations. On an average, the discharge bridge gave divergence of 1.187 greater than current meter observations. It was noticed that generally the divergences were more on the windy days than on calm days, In the opinion of the writer, the Discharge Bridge appears to give results which are correct within about 1%, while carefully taken current meter discharges may be as much as 5% in error. It is because there are five possible sources of error with the current

meter namely (i) the length of wire (ii) the soundings (iii) the depth of the current meter (iv) the calibration of the current meter and (v) state of the weather. In the discharge flume, however, there is only one variable source of error: the gauge reading.

The next meter was constructed at the head of the Upper Sohag Branch. This consisted of a single span, 15 ft wide at the site of the downstream gauge with crest raised one foot, and 25' wide at the upstream gauge where the floor was at bed level. In this meter, the crest was all at one level, and a bypass was provided for unwatering the canal when necessary. The cross sectional dimensions of Upper Sohag branch are too large for its new requirements. When it would get silted up, there would be little or no fall at the site of this meter. During the past summer, the meter has been acting as a free fall and discharge can be measured from the broad crested weir formula as well as by the method described in this paper. The comparative results showed that the latter gave discharges which averaged 2.4% less than weir formula.

In this case and in this case of meter at the head of Jallalabad Branch, a new idea is being introduced which may prove useful for flumes in general. A divergence of 1 in 15 will, for the velocities with which we deal, recover all the head which is possible to be recovered, and a divergence of 1 in 10 may be nearly as good. A change in the direction of flow along the side walls is caused by water pressure at right angles to the latter. It should therefore be our aim to keep this pressure constant, and as the velocity of the water drops, the radius of curvature of the side wall should decrease. In the designs of Eastern and Jallalabad meters the initial radius has been made 200 ft. This type of divergence is cheaper than a splay of 1 in 10.

The following factors governed the design of the meter at the head of Jallabad Branch:-

i. There must be no appreciable loss of head with half share supply.
ii. There may, if otherwise desired, be greater loss of head with maximum supply.
iii. Great accuracy is not required with maximum as with half

share (one third of maximum) supply.

iv. Experience derived from Dipalpur Design shows that if we take Fane's coefficient, and a loss of head as shown by him of 0.2 ft, the result will give a design requiring no loss of head. Therefore we may start off by assuming a loss of head of 0.2 ft. with minimum supply. Various crest heights are tried along with widths at upstream gauge giving silt clear waters, and finally depression in water surfaces are determined. From the various alternatives the one combining suitability least cost is adopted.

Note:-

Paper No. 125 appeared at pages 1 to 14 of the Proceedings of Punjab Engineering Congress 1929. Vol. XVII. The author has given detailed calculations in Appendix. The discussions on the paper are given at pages 14a to 141. There are 9 Plates and 5 Diagrams.

Paper No. 138
Year 1930

HYDRAULIC GRADIENTS IN SUBSOIL WATER FLOW IN RELATION TO STABILITY OF STRUCTURES RESTING ON SATURATED SOILS

By

A. N. KHOSLA

Paper No. 138
Year 1930

HYDRAULIC GRADIENTS IN SUBSOIL WATER FLOW IN RELATION TO STABILITY OF STRUCTURES RESTING IN SATURATED SOIL

By

A. N. KHOSLA

The Author has noticed serious effects of the sub-soil flow on the upstream and downstream floor of some, of the drainage siphons on main line of Upper Chenab Canal for the first time in 1915 which caused boiling below the drop wall of Dugri syphon at RD 35800. Despite repairs and other necessary measures, piping showed marked increase in 1921 resulting in settlement of the right wing Wall cracks in the face wall and barriers. Provision of 20 feet deep sheet pile below the bed level also proved to be ineffective and damage extended to the upstream floor. Further measures included rebuilding of the upstream floor in cement concrete and constructing new side walls in 1923-25. The downstream wing walls were remodeled and placed on wooden piles and the downstream floor was strengthened. Springs continued to blow sand at the end of upstream and downstream floors and resulted in settlement cracks in 1928. The repair and extension work was based on a hydraulic gradient of 1 : 10 without considering the high spring level around the work. The object of the paper is to show the shortcomings of Bligh's Hydraulic Gradient Theory adopted for repairs.

Similar damages were noticed at Jauryan Syphon. Wing walls and face wall showed cracks with settlement of barrel lips in 1926. After a thorough examination of the alternatives 2 feet thick puddle did not reduce uplift pressures under the floor. The pressures were rather governed by sub-soil water level around the work. The floor of syphon was remodeled in 1929 but it was ineffective in controlling further

damages. A close study of the sub-soil flow conditions revealed that shallow end curtain wall was the real source of high uplift pressures. A sheet pile, 8 feet deep along-with strainers was provided at Dugri Syphon floor in 1929 and a similar sheet pile was proposed for Jauryan Syphon.

A flaw in the existing concept of flow through sub-soil emerged as the main conclusion of the investigations. An improvement in the principles for design of structures resting on saturated soil called for experimental work on scientific basis. Pressure pipes were installed at suitable depths at both the syphons to determine the uplift pressures with varying canal supply level and the spring level. Another objective was to find exact location of relief strainers, their contribution in reducing the uplift, and to ascertain the true free water level in the open bed against the apparent water level. It was observed that water level in the pipes recorded rise as the well points were sunk deeper indicating the existence of a relationship between the depth of a well point and the pressure recorded in the pipe. A number of observations were made covering depth below floor upto 23 feet. The normal spring line (NSL) was obtained by joining the pressure levels in the pipes embedded in upstream floor. To derive a general law, difference between NSL and indicated pressure in each pipe (strainers closed) was recorded for each depth of filter point. These results were found to be independent of canal water level.
The true free water surface is the level to which water rises in subsoil due to static head if there is no flow in vertical direction. Vertical flow commences when difference of static head exists, for example in presence of drain in which water level is lower than NSL. A pipe inserted in the path of flow would record water level below indicating a loss of head. Data was plotted for all the head losses to generate a "Loss of Head Curve".

According to Darcy, velocity of flow in sub-soil varies directly with head and inversely with length of flow. Darcy's relation, in form of Mathematical relation can be written as V= Ki = Kh/y:v is velocity of flow, K is the transmission constant, i is the fall gradient or head h divided by distance y. v =Kdh/dy. A curve plotted between h and dh/dy gave a straight line confirming the relation h = k dh/dy. It can readily be concluded that V = k"h where K"= constant, which means that velocity of flow at any point in sub-soil is directly proportional to loss of head

from normal spring level at that point the "loss of head curve" show that rate of loss of head increases as the depth below the surface" decreases. The velocity, therefore, increases as the depth decreases which implies an increase in the tendency for dislocation of sub-soil as the -depth decreases. There is a critical velocity for each type of soil particle and a velocity above it will dislocate the particles. Phenomenon of dislocation is vigorous at the bed level and decreases with increasing depth till critical velocity is reached where dislocation altogether stops. This depth is termed the "critical depth" and corresponding loss of head from NSL is called the "critical head".

Rate of inflow per foot length of strainer will increase as depth below ground decreases. The loss of head due to friction is greater in the presence of strainer as compared with sand column. The formation of springs leads to degradation of bed and consequent lowering of point of critical flow. If velocity and loss of head attain critical values at a certain point underneath the floor, the particles will get dislocated for some distance and cracks will appear in the floor due to settlement. As the process continues the degradation and consequent settlement accelerates and will extend to the face wall and endanger the main structure.

"Blowing up" or the uplift is caused by static head and "Blowing out" pressure responsible for undermining the floor is due to the kinetic head at a point. The sum of these two heads equals the drop from the normal gradient line to apparent water level in the drain. Reasonable floor thickness and thicker inverted filter provide a good combination to counteract both the heads for safety of structure. A comparison between performance of wells and sheet piles shows that wells do not provide an absolute cutoff like the sheet piles which are however, not self-supporting. Pressure is not normally built up upstream of wells due to presence of slits between them. Floor may be adequately reinforced to withstand stresses resulting from a possible building up of pressure on the upstream face of the end sheet pile. The strainers installed immediately below the sheet pile at upstream end are helpful in reducing the depth of the sheet pile. The strainers provide an additional safety to floor by disallowing the particle movement towards the open bed. It is desirable to provide sheet piles reaching critical depth and provide strainers for additional relief. The slit size must be very fine to

block the carrying away of fine particles for the sub-soil. The strainers may, however, be blanked off in the clay strata because fine day particles pass through the slits to initiate cavitation.

The design of structure on a given soil depends on the critical head for its particle the critical head for Punjab sand is 6 inches to one foot. For Dugri Syphon the critical gradient is assumed as 1:9 against critical depth of 11.4 ft. Further experiments are needed for determining the critical head and critical gradient for different types of soil. Sufficient observations are not available at present for establishing a general low for relating rise in the pressure pipes with increase of depth in vertical direction or distances in horizontal direction, Practical application of the phenomenon outlined in this paper has been discussed in a separate paper by the author.

Paper No. 145
Year 1931

CONSTRUCTION OF A RAILWAY BRIDGE OVER THE RIVER INDUS AT KALABAGH

By

W.D. CRUICKSHANK

Paper No. 145
Year 1931

CONSTRUCTION OF A RAILWAY BRIDGE OVER THE RIVER INDUS AT KALABAGH

By

W.D. CRUICKSHANK

The construction of bridge over the Indus at or near Kalabagh had been considered for many years. The first surveys were carried out in 1888 followed by investigation from 1919 to 1924. Finally in 1927 the project costing Rs. 40.36 lac was approved for construction. No provision for roadway was made. The bridge carried a single line broad gauge railway.

The bridge will connect the broad gauge (5-6") system of the Railway on East of the Indus with narrow gauge (2-6") to the West. With the completion of the bridges over the Chenab at Chiniot and over the Jhelum near Khushab, an alternative and direct route is available from Lahore to Waziristan. Commercially and strategically the bridge will play an important role.

The site of the bridge is about 1.25 miles below the gorge from which the Indus emerges into the plains. The course of the river at this site is stable. Character of the river bed at this site is such that from the left (Mari) bank half the width of the bed consists of an uppermost layer of fine sand covering a layer of coarser and sharper sand with small pebbles. The uppermost layers of sand disappear as the deep water channel is approached. Below this is a compact stratum averaging 45" thick of boulders set hard in sand. The bed of the deep water channel in the other half of the river consists of loose pebbles and boulders above the compact boulder bed. An alternative proposal of combined weir for

Thal Canal Head Works and Railway Bridge was earlier rejected mainly because the site of weir would be 4500' long instead of 2500' at the adopted site.

The Punjab Irrigation records showed that an extraordinary high flood occurred in the year 1878, which was calculated as from 757,000 to 770,000 cusecs. For the design of the bridge the maximum flood was taken as 8 lac cusecs. But there occurred in 1929 a flood of higher magnitude during the currency of work and the HFL at bridge site rose to 705.3'. It was estimated as 12,00,000 cusecs.

The design was changed accordingly, adopting 12 lac cusecs as peak flood discharge. Waterway provided initially consisted of 9 spans of 250' clear (263' centre to centre of piers). After the flood of 1929, it was decided that the bridge should be extended to cover the full width of the river between Mari and Kalabagh banks. This entailed an extension of the bridge at the Mari end by 4 spans of 175'-4" c/c. Girders of standard M.L. of 1926 (for 22.5 ton axle loads and a train of 2.3 tons per foot run behind the engine) are designed to carry a single broad gauge line of Railway. The live load is carried directly on an open flooring of cross girders and stringers by N type trough trusses with curved top chords and eight sub-divided panels, the maximum depth of truss being 30-7.5". The load is transmitted to the piers through knuckle bearings. Temperature and elastic extension is provided for by roller bearings at one end of each span, there being one pair of fixed and one pair of roller bearings on each pier.
Piers were initially designed to be made of concrete blocks but later it was intended to construct them of mass concrete 1:2.5:5 to avoid handling of blocks. Piers will rest on 2 feet thick 1:2:4 reinforced concrete bases keyed to top of wells. The maximum intensity of pressure at the base of the pier is 9.5 tons per square foot.

The wells were of twin octagonal type 38'-3" long by 22'.1.5" wide. The stein is 6 11.75" thick, leaving two circular dredging holes each 8 2" in diameter. The depth of wells fixed were those considered as probable safe depths. Deep water wells were taken 40ft into the boulder stratum leaving 36 ft of the well and pier exposed at high flood level. As some of the wells had to be sunk in deep water and as it was considered that pneumatic sinking would be necessary after the lighter soil had been

penetrated and the compact boulder stratum encountered, all well curbs took the form of caissons which would permit attachment of air domes and shafts for pneumatic work. Initially ten wells were proposed to be sunk. Bur after the addition of 4 spans, three more wells were added.

Position at the end of working season 1928-29 was that sinking of 5 wells was complete. The wells were plugged, R.C pier footing and 5 6" of pier masonry built. Sinking of 4 more wells was in progress whereas work on the 10th well was not commenced. The characteristic feature of the seasons work was the realization of the necessity for pneumatic sinking. It became evident to the contractor that open dredging in the compact boulder stratum gave very slow progress and was in fact impossible below a certain level. Therefore by March, 1929 the contractors had obtained a pneumatic sinking set. Precaution against scour was taken by protecting wells with pitching. This was necessitated by flood of 1929 which had caused tilting of wells.
Tenders for steelwork of girders were called for in England and India; while it was decided that substructure from the top of piers to the bottom of wells of the bridge should be given out on Lump sum Contract but, excluding the supply of caissons which were obtained from England by tendering. The substructure work was let out to a company from Bombay.

Note:

Paper No. 145 appears at pages 65 to 104 q of the proceeding of Punjab Engineering Congress 1931 Vol. XIX. It also has 8 Photographs and 18 Plates. Details of structure components and construction etc. are given in the main paper which may be referred to by those interested.

Paper No. 153
Year 1932

TUNNELLING IN CONNECTION WITH THE UHL RIVER HYDRO-ELECTRIC PROJECT

By

G. H. HUNT, R.D. KEARNE AND N.V.DOROFEEFF

Paper No. 153
Year 1932

TUNNELLING IN CONNECTION WITH THE UHL RIVER HYDRO-ELECTRIC PROJECT

By

G. H. HUNT, R.D. KEARNE AND N.V.DOROFEEFF

This paper deals with the Uhl River Hydro-Electric Project planned to provide cheap and better electric power for domestic as well as industrial users of Punjab. The site of this project is located in Mandi State, about 200 miles North-East of Lahore. The difference of levels between Uhl and the adjoining Rana Valleys is about 2000 feet. In order to utilize this difference to generate hydel power, diversion of Uhl water by tunneling through the ridge separating the two valleys is the only practical alternative available. The problems encountered in planning, design and construction of this tunnel are discussed. This power project shall be in three stages to produce a total of 120 MW. Work on the first stage is in progress, and this requires storage of 7 million c.ft in the Uhl. For the 2nd stage, a 250' high dam is proposed. The 3rd stage shall be within Rana Valley by using a 1200' drop and building another power house.

The main geological feature of the ridge is a wide cyncline. Composition of the ridge through which tunnel is driven consists of granite gneiss, mica schists, felspathic quartz gneisses, white and gneiss quartzite′s etc.

A base line 900 feet long was chosen for the start of work for layout of tunnel and triangulation right over the hill was carried out. The proposed tunnel centre line was connected to this line near the north portal and at surge shaft and tunnel exit. Vertical angles were read as a

check on double precise leveling which connected up the power station and the headwork's area.

A number of factors governed the exact location of tunnel. The tunnel intake would be downstream of the junction of the Uhl and Lambha Dag streams. The locations of the diurnal reservoir and dam site finally fixed the location of tunnel intake on northern side. Surge shaft and pipe: line location finally decided the position on the Southern side. Suitability for establishment of haulage facilities over the high ridge, level 8540, also played a role in selection of the tunnel site. Surveys indicated that a side entrance (adit) could be obtained through a nullah for expediting the work of tunneling. However in order to decrease the length of adit by nearly 700 feet a bend was introduced in the main tunnel and its length increased by 80'. Its total length is about 14000'.

Originally, the tunnel had been designed of circular shape to withstand heavy internal hydrostatic pressure the friction loss was calculated to be 2:22 ft. per 1000 feet with a friction co-efficient N of 0.014 for a discharge of 600 Cs at 9 ft. per second velocity in a tunnel of 9-3" diameter. -Circular section proved to be a failure in the areas of poor quality or, granite gneiss and a plain arch section was adopted in such areas. The gradient of 0.8997%, originally projected for the northern heading, was changed to 3.5% owing to the likelihood of large gushers of water being met in the granite gneiss. The gradient was fixed in such a way that the natural drainage system would work through the southern adit.

Special equipment and total power of 1080 K.W. had to be provided for construction. The compressors were located alongwith the sub-station plant at portal: two of these at north portal and three at south portal. Later on, one more was added at the southern side. Each compressor station was also equipped with a pair of blowers which were necessary for ventilation. Centrifugal pumps of 2" and 4" size were used for the water requirements of concrete work. The mucking machine worked with water taken from different pumping stations located along the tunnel site.

Size of the cross-section to be excavated, the nature of rock and type of

equipment available are important factors which determine the method of tunnelling. In this case as the tunnel cross-section was small and rapidly working mucking machines were available, the entire section has been drilled and blasted in one operation. For loose rock, a top heading method was adopted. Where mucking or haulage were interrupted by other work in the tunnel, a bottom heading had been used. Cement grouting proved successful for the areas of loose boulders.

The quality of rock being excavated fixed the depth and spacing of holes to be drilled for blasting. Jack hammers, weighing 55 Lbs, were used for almost the whole of the tunnel. Three face blasts per day were possible in 8 hours shift but this target usually could not be achieved due to other works. Pilot holes from 12 to 16 feet were drilled to locate the water areas in granite fissures. Gelatin dynamite (60%) commonly known as gelignite suitable for work in wet conditions was the principal explosive used with electric detonators in all headings. The state of rock in the south heading was not of good quality and therefore more over breakage occurred as compared to the north heading. Due to this fact, 100 percent excess quantity of concrete was needed for strengthening of roof. Usually no heavy blasting was permitted within 800 of lining in order to prevent damages to concrete.

As timber initially used for supporting the roof proved to be decaying quickly in underground conditions, it was replaced later on by steel frames and precast RCC slabs in all headings. This arrangement showed satisfactory results and its use was all the more necessary because timber should not be concreted in along with the lining.

Two Myers-Whaley Type-4 mucking machines for the Northern heading and one of the same types for the Southern heading were employed. These machines proved satisfactory in dry conditions but when considerable quantity of water was encountered, serious delays occurred. In the Southern heading, conditions for the use of these mucking machines were more favourable because of no accumulation of water and rapid haulage of muck out of the tunnel due to down ward natural gradient. Ten ton electric locomotives on narrow (2'-6") gauge rail track were used for haulage. It was found that there was very little difference in the cost per foot of tunnel excavated whether hand or

machine mucking was employed.

Different methods and rates were employed on different headings. On the Southern side rock was in shattered shape, therefore more difficulties were encountered as compared to those on Northern side. On the Northern side, all work was carried out by daily labour with some incentives of bonus. At the Southern heading ordinary muster roll system of payment plus some bonus was tried but because of the hard nature of work, monthly work orders system offered to contractors proved more workable. The rate of labour and explosives paid to contractor was Rs. 60 per foot run with a progress of 150' per month. An average progress of 5.4'per day has been maintained in the Northern heading with 3.6' for midpoint heading and 4.2' for surge shaft heading. The total cost of driving per foot run inclusive of timbering or steel setting and RCC slabs amounted to Rs. 145 for Northern and Rs. 178 for Southern heading.

In the design of a tunnel the forces which must be considered are; the pressure of the materials through which the tunnel is derived, the external water pressure, the internal water pressure and the stresses due to changes in temperature. Rocks deform under the influence of pressure in two ways, one of a permanent nature and may be termed as the plastic limit, and the other of a temporary nature and may be defined as the elastic limit Plastic deformations which are very dangerous in pressure tunnels may be minimized by preventing movements during driving which may otherwise develop very high pressures. it is, therefore, advisable to be careful in the removal operation of timbering and the quickest possible placing of lining. The risk of leakage is always minimum in those areas where external water pressure is considerable; therefore pressure tunnels should be located as deep as possible below ground level. Experiments have shown that generally, influence of variation in temperature does not reach more than about 10 feet beyond the lining.

The lining of the pressure tunnels should be capable of withstanding any external pressure which may be exerted by the surrounding rock. Therefore if all cavities are carefully filled, a circular concrete lining of one foot thickness can take any pressures likely to rise. Grouting of cement sand slurry was extensively applied to fill the cavities. Effect of

internal hydrostatic pressure can be minimized either by reinforcement of concrete lining or by high pressure grouting of the surrounding rock. In this tunnel after the main concrete lining, an inner layer of reinforcement was added which was covered by means of gunniting with cement sand slurry 1:2.5 with a cement gun. This system needed no centering and a high rate of progress was achieved.

The total cost per foot run of lining, excluding the mantle, of the surge shaft heading was Rs. 157 and of the mid-point heading Rs. 162, The cost per foot run of mantle in the Southern heading amounted to Rs. 160.

At the time of writing the paper, a tunnel length of about 1300 feet was still to be driven for the connection of two headings. The actual behaviour of the tunnel could only be seen after putting it in operation. The power supply operations were planned by 1933.

Note:

Paper No. 155 appeared at pages 35 to 76 of the proceedings of Punjab Engineering Congress 1932 Vol. XX. It has some photographs and 12 plates. For details of construction the interested reader may see the original paper.

The discussions and later developments are given at pages 76a to 76p. The two main headings met on 29.2.32 the discrepancy in level being 1/10 inch and in alignment 11/16 inch. The tunnel was tested in September and found to be water tight. In fact due to high external pressure there was an inflow of about i cusec.

Paper No. 162
Year 1933

PRESSURE PIPE OBSERVATIONS AT PANJNAD WEIR

By

A.N. KHOSLA

Paper No. 162
Year 1933

PRESSURE PIPE OBSERVATIONS AT PANJNAD WEIR

By

A.N. KHOSLA

The author had earlier presented two papers Nos. 138 and 142 in 1930 Session of Congress in which necessity of further research & observations on the subject was emphasized. This paper is a follow-up on those papers. Panjnad Headwork's was built with 33 bays during 1927 to 29. During 1930-31, 16 more bays were added on the right side on the recommendation of Islam Inquiry Committee of 1929. The additional bays were designed according to the conclusions drawn and recommendations made in the earlier papers. The construction work further provided an opportunity for full scale prototype research. In all 90 pipes were installed in new bays No. 43, 44, 46 and behind the two flank walls of the new weir (extension). Panjnad weir is unique as regards the location and installation of pressure pipes. The old bays (i-33) and extension bays (34-47) are divided by a Junction Groyne.

The section of the old weir consisted of 60.5 of pervious floor on the upstream, 110' of similar pervious floor on downstream with an impervious length of 204' in between ending in 5' deep top wall on upstream and 6' deep top wall on downstream. There is 30' deep sheet pile line under the crest. Another sheet pile 25.5' deep was provided at the downstream end of 26' length of loose blocks converted into semi-pervious floor. The "Extension" consists of 60.5' of u/s pervious floor, 163' of impervious floor with 20' deep sheet pile on the upstream end, a similar sheet pile 45' deep downstream of it and another at the end of impervious floor. This was followed by 20'.5 long concrete blocks over 2.0' graded filter, 20' deep wells and 100 ft. long loose apron.

The pressure pipes have been located at different depths on both sides of the sheet piles and at suitable position along the horizontal floor as well as in the subsoil underneath. The pressure pipes were located with a view of study the influence of sub-soil flow on the lines of flow and variation in uplift pressures under the structure and to determine the projection of sheet piles beyond the flank walls. The level at which the pressure in pipe is recorded is the level of centre of strainer in case of horizontal strainer and its top in case of vertical strainer. The underlying strata contains medium to coarse sand from top down, and at places coarse sand is mixed with Kankar or clay. The strata has however been assumed to be a homogeneous medium.

The water level in the pipes was recorded by either lowering a tape weighted at the end or a float suspended in a metal cylinder and the float on touching the water surface lighted a lamp placed in the circuit. The first method was very crude and the second was some-what complicated. Another device was therefore resorted to; this was a bell sounder consisting of a brass rod of 7/8" in diameter, 3.5" long ending in an inverted cup. The sounder is lowered by means of a steel tape. The cup produces a sound on touching the water surface. This method gives accurate readings upto 1/16th of an inch.

The Panjnad canal was run for the first time in April 1932. The observations were started with the first ponding and continued upto October, 1932 when pond was released. The second series of observations were done when pond was again raised partly. The observations were repeated during lowering of the pond. Pressures were also recorded at 15 to 30 minutes intervals with rapid raising or lowering the pond to determine the time lag in various pipes. The change in the relative drop of pressure in certain pipes for the same head between the observations of the two series may be attributed to rapidly changing pond level, its low temperature and varying depth of silt on upstream pervious floor.

The observed pressures were plotted along the shortest distance between bottoms of end sheet piles and against the creep line. The pressure line joining the pipes located vertically below the upstream, middle and downstream sheet piles represent the normal sub-soil flow under the weir. Analysis of data from piers No. 43, 44, 45, 46 bays 46,

23 and 30, the right flank and junction Groyne indicated that the ratio of pressure against total head (p/H) for any pipe is constant, pressure variation in the vertical direction at the upstream and downstream ends of floor is either logarithmic or parabolic and linear in horizontal direction under the entire floor. A vertical obstruction at the end or in the length of impervious floor not only deflects the stream lines but also changes the pressure distribution. Two pipes symmetrically placed between two upstream sheet piles (extension) always showed constant difference for liner range of observations irrespective of head and temperature changes.

For variation of pressures at the upstream end of floor, hyperbolic curves show a better fit if there is no silt deposit on U/S floor. At the downstream end both the logarithmic and the hyperbolic curves are equally good.

Bose had mathematically calculated that (Discussion on Author's paper 138, 1930) pressure drop on either side of an intermediate sheet pile is equal this is approximately correct. In extension bays, there is a total head loss of 6.8% at the 20' intermediate sheet pile out of which 3.8% is on U/S. The sheet pile at U/S end of floor is responsible for 50% head loss whereas similar sheet pile at D/S end gives a loss of 20% of total head. The intermediate sheet piles merely serve as a second line of defence in the eventuality of disaster and is otherwise not very effective. The plot of downstream pressures shows that residual head at exit end of floor is 1.12 i.e. 7% of total head of 16.4 and increases to 1.37 for a head of 19.5 for which the weir is designed. It therefore seems necessary to conduct experiments to determine the safe residual head with and without inverted filters. It is noted that in case of Panjnad weir at downstream end the pressure and velocity decrease in the direction of flow i.e. upward, which is unlike the phenomenon at Dugri Syphon (Paper No. 138).

The scanty data available from the three pipes inserted in each of the bays 23 and 30 afford an opportunity to compare the behaviour of two sections of the weir. The uplift pressures below the gate line of old weir are lesser by 1.3 to 4.1% but the residual head is higher, 2.46 for total head of 19.5' as compared with extension this could cause piping through relief pipes and should be plugged to avoid considerable

damage expected from their operation. The stream lines are deflected from vertical to horizontal by 31' in the old I weir and by 26' in "extension". It shows that the removal of entire U/S floor of original weir will cause a small change in uplift pressures. The pressures can improve if the 30' sheet pile under the crest is moved to under the U/S end of U/S glacis. The data further indicates that if 40' long upstream floor end one sheet pile could be omitted in extension, resulting increase in pressure below the gate line will be 1. to 5%. A single 30' sheet pile is better than two sheet piles of 20'. The intermediate sheet piles of 20' and 30' give a drop of 6.6% and 10.6% respectively. The length of upstream floor can be considerably cut down without causing significant increase in uplift pressures on rest of the impervious floor.

Analysis of pressure pipe data leads to the following conclusions:-

1. The flow of water under the weir is streamlined and obeys laws of hydrodynamics for flow of very viscous fluids.

2. For any point, the ratio (P/H) is constant but silt deposit on pervious floor and temperature changes may influence it.

3. The pressure variation in vertical flow outside U/S and D/S ends is either hyperbolic or logarithmic. The velocity is maximum where water enters the subsoil and minimum at the exit end.

4. The rate of pressure drop along the horizontal floor between U/S and D/S pile lines is constant and bears a linear relation with distance. The Bligh Creep Theory is applicable only for this distance.

5. Head loss (51%) is maximum at U/S sheet pile (20') in extension.

6. A head loss of 20% is obtained with 163' long floor in extension.

7. A head loss of 21% is obtained due to downstream sheet pile.

8. Out of 28% head loss contributed by the horizontal floor, only 6.6% is due to intermediate sheet pile. The 30' intermediate

sheet pile in original weir gives a local drop of 10.6%.

9. In extension the residual head at the exit is 1.36' for maximum head of 19.5'. It needs provision of inverted filter.

10. Correct knowledge of subsoil flow can help in achieving considerable economy in large hydraulic structures by cutting down the length of U/S floor and omitting superfluous sheet piles.

11. A 30' pile line is 50 to 75% better than two 15' pile lines because it is only the upper pile line that makes the major contribution.

In a large hydraulic structure, as a rough rule the depth of end pile lines should be equal to total head for the structure but in river works not less than 20' in any case. The depth of D/S pile line should be adequate to result in safe residual head at the exit end. On an average, the percentage loss of head at U/S sheet pile may be taken as twice the depth of pile line and that at the D/S sheet pile the same as the depth of sheet pile. The floor length should be enough to dissipate the balance of head assuming the gradient between 1/20 & 1/30 (for sand) and with due consideration for standing wave and retrogression of river bed.

This investigation aims at presenting comprehensive but simple formulae for design of weir, which conforms to laws of hydro dynamics. It should be a policy to install pressure pipes in future in all the hydraulic structures in consultation with the research authorities.

Note:

Paper No. 162 appeared at pages 50 to 88 of the Proceedings of Engineering Congress, 1933 Vol: XXI. There are 5 appendices and 13 plates giving details of pressure pipes.

Paper No. 169
Year 1933

SILT EXCLUSION FROM DISTRIBUTARIES

By

H.W. KING

Paper No. 169
Year 1933

SILT EXCLUSION FROM DISTRIBUTARIES

By

H.W. KING

The paper deals with the subject of silt entry into irrigation channels. It describes different methods of exercising a check on silt entry in a channel where enough water is available to carry away the excluded silt. According to Dupuit silt carrying capacity of flow at any point depends upon the difference in velocity of filaments of flow just above and below that point. A small obstruction over the bed keeps throwing up the silt which falls back and is thrown up again. This motion of silt particle is termed siltation, and depends on bed roughness, velocity and particle size. For a given depth and mean velocity, a channel with rough bed can transport more quantity of silt than one with smooth bed although the smooth bed is capable of transporting silt of coarse grade by rolling. The lower layers of water contain coarse grade of silt and the particles moving along the bed are known as "rolling silt". The smoother the bed of the channel the greater is the depth of rolling layer to carry a particular grade of silt. Silt excluders are devised on this principle.

The proportion of silt charge in an off-taking channel is more as compared to silt charge in the parent channel because, firstly the lower layers of flow in the parent channel containing the coarse silt are easily deflected into the off-taking channel and the upper layers having high momentum pass by. Secondly, water enters an off-take in a curve and because of higher free level on outside of the bend; water at the bottom must be having a tendency to carry gravel, sand etc. inwards. Thirdly, there are cross currents generated by the obstructions on the sides. With a straight parent channel, a channel off-taking at an angle can be made to draw smaller proportion of silt if the off-take orifice is kept at mid-

depth of parent channel whose bed is thoroughly roughened by constructing transverse low walls over the bed in full width. The Placing of low walls can preclude the possibility of throwing up of lower layers of water entering the off-take.

Common causes of excessive silt entry into off-taking channels are: head-regulator projecting into the parent channel, bushes and stakes in parent channel, uneven berms just upstream of the off-take and a cill in set back position. In case of channels off-taking at right angle, the head reach often silts up, thereby regenerating a new slope to carry the silt charge. A skew head or "curved wing" can provide the solution in these conditions. Research by Kennedy and Lacey confirms that deep channel can carry less silt charge. Lacey's 'f' decreases with increase in depth and depends on particle size. The influence of rugosity of bed on mean velocity on a vertical line from the bed is more marked in shallow than in deep channels. For a given mean velocity, velocity of water near the bed in a shallow channel is greater than if the channel is deep. Scour takes place as the silt is thrown up by irregularities on the bed and in addition the bed material is picked up by the vertical eddies. Likewise rough and uneven bank is likely to be eroded more rapidly than a smooth and well-dressed bank. Experiments on the Lower Chenab Canal supported the above submissions.

The Author concluded after his experiments that silt vanes and silt vanes-cum-curved wing proved to be the most efficient arrangement of all the devices tested. The use of silt vanes is not benefitting if an off-take draws more than one-third discharge of the parent channel. The parent channel needs steeper slope downstream of the vanes to carry higher silt charge. Silt vanes may cause scouring in an off-take but poorly designed vanes built at a wrong location can aggravate the conditions.

The efficiency of the vanes increases with larger radius. A radius of 40' or so is desirable for short vanes in a small or a medium channel but should not be less than 23' radius. The downstream end of the vanes should be tangential to lines making an angle of 27° with the channel. The vane nearest to the off-taking channel (longest vane) should not be within the influence of too strong "draw". The space occupied by the upstream end of varies should be about half the width of parent channel and height of vanes is ordinarily kept at 1/4 the depth of parent channel.

Thinner vanes are more efficient. The spacing between the vanes should be 1.5 times the height of vanes, and the surfaces should be smooth plastered. Bed and side slopes of the channel on the side of off-take must be pitched in a distance of 50' to 100' upstream. A short distance downstream is also pitched. The upstream end of vanes must slope at 1:3 and be finished in cut water shape.

The channel must be free of all obstruction in a distance of 200-300 ft. upstream of the vanes. The cost of construction can be reduced by either reducing the radius of vanes or reducing the number by increasing inter-spaces. Omission of plastering the vanes and pitching considerably reduces the efficiency of the device. The use of curved silt vanes is not suitable if there is not enough space for constructing them as for a small distributary or in case of a small off-take from a deep parent channel. The experiments showed that in such cases straight vanes built at an obtuse angle not greater than 3:1 or 4:1 and with sloping upstream ends in cut water shape are effective. Low silt vanes are more effective in excluding silt as compared to high vanes.
The contribution of silt vanes is enhanced when curved wing wall is used in conjunction with the former. High vanes especially deflect large proportion of water towards one side, and water from upper layers rushing to take its place produces rotary flow over the vanes. As a result, greater volume of water is enclosed within the wing than required by the off-take. The combination is very useful for a narrow parent channel. Low vanes of 0.4' or 0.8' height may be constructed at first and if conditions warrant, the height may be increased subsequently.

Silt tunnels or silt platform is another effective device for excluding silt. It is a reinforced concrete slab placed horizontally in parent channel opposite the off-take head and supported by walls or piers. A curved extension of downstream wing built on the top of platform to guide the clear water into off-take could be useful. The height of tunnels is not to be kept less than 2 ft. to avoid the risk of their choking since in that case all the silt would be thrown to the surface and would enter the off-take. The top of platform must be set at a level that would allow flow of ample water on its top to feed the off-take even during minimum supply in the parent channel. The width of platform is calculated from discharge that has to pass over it. The upstream end of walls or piers

must terminate in 1:3 slopes with cut-water noses. The flow into the off-take has considerable velocity which eliminates the tendency of silting in the head reach. A few vanes built just below the exit end of tunnels would lead silt to the middle of the parent channel.
A simple platform without curved wing is difficult to design as the unknown proportion of silt free water is passing into parent channel down the off-take head. In author's opinion, the platform should be wide enough to take over it the discharge of the off-take with 25% extra. Its upstream and downstream edges should be built at an angle of 60° to the parent channel. The upstream edge of platform at the edge of parent channel must be kept 5' to 10' upstream of the upstream edge of off-take head.

The curved wing wall is probably the third best device as a silt excluder. It is an extension of downstream wing wall into the parent channel, in a curve concave towards upstream in order to force water from entire depth of parent channel into off-take. The off-take therefore draws same proportion of silt as carried by the parent channel. The projection of wing wall into the parent channel should be enough to enclose the water required to feed the off-take and extend to cover 3/4 width of off-take. This device should be used if both the parent as well as the off-taking channel have same silt transport capacity. An off-taking channel with less silt transport capacity indicates the use of silt vanes or silt tunnels.

A raised cill or a wall across the mouth of off-take head is used to allow a definite discharge into the off-take. It was probably the first device to be used for excluding silt but has not proved effective. This together with skew head and curved upstream wing were introduced in 1908 in Punjab as an improvement on the simple right-angled off-take.
There are of course situations where standard devices cannot be used and such cases have to be dealt with intelligent application of principles of silt exclusion. The Ashford syphon was designed for Madhopur Headwork's for excluding shingle entry into Upper Bari Doab Canal but the device failed to achieve the objective. Gibb's Semicircular wall built opposite the off-take and completely enclosing it, is a modified form of raised cill. It did not prove to be an effective device either.

Paper No. 174
Year 1934

METALLIC ARC WELDING AS APPLIED TO BRIDGES AND ALLIED STRUCTURES WITH SPECIAL REFERENCE TO THE NORTH WESTERN RAILWAY

W.T. EVERALL AND P.S.A. BERRIDGE

Paper No. 174
Year 1934

METALLIC ARC WELDING AS APPLIED TO BRIDGES AND ALLIED STRUCTURES WITH SPECIAL REFERENCE TO THE NORTH WESTERN RAILWAY

By

W.T. EVERALL AND P.S.A. BERRIDGE

Joining steel or iron members of a structure together by Metallic Arc, also known as Electric arc Welding, instead of using rivets or bolts is becoming an important branch of Structural Engineering. In Western Countries the method of Metallic Arc Welding has already become very common.

The process means the fusion together of two surfaces of metal so that the junction shall have all the qualities of the parent metal. For this, an electrode applied along the line of the weld is fused into the parent metal by the heat of the electric arc. The quality of a welded joint depends on three things: the electrode, the temperature of the weld during fusion which is in direct relation to the current used, and the skill of the operator.

There are 3 types of electrodes, i.e. bare wire, paste coated and asbestos covered electrodes. Welds made with bare wire electrodes are unprotected from the atmosphere during fusion and they require higher current and longer arc. The resulting deposit is porous and brittle. Paste coated electrodes are cheaper than the asbestos covered ones but the slag produced has a high melting point and does not become sufficiently fluid to afford complete protection from the oxidizing influence of the

atmosphere. Also the paste, chiefly chalk, has tendency to flake off as the electrode gets hot leaving the wire bare. Welding with asbestos covered electrodes is like modern steel making where a flux, having a lower melting point and a lighter density than the steel, added to the charge, protects it from the atmosphere. For welding in Bridge Work where the ductile properties of the parent metal have to be retained, the last mentioned electrode is used.

Tensile strength tests were carried out on the welds made by these types of electrodes and the result showed that the best were with certain types of asbestos covered electrodes. These results were also confirmed by 1 zod Impact Test. Tests were carried out on 12 specimen of each type and result averaged. Some variations in the results are attributable to human factor. Test was also carried out in which specimens were tested in an alternating stress testing machine. The number of reversals before fracture was noted and the results compared. For weld deposit number of reversals varied from 840 to 1950 and ultimate stress varied from 17.9 to 28.9 tons p.s.i, whereas for Mild Steel Plate, number of reversals was 24000 and ultimate stress was 28-33 tons p.s.i. The apparent lack of consistency between the results is interesting and it shows the necessity for investigating the ductile qualities of weld metal subject to suddenly applied stresses.

Direct or Alternating current may be used with asbestos covered electrodes which although generally attached to the positive pole, can be attached to either pole. To weld with covered electrodes about 30 Volts on Direct and 70 Volts on Alternating current circuit are required but owing to the resistance of slag or other causes a pressure of 100 Volts may be needed. The amperage to be used depends upon the cross sectional area and covering of the electrodes, and thickness of the parent metal. The penetration of the weld metal with the parent metal is dependent on the temperature developed, which is proportional to the current used. Too high temperature in the metal adjacent to the weld will enlarge the crystal structure and render the joint brittle. Too low temperature may result in lack of penetration and consequently a weak joint. Inexperienced welders are apt to use excessive amperage as it is easier to maintain the arc and the work is done more quickly. The strength of the welded work should be checked periodically.

Overhead welding is more difficult than vertical or horizontal flat welding. The operator is provided with a screen helmet, gloves and fireproof overalls. There is no danger to the staff working on a welding job so long as they do not look at the arc without using proper screen of specially coloured glasses.

Distortion of the structure during welding is allowed for otherwise internal stresses will be set up. To avoid expensive straightening after welding, the amount of deposit on either side of the neutral axis is kept as nearly equal as possible. In this way the distortion on one side will balance that on the other. Another method of eliminating the distortion is that of "peening" the joint i.e. after each run of weld has been allowed to cool; it is lightly hammered with a round faced hammer.
In a girder structure, the types of joints usually used are butt joint, longitudinal fillet joint, cross fillet joint, angle fillet joint and angle weld joint. But welds are used to transmit direct stress or longitudinal shearing stress or both with or without bending or torsional moment having no component about the longitudinal axis of the weld. Fillet welds are used to transmit longitudinal or transverse shear or both.
The use of welded junctions enables the designer to place joints axially to the members which is simpler than when designing with riveted joints; but every welded joint requires careful consideration in design to avoid serious concentration of stress. Butt welds are useful in direct compression or tension, and fillet welds in end or side shear. Where a member carrying direct stress is joined to another member the centre of gravity of the welded seams should be on the centre of gravity of the members. It is usual to provide joints that are capable of carrying the variety of stresses induced within them by taking values proved to be reasonably conservative by experiments.

The structures which have been designed at the outset for arc welding compare favourably with those designed for riveted joints economically but this is not the case if structure has been initially designed for riveted connections. An economy of 25 percent in weight has been shown in comparison with a riveted structure when electric arc welding was used. In the North Western Railway Electric Arc welding has been used successfully in case of Indus Bridge at Kotri (roadway brackets), strengthening of Plate Girder spans in Quetta Division, two 50 ft. span

all welded truss purloins, and bottom of High Service water tank at Lalamusa.

Note:

Paper No. 174 appeared in the Proceedings of Punjab engineering Congress, 1934. Vol. XXII at pages 79 to 93. It has 13 Plates showing details of welded joints. Discussions are at pages 93a to 93m.

Paper No. 195
Year 1936

RECONSTRUCTION OF THE KHANKI WEIR

By

A.N. KHOSLA

Paper No. 195
Year 1936

RECONSTRUCTION OF THE KHANKI WEIR

By

A.N. KHOSLA

The Khanki head works situated near the Khanki village at a distance of nine miles downstream of the Alexendra (Wazirabad) bridge, is very vital for the prosperity of the province. It feeds lower Chenab Canal which irrigates two and a half million acres each year and brings in annual gross revenue of two crores of rupees. The construction of the weir was completed in 1891 and started functioning in March 1892' Originally a weir across the Chenab was constructed and it consisted of 8 spans of 500 feet divided by 10 feet piers. On the extreme left a set of 12 undersluices spans of 20 feet and the canal head regulator with 12f spans of 24.5 feet completed the headworks.

The section of the weir was extremely flimsy and a considerable damage occurred due to the severe floods in the very next year of its opening. In 1895 the damaged portion was dismantled and rebuilt but a further subsidence of the crest took place in October & November of that year.

The Khanki weir was the first weir in the Punjab constructed on the alluvial sandy bed of a river. The incidences of failures prompted Col. Clibborn, Principal, Thomason Civil Engineering College, Roorkee to investigate the laws of flow of water through subsoils below hydraulic works. In 1896, Col. Clibborn recommended that Hydraulic Gradients along the path of flow should form the basis of design. It can be said that the history of failures, repairs and remodeling of this weir is the history of evolution in design of weirs on sand foundations.

Between 1897 and 1931, repeated undermining of the impervious floor of the weir and appearance of leaks & springs through sand, were observed at regular intervals. The damages were repaired with the help of grouting at different locations. In addition to this, a line of wells/sheet piles was added for the protection of the upstream floor. But even with these periodical repairs, the process of undermining was not stopped and large cavities continued to form but remained undetected and ungrouted. Serious damage occurred to the downstream protection of the undersluices and in the right half, block protection was completely carried away and a 27' deep scour hole below the floor level was formed. In the August of this year, Bay 3 and 4 weirs also faced with grave damages. From the site indication, it appeared to have been a clear case of undermining of the subsoil by piping. This was also confirmed by the very little resistance by pressure pipes inserted in the weir and the extensive grouting in every previous year. Because millions of acres of cultivated area was dependent on the safety of this headworks, the need for the reconstruction of the weir is evident.

A comprehensive scheme of reconstruction was prepared by Mr. H.W. Nicholson, Superintending Engineer which was duly sanctioned by the Government and this project is the subject of this paper. This scheme of reconstruction aimed at securing firstly, the safety of the weir and secondly, the exclusion of harmful silt from the canal. Under sluices, weir bays, Bell's bunds and silt tunnels are the important components where partly or wholly, reconstruction was done.

The silt trouble of the canal started with the first opening of the canal and a large amount of money has to be spent on silt clearance. In the Khanki weir, still pond system, which was very successful at Rupar, was adopted. Due to lack of sufficient attention towards the periodic and adequate scouring of the pocket, the approach was silted up and the control of the river was lost. In 1910 - 11, this method was replaced with open flow system according to which certain quantity of water had to be continuously escaped through the pocket to keep the latter reasonably clear of silt while the canal was in flow. The weir crest was raised by 2 feet in bays 5, 6 and 7 and a subsidiary regulator of 6 bays of 24.5 was added to the main head regulator on the left. Still pond system was again adopted in 1916 and a 2 - feet raising of weir crest in remaining bays was done. Despite these efforts, silt trouble in the canal

remained acute in 1920-22 and a further raising of 2 ft. in crest was done. Generally unduly high discharges, upto 30,000 cusecs, were passed through the undersluices which resulted in undue development of the left channel and in almost complete choking of the right channel. Water flowed in high floods parallel to the weir from the left to the right. In 1932-33 the existing right channel was closed by constructing a bund and a straight cut of 60' wide was excavated. When the river supply rose, it developed into a width of 1200 ft. carrying nearly the same discharge as of the left part of the weir. This development of right channel helped in the control of silt in the canal. The left channel at the bifurcation took off on the outside of curve and right channel took off on the inside of the curve where silt charge was maximum. Bay 4 and 8 were depressed and helped to achieve similar conditions of curvature above the approach to the pocket. In addition to the above measures/reconstruction, 12 main regulator bays had been equipped with 6 tunnels with roof at 5 feet above the raised crest in order to exclude more silt from entering the canal. Partial still pond system is the mode of current regulation. These measures effectively reduced the silt entry into the canal and it is anticipated that the silt trouble of the canal will disappear completely.

Six bays 1,2,3,5,6, & 7 have been reconstructed as weir bays with crest level remaining unchanged. The crest block of the glacis down to the first toe wall has been left intact. The rest below this toe wall to the block area has been reconstructed as impervious floor down to the second toe wall. A line of 'Universal' interlocked steel sheet piles has been driven on the upstream side of the crest. Below the downstream pile line, the reconstructed floor is an inverted filter and after that flexible protection of 40 'length is repaired.

On the block area, a series of arrows have been constructed which throw up and deflect the bottom high velocity jets to the top and dissipate energy. The toothed floor surface, the stepped compartment and the arrows form an excellent combination for dissipation of energy. During the course of reconstruction, a number of cavities were found which were grouted by a grouting machine. The biggest cavity was discovered under pier 5' which extended at least 20' on one side of pier & 13' on the other side.

Bays 4 & 8 were depressed in order to get favourable curvature for silt control. A cross line of piles runs under the divide piers linked at the upstream end to the crest pile line and at the downstream end to the downstream pile line. The piles were driven in the old piers and this part of the work was done with great care. It demonstrates that these - types of risks can be taken for such works only with proper planning and supervision. The presence of the Clay substratum in bay 4 was responsible for a second pile line at downstream of the crest pile line. The bottom of first line of piles was penetrated in the clay substratum and the indication was that there was a leak from below the clay along the piles into the sand layer above. As the pressure in the clay was more than that in the sand and if in future this leak occurs then the floor will lift up. This possibility was removed by the provision of second line of piles.

The entire concrete in bays 4 and 8 was laid in layers and not in one mass. This type of concreting has certain advantages and disadvantages. The biggest disadvantage is the lighting up of floor under uplift pressures because of the separation of different layers and not acting as a single mass. This was effectively avoided by careful design of different layers and the provision of vertical stirrups. In addition to mechanical bond, the layers were thoroughly cleared and a cement grout was applied just before concreting. The drawbacks of mass, concreting such as vertical joints, indeterminate internal stresses, high costs etc. were avoided.

Baskets were used for the entire concreting of the floors the concrete was brought from mixers to the platforms by trucks. The direct dumping from trucks involved certain drawbacks such as mixing of foreign matter by truck wheels in the concrete and segregation of mortar from the aggregate and these were overcome by the use of baskets.

Precast concrete units were also used for greater progress and for economy. Liberal use was made of plums in mass concrete of groynes and blocks. A fish ladder in bay 8 and trough bridges were also constructed. The entire steel work was manufactured in the Central Workshop at Amritsar. Portable pumping sets with 8" to 10" pumps were used for entire pumping. A power house on the left bank consisting of one 40 K.W. and two 10 K.W. sets supplied the required

electricity for lighting and pumping purposes. The Universal and Ransoms uniform (D) were to two types of piles used. The Universal type is heavier in section and good for hard soil. The ransom uniform (0) is also good for hard soils but it bends when driven through stone but it is more water tight and cheaper because of lighter section. Oxy-Acetylene flame was used for the cutting of piles. All building stone and pitching stone were obtained from the Irrigation quarry at Baghanwala 87 miles from Khaniki. Ballast was obtained from Jummu sixty miles away. The entire plant and machinery used was old. Proper planning of every step of reconstruction saved a lot of money.

Paper No. 197
Year 1937

WATER LOGGING ON THE UPPER CHENAB CANAL, its CAUSES AND CURE

By

B.N. SINGH

Paper No. 197
Year 1937

WATER LOGGING ON THE UPPER CHENAB CANAL, its CAUSES AND CURE

By

B.N. SINGH

The Upper Chenab Canal feeds Lower Bari Doab Canal by crossing Rechna Doab with a full supply discharge of 12000 cusecs. The danger of water-logging was anticipated by the designers as the alignment crosses all the drainage lines of the Doab. As a preventive measure, the area was divided on the basis of depths of spring level of more than 35 ft, between 30 & 35 ft and less than 30 ft. respectively. A & B zones were perennial having 60% intensity of irrigation whereas C zone was non-perennial with 25% irrigation intensity.

The Upper Chenab Canal started functioning with its full capacity in 1916 and water logging was reported in 1918. The water logging went on spreading and became very serious in 1925. Government ultimately appointed a Water logging Enquiry Committee for the investigation of causes of water logging. This Committee accepted 28,500 acres as damaged in 1927, and a close and constant watch on the situation and a systematic treatment of the subject were considered essential. This led to the establishment of Water logging Board in 1928.

For the discussion in this paper, the command area of this canal is divided into four tracts according to the intensity of irrigation, the distance from the main canal and the anti-water logging measures undertaken. The area within a distance of 3 miles on both sides of the, canal is termed as Tract I. The tracts lying within the irrigation boundary of Raya Branch and Nokhar Branch are called as Tract II & III respectively. Tract IV is irrigated by perennial distributaries of the

Upper Chenab Canal and it is lying between Main Line, Upper Chenab Canal and the irrigation boundary of Lower Chenab Canal.

The maximum water logging occurred in Tract I and in this tract, maximum expenditures were also incurred. Pipes at regular intervals were fixed all along the main canal for monitoring of anti-water logging measures. Average rise or fall of water table was plotted for three Divisions, Marala, Gujranwala and Sheikhupura. It was apparent from these graphs that water table was rising in Gujranwala and Sheikhupura Divisions even before the opening of the canal but this rise was accelerated from the year 1916 when the canal started running with high supplies. In Marala Division, the rate of rise slackened after 1918. The maximum rise was in 1926, the year of heavy rainfall and from 1926 to 1933, there was practically no rise in all of the Divisions. Conditions within half a mile of the canal were very bad. In 1923, 65 miles were reported to have been affected which caused great alarm. Seepage drains along the canal, lowering full supply levels of the Main Canal, tube well pumping, restriction of supply for irrigation, pumping in local areas, surface drains and tail reach diversion were some of the measures adopted singly or jointly.

First signs of water logging appeared along the canal in the form of water standing in borrow pits, pools and other low areas. Seepage drains along the canal, therefore, seemed to be a good measure for the control of water-logging and a large number of constructed drains did help to give local relief. But because of increase of percolation head, springs started appearing in the bed, caused rapid silting up of the drains, and did not help in the control of water table. Further these tended to increase seepage losses from the canal owing to the increase of percolation head and most of these drains, therefore, have been abandoned.

Lowering of full supply level of the Main Canal was also done to get relief in water logging. Firstly, observations to examine the effect were taken from open pits but these were discarded because open pits were not reliable as rain and irrigation water entered into them. The pits were replaced by pipes with filter points at the bottom. A number of graphs and tables conclusively proved that the lowering of supply levels did not have any marked effect on the water table in the tract outside half a

mile of the canal. There was no practical effect on the belt between half a mile and three miles from the canal.

After the successful control of water logging in California, tube wells were installed in the Gujranwala and Ferozepur areas. The results show that tube wells are not a good remedy for lowering the water table permanently and even for temporary lowering they are only effective so long as there is no rainfall. These have to be installed at one mile intervals and will be very costly for Upper Chenab Canal command area of over 2000 square miles. The success of tube wells in California may be due to the different geological conditions of subsoil strata as the soils there are much coarser and more porous containing an admixture of sand and gravel and depression heads are consequently much bigger.
Another measure adopted for control of water logging was restriction of water supply for irrigation. The results showed a very low effect on the water table and it must be discarded as it also effects cultivation.
Pumping in local areas was done at tails of certain seepage drains which had no gravity outfall and near certain important towns where the water-table was rising dangerously close to the foundations of houses causing settlements. These proved to be very useful for control of water table locally near large towns and villages, but in general they did not show any considerable effect.

Ferozewala, Khoth Nalla and Nikki Deg were the only surface drains constructed within the 6-miles belt of the canal. These drains have been very beneficial and constitute an efficient drainage.

The tail reach of the Upper Chenab Canal below RD. 280,000 runs almost parallel to the river and water logging in this area became very serious. Tail reach of the canal was diverted via Deg Diversion Channel. The effects of this diversion are not yet known.

Tracts II & III i.e. the areas commanded by Raya & Nokhar Branches did not experience any significant water logging and in both tracts, the water-table is steady at about 11 ft. below ground at present. In Tract IV, water-table is generally rising and uptil now, no measure has been taken. The water table is at present about 9 ft below ground level.

Water logging also affected the tract on the right side of Main Line above Nokhar branch. The Sambrial-Aik Nallah Drainage Scheme has changed the entire area and at present there are no signs of water logging.

In the second part of the paper general causes, mechanism and effects of water logging and anti-logging measures, in the Punjab have been discussed. There was no planned policy till 1933 when first five years plan of drainage construction was adopted on the author's proposal. This plan was necessary in order to secure land from the evil progressing of water logging.

It can be said that in the pre-canal period, the water table was in state of equilibrium i.e. inflow was equal to outflow and the subsoil drainage was sufficient to cope with the inflow. Before the construction of canals in Doab, infiltration from rivers towards bottom of the trough, absorption of rainfall and subsoil flow from upper regions of the Doab constituted inflow whereas outflow was the subsoil flow towards the lower regions. After the construction of canals, seepage from canals disturbed the equilibrium condition because of insufficient subsoil drainage. The studies of this area show that water logging is to a large extent due to seepage from the canal system and to a small extent due to increased absorption of rainfall resulting from breaking up of new areas for cultivation.

The extent of water logging can definitely be reduced if amount of inflow into the subsoil can be reduced. Irrigation canals, subsoil flow from upper regions and rainfall are the source of inflow. Irrigation cannot be reduced; seepage from canals can be controlled by lining but uptil now no lining material has been found which would be effective as well as durable, cheap and capable of being applied with in the short time during closures. Subsoil flow from the upper reaches is a natural phenomenon and, hence, cannot be stopped. The only option left to reduce inflow in the Doab is the check of rainfall run-off. A large amount of water can be drained out naturally as the country is made up of water sheds and drains with almost scientific regularity. Artificial channels of suitable size should be provided along natural drainage lines so as to carry away the storm water rapidly to the rivers without causing undue flooding in the surrounding area. Greater number of these drains will drain out faster the run-off. This method is only

beneficial in the areas where rainfall is the main cause of water logging. In other areas, the lowering of water table in the upper reaches of the MO through lining of canals can also be done.

Paper No. 211
Year 1938

SILT EXCLUDERS

By

F.F. HAIGH

Paper No. 211
Year 1938

SILT EXCLUDERS

By

F.F. HAIGH

The basic principle upon which the design of silt excluders is based lies in the fact that in flowing stream carrying silt in suspension, the concentration of silt in the lower layers is greater than in the upper ones. Thus if lower layers of water can be escaped without interfering with silt distribution, the remaining water will have less silt in it per unit volume. Elsden's design, who first floated the idea in 1922, consisted of a regulator divided into two portions by means of a horizontal diaphragm over which the upper water passed into the canal while the heavy silt laden lower layers escaped through the tunnels to waste. With modification in detail this form of silt excluder was constructed at Khanki Headworks in 1926. A smooth approach channel is an important feature of silt excluder design. Its object is to permit the silt to settle more effectively and hence to increase the efficiency of its exclusion.

The construction of the first excluder at Khanki was followed by construction of other such devices including three extractors and two excluders on Upper Jhelum Canal. An excluder is at the head of the canal and this excludes a proportion of the silt, while the extractor being placed at some distance down the canal extracts or ejects silt which has entered the canal.

The first point to be considered in the design of such devices is the approach conditions. A long straight approach channel should be provided in which silt can settle into the lower layers.

The first point to be considered in the design of such devices is the approach conditions. A long straight approach channel should be

provided in which silt can settle into the lower layers. If the approach is not straight but curved or the bed or sides are rough then silt concentration will be disturbed. Optimum silt distribution can be obtained when some of the silt is rolling along the bed and with this condition it would be immaterial whether the bed was lined or not. It is possible that a narrow deep section would have a greater efficiency than a shallow wide one. The approach channel should be designed with the flattest slope which will suffice to carry the heaviest grade of silt, likely to approach the work. However for reasons of economy approach channel should be kept as short as possible. An approach channel can easily be provided in case of an extractor but in case of excluder, it is usually necessary to turn the water through a right angle bend to draw the canal water from the outer curve. In any case, the approach channel, which consists of a natural river bed, will probably have curves in it and will certainly have very irregular boundaries.

The proportion of escapage from the canal supply is also to be decided very carefully. The efficiency of an excluder may be defined as the reduction per unit of the silt intensity in the canal supply when compared with that of the water approaching the work. This, though practical, is a false criterion. The true measure of the efficiency of an excluder is unity minus the ratio of the silt entering the canal to that which would enter where the excluder is not working. The point about this distinction is that the addition of the escapage discharge to the canal discharge increases the silt approaching the canal and increases it in a proportion greater than that of the discharge. It was demonstrated by Crump that an increase of the escapage discharge is always accompanied by a marked reduction in the ratio of the silt to the water i.e. intensity. We must not, therefore, assume that the greater the escapage the greater the efficiency. More research is necessary on this. For the present, however, about 20% is considered reasonable.

Another problem is the separation of escapage from canal supply. The separation of the escapage water from the canal supply at the edge of the diaphragm should be arranged without disturbing the silt distribution. It is easy enough to arrange this for fixed canal and escapage discharges by placing the diaphragm at a height such that it divides the normal stream into the correct proportion. In practice, however, it is always necessary to vary both the canal supply and

escapage and if the height of the diaphragm is fixed it will generally not suit the proportions of the two.

Another component of silt excluder is the tunnels. The tunnels must be arranged to evacuate the escapage at a high velocity not less than 10 f.p.s. They must also provide control of the discharge so that the same velocity is secured at the entrance to each tunnel. This may be done by keeping the same tunnel dimensions and varying the width of canal served by each tunnel or vice versa. The low velocity at separation is to be transformed to the high tunnel velocity at the entrances. This must be done without effecting the velocity distribution upstream of separation. Moreover when escapage head is valuable a gentle transformation is necessary to avoid its loss. The former point can be secured by placing the entrances at a sufficient distance downstream from the edge of the diaphragm. Thus the tunnel roof should be designed to take the full water pressure above it with the maximum pressure which may occur inside it assuming the entrance to be blocked. If the tunnels act as a weir for the canal supply, the possibility of uplift occurring with the tunnels closed at the downstream end and/or velocity depression over the roof should be studied for design. The escapage if less than full supply is regulated by gates on the downstream end of the tunnel. Tunnels can also be provided with trash racks to prevent jungle/debris blocking them.

If the canal is flumed then the crest of the canal flume cannot be below the diaphragm level but may be above it. By varying the section of the canal the diaphragm level may be varied over a large range. When the canal flume forms a control point it is necessary that the downstream edge of the weir should be normal to the stream. The inclination of the upstream edge resulting from the triangular tunnel plan is immaterial except that the crest level may be varied to counteract the varying co-efficient of discharge resulting from the varying length of crest.

A tail race is provided to pass the escapage back to the river if necessary. It might be expected that a steep slope would be necessary to carry the heavily silt charged escapage. However tail race is found to work with a flatter slope and with a C.V.R which is much the same as that of the canal.

Silt excluders have so far been applied and used in a primitive fashion. Wherever silt entry was affecting the regime of the canal to a dangerous extent, a silt excluder was built to be as efficient as local conditions would permit at reasonable cost. If an excluder was too effective and resulted in too rapid retrogression of the canal, it could be disused or worked intermittently. No attempt was made to regulate the silt at entry to a grade or a quantity suited to the slopes for which the canal had been designed but such regulation must be the aim in the case of an established canal system. When the silt contents of a stream running in a bed of self-borne material are reduced, retrogression results which also flattens the slope until a new canal regime is established. The delay involved in this process can be eliminated by artificially regarding the channel to a flatter slope. Such regarding is necessary as otherwise the retrogression may affect the safety of masonry works etc.

In order to study the working of excluders, the discharges and silt contents of the water passing them are observed periodically. Silt observation may be made by various methods as below;

(a) Trapping the whole silt of the stream for a period.
(b) Sampling from turbulence
(c) Sampling from the normal stream from points spread over the section horizontally and vertically
(d) Sampling from single points on verticals.

The first method is difficult but most correct and may be used as cross-check on other method.

The method of calculating efficiency in general use gives the reduction of silt intensity in the canal water as compared with that of approach flume. The factors which affect efficiency are:

(a) The proportion of' the supply escaped. As discussed earlier the efficiency will not vary directly with the escapage. Since the intensity decreases rapidly with depth, additional escapage will increase the efficiency but slowly.

(b) The grade of material carried by the water. The same excluder may be expected to work more efficiently where the proportion of coarser silt is greater. On the other hand, the coarser and heaviest grade of silt carried, the greater the slope and velocity will be, and consequently the lesser the concentration of silt in the lower layers. It seems probable that the coarser the silt present, the greater the efficiency would be when based on the same grade, but this is by no means proven by our present knowledge of the subject.

Note:

Paper No. 211 appeared at pages 53 to 72 of the Proceedings of Punjab Engineering Congress, Vol. It has 5 plates. The discussions are recorded at pages 72a to 72w. The discussions were mainly regarding the design options and the behaviour of various silt excluders mentioned in the paper.

Paper No. 215
Year 1938

RECONDITIONING OF MARALA WEIR

By

E.O.COX AND R.B. GANPAT RAI

Paper No. 215
Year 1938

RECONDITIONING OF MARALA WEIR

By

E.O.COX AND R.B. GANPAT RAI

Marala weir consists of 8 bays of 500 each with undersluices on the left flank where upper Chenab Canal takes off. The weir was built on sand foundation like similar wide shuttered weirs at Khanki and Rasul, which failed most probably because of piping. This weir is very important because on it depends the water supply to the upper Chenab and Lower Bari Doab canals, the combined net annual revenues from which are about Rs. 1 crore. Therefore its likely failure as per experience of Khanki and Rasul weirs had to be avoided.

Construction of Marala weir started in the end of 1908 and was completed in May 1910. It was designed for a maximum head across of 10, and has three well lines, that is on upstream end of impervious floor (A), half-way down the downstream glacis (B), and at the downstream end of impervious floor (C). Total length of work was 320', out of which impervious floor was 140' and the upstream and downstream base protection 70' and 110' respectively. The hydraulic gradient was 1 in 14 as against 1 in 15 recommended by Bligh. Glacis between B and C line of wells was semi pervious. This form of construction was probably adopted to relieve any residual pressure without blowing sand. At the same time, glacis was required to be strong enough to stand upto the dynamic action of water. The structure met more of these design requirements.

Marala weir was also facing problems right from the time of its construction. During the dismantling of the downstream glacis, gaps upto 1' in width were found between the two well lines and the wooden piling. In the glacis above the second line of wells, the bottom layer of

masonary had been laid dry and a weak grout of lime surkhi and sand had been poured over it which did not completely fill the joints between the stone. This was quite evident from the examination during dismantling. The state of these open joints showed that water had been flowing freely through them over the sand below.

In order to keep down the upstream water level and facilitate pumping and subsequent diversion of the river, the glacis in bays 1 to 6 was first built to a reduced section, the crest being kept at R.L 797.5 i.e., 2.5' below the final crest. Later in 1909, while raising the reduced section to full height, existence of transverse cracks in bay No. 3 was noticed. The rubble masonry has also settled 2" to 3" in some places below the crest the weir was first put into commission in April 1912 and the following October, after a monsoon of moderate floods, the glacis was found to be full of cracks and springs. In the following year fresh cracks and springs appeared and in 1917 after a flood of 550,000 cusecs the stone on edge course (Kharwanga) in bay 6 was found to be uplifted. An area 90 x 15 above B line of wells had bulged. In subsequent years this bulging occurred almost all over the glacis between the crest, and the C line of wells. In the record flood of 6,86,000 cusecs on 1.9.1928 and another of 66,000 cusecs the following year on 29.8.29, a good deal of damage was done below the weir.

A detailed examination of weir in October 1934 showed that the condition of weir was far from satisfactory. Model experiments were carried out in the Hydraulic Laboratory Lahore, with the following assumptions (i) The well lines were leaking (ii) the top 1.5 of sand below the wire was coarse (iii) there was a hollow between the Kharwanga and the loose stone below between the B and C lines. Of these (i) and (iii) were subsequently found to be correct. Pressure pipes were installed in the weir and observations were made on a number of occasions. These observations showed that there was either no drop from the crest to B line or that its extent was negligible, and that in bay 2 the drop between the B and C line was small and there was residual pressure of 26% above the third line. It appeared from the results that there were cavities under the floor in many bays. This combined with the high residual pressure above C line and the fact (which was verified during the dismantling of the weir floor in bay 8 that some of the wells in this line were only 4 to 5 feet deep led to the conclusion that the weir was in serious danger.

From the above discussion it is clear that weir was designed on wrong principles due to insufficient knowledge of hydraulics. One bay partially subsided under no head during construction, and since the weir has been brought into operation, it had been maintained intact only at considerable expenses. The whole of the impervious section from the crest to B line was full of cracks and open points through which springs were working. The pressure pipe observations gave every indication that cavities existed below. Below B line which had to stand the pounding action of the standing wave, the glacis was too thick and cavities existed along it almost from end to end.

The main defects in the Marala weir were the flat slope of the downstream glacis and the comparatively high level at which the downstream loose protection was laid. To remedy these defects it was decided to dismantle the existing glacis from a few feet below the crest and rebuild it to a new design so as to give a depth in high flood greater than at Khanki.

The work of reconditioning was to be completed in a single season by early March. Besides the problem of unwatering, winter freshets posed another problem. Another consideration was that not only Upper Chenab Canal should run but some water would have to be escaped below the weir for Khanki. After considering various proposals, it was decided to start work from the left flank of the weir and proceed towards the right. Left flank is subject to direct attack of floods from Jammu Tawi. The more difficult part of the work would thus be taken up and completed earlier before frequent freshets which occur after mid-December. The work consisted principally of the following items:

(i) Driving 14 deep continuous line of sheet piles above the C line of wells.

(ii) Replacement of the semi-pervious sloping floor between the B and C lines of wells by a horizontal concrete floor 4 feet thick, the new floor being depressed 4 feet below the original level.

(iii) Reconstruction of the glacis above B line to a slope 1 in 4 until

it met the old glacis about 11 ft. below the downstream edge of the crest

(iv) Lowering of block and loose stone protection below C line by 4 feet and Provision of two rows of raised and staggered blocks of size 5' x 2' x 2' on the horizontal floor between B and C lines of wells and three rows of such blocks downstream of C line.

(v) Grouting of upstream glacis to increase water tightness.

The procedure followed in carrying out the works was that after the ring bunds, up and downstream had been linked and sufficiently strengthened pumping was started followed by dismantling of the weir floor, excavation of the foundations, pile driving, concreting and stone masonary. Work was done in three stages. Bays 1, 2 and part of 3 were taken up first, the remainder of bay 3, bay 4 and part of bay 5 next, and the remainder of bay 5 and bays 6,7 and 8 last of all. In the construction of ring bunds it was kept in view that the bunds should be strong enough to enable the work to go on under reasonable safety, the bunds in the part of the weir to be next enclosed should be started well in time and that at all times sufficient escapage capacity should be available over the weir and through the undersluices for the passing of freshets.

Main pumping units installed for dewatering consisted of centrifugal pumps of 10" to 14" size with portable steam engines. For local pumping, Petter crude oil pumping sets or electric sets were used in open sumps or tubewells., The type of piling generally used was Ransome uniform D but where it was necessary to drive piles through stone, the type known as "Universal" was used. For lowering of block area and stone apron, old concrete blocks weighing 3 to 3.5 tons each were to be removed and re-laid. Two old dragline excavators working on caterpillar wheels were used for this purpose. Major plant and machinery was collected from within the department. Advantage was taken of the work of reconditioning of the weir to put in a number of observation pipes under the floor of the weir. Due to non-occurrence of high flood so far, position of standing wave after reconditioning has not

been ascertained.

The work was started in end September 1936 and was completed by end March 1937. The sanctioned cost of work was Rs. 15.5 lac. The final expenditure is not yet available. The methodology adopted ensured that a work which would have normally taken two seasons to finish was satisfactorily completed in one season within the project estimate, resulting in a saving of 3 to 4 lac of rupees.

Note:

Paper No.215 appears at pages 153 to 195 of the Proceedings of Punjab Engineering Congress 1938, Vol. XXVI. It has 15 Plates. Discussions are recorded at page 195a to 195s.

Paper No. 221
Year 1939

LINING OF THE HAVELI MAIN CANAL

R.S. DUNCAN

Paper No. 221
Year 1939

LINING OF THE HAVELI MAIN CANAL

By

R.S. DUNCAN

The lining of Haveli canal is an attempt to avoid water logging and save water. Its cost is estimated at Rs. 57.4 lakhs, while saving in cost of excavation, land and masonry works amounts to Rs 5.9 lakhs. The net cost of lining of Rs. 51.5 lakhs would be recovered in 16 years through revenue accruing to the government on the anticipated saving of 330 cusecs of water due to less percolation.

To determine the percolation losses through various types of lining, experiments were conducted by Gupta, on the left bank of the Lahore Branch of UBDC. These showed that "Sandwich" lining made of two layers of brick tiles in cement sand mortar with an intermediate layer of cement plaster was comparatively efficient and cheap. The concrete lining would require mixing machinery and better supervision. The cost of 1 : 3 : 6 concrete with brick ballast is also higher by Rs. 8.50 per % Cft. than "Sandwich" type. Sandwich lining as finally decided consists of a lower course of tiles 12" x 5.875" x 2.5" in 1 : 6 cement mortar, bedded on 1/2" layer of same mortar. Top is covered by 1/2" layer of 1 : 3 cement sand plaster to enclose longitudinal and transverse reinforcement of 1/4" bars. On top of the plaster is the upper course of tiles in 1 : 3 mortar.

The Haveli canal would be lined from RD 2,000 to RD 227,800 in length of 41.56 canal miles. There are 31 kilns for brick burning. The basic rate of tiles was fixed at Rs. 15/- per 1000. A kiln was located in each reach of 7500 of canal and was required to produce 3.5 to 4 lac tiles per month as per agreement. Five more kilns were later added in order to finish all the lining by the end of March 1939.

An overseer is in charge of a "heading" or working site and looks after the kiln excavation and other masonry work in his reach. Entire project comprises five Sub-Divisions which fall in the jurisdiction of two Divisions. S.D.O's are responsible for supervising and maintaining quality control in manufacture of tiles and works. Cement is supplied by Punjab Portland Cement co. (Wah) at Rs. 5 per ton, sand is obtained mostly from canal excavation and steel reinforcement is supplied by Indian Steel and Wire Product Ltd. Tatanagar.

The plant comprises steel tanks for soaking bricks, wooden scaffolding for building the masonry on slopes, templates for dressing the side slopes, and G.I. pipes for water supply etc. A water course runs along the outer toe of the canal bank to serve the headings. For reaches where no canal supply is available and for canal closure, some tube- wells have been sunk. The cost of running the pumps for the whole job is estimated at Rs. 9000.

Well rammed puddle in 6" layer is put behind the lining above the natural surface to minimize subsequent settlement of earth backing. The templates give correct profile of the canal where the outer edge of the vertical scantling is truly vertical over the tangent point given on the brick in the bed. Three templates give two spaces of 25 feet each for the dressers work. Dressing of side slopes and bed is done first with kassis and then with scrapers for an accurate smooth surface that serves as a base for the masonry lining. The dressing of bed and sides is kept 2 to 3 chains ahead of masonry work.

Half an inch of fairly wet 1 : 6 cement sand mortar is spread over the bed on which masons start laying transverse rows of tiles and retreat longitudinally. For the, joints to be continuous in straight lines for making straight grooves for reinforcement, masons lines are stretched longitudinally from the grooves. The masonry on side slope is laid from scaffolding. The 12 feet planks enable a 10 feet length of masonry to be laid between two supports of scaffolding. For side slope mortar is to be richer than the bed mortar and requires more sprinkling of the formation. the hollow joints are detected with a broad chisel shaped iron bar weighted at the middle. The weight of the bar breaks the upper crust of the hollow joint. The bottom course of the tiles is scraped on the 3rd

day with wire brushes. The reinforcement is laid allowing an overlap of 40 diameters and a cover of 3 inches at the top of side slope without being cut. The plaster 3/8" thick in the bed is laid on the scraped course. The plaster in the bed is cleared and scraped on 5th day and top layer of tiles is laid on 1/8" layer of 1 : 3 mortar. On 7th day hollow joints are detected and repaired. On 8th day the area is flooded after making earthen cross bunds. On the side slopes tiles ar laid on 4th day after spreading over 1/2" layer of slushy mortar on bottom course. At least 6" depth of water is maintained over the bed masonry for 2 or 3 months by making small earth cross bunds and this pond is extended every day to include new masonry work for watering. The watering of side slopes covered with mats or cement bags is done with buckets by labourers for 30 days.

At least 1000 of fully excavated canal reach should be available ahead of lining. The ideal programme for heading is to line upstream half of the reach working downstream and vice versa for downstream half. Experiments performed in tanks to study whether 1 : 3 plaster 1/2" thick should be laid separately or in combination with top course of tiles, revealed that two methods of laying plaster are equally good. However separate layer method was adopted for the reason of water resistance. To test the as built lining two earth bunds spaced 1000' apart were made across the finished lined canal. The inside slopes of the bunds were lined with standard type of lining. The tank was filled to a depth of 10 feet. The percolation rate in November was determined to be 0.1 cusec per million square feet of wetted perimeter.

Note:

Paper No. 221 appeared in Proceedings of Engineering Congress 1939, Vol. XXVII at pages 39 to 57. Discussions are recorded at pages 57a to 57h.

Paper No. 228
Year 1939

FINANCES AND ECONOMICS OF IRRIGATION PROJECTS

By

KANWAR SAIN

Paper No. 228
Year 1939

FINANCES AND ECONOMICS OF IRRIGATION PROJECTS

By

KANWAR SAIN

In British Punjab, area under cultivation was 309.99 lac acres. About 143.8 lac acres was still lying as cultivable waste with possibilities of further extension of cultivation. Government canals were irrigating 67% of the total irrigated area. Cost of irrigation by wells is higher than irrigation by canals. The total value of crops matured by Government canals in 1937-38 was estimated as over Rs. 40 crore.

Government has to incur expenditure on the construction of an irrigation project. The cost incurred is later realized from the farmers either by a system for repayment of the capital cost over a number of years or by charging water rate to meet the interest on the capital cost incurred and the annual charges on account of administration and maintenance. However the water charges are not the only cost incurred by the farmer after the introduction of irrigation system. The distribution of water on the farm is developed by him at his own expense. For new lands a certain amount of expenditure has to be incurred by the farmer for the development and preparation of land for cultivation. There are annual costs for maintaining the watercourses and leveling of land. Thus no irrigation project can be financially successful unless the returns both to the financer and the farmer are reasonably adequate.

Canal systems were constructed from borrowed funds as commercial undertakings. In 1867, Government decided that irrigation works should be constructed by their own agency, and their viability tested as below;

(i) By considering the capital cost of any work to be simply the sum actually spent on its construction

(ii) By debiting the yearly accounts with;

(a) simple interest on the capital cost of the works at the commencement of the year

(b) the working expenses of the year

(iii) By crediting the revenue accounts yearly;

(a) with direct receipts

(b) with indirect receipts.

It was admitted that irrigation works could not be expected to pay back within 10 years of opening of the canals.

In U.S.A, irrigation projects were financed by individuals, partnerships and corporations. The area financed directly by Government by a revolving fund amounted to 8% of the total development. Generally expansion of irrigation involves an ever increasing expenditure per acre as it can only depend on residual stream flows necessitating relatively greater outlay for storage schemes. Experience in India and in U.S.A led to the same conclusion that large irrigation projects cannot be undertaken by private enterprise. In U.S.A, a standing Land reclamation fund was created. The capital cost without interest is recovered from the farmers in 40 years. A charge per acre is levied on account of annual maintenance and operation. In India, capital required for financing an irrigation project is raised as loan in the open market on Government security. Interest on this capital is met yearly from the revenue budget by debit to the administrative accounts of the project. The farmer pays only a flat rate per acre for water based principally on the value of crop harvested. The cost to the state may be grouped under three heads:

(a) Interest on the capital cost and areas of interest for the

construction period

(b) cost of administration

(c) cost of annual repairs and maintenance

In the administrative accounts of the project, the capital cost consists of direct and indirect charges. Direct charges include cost of works, establishment, tools and plant. Indirect charges consist of capitalized abatement of land revenue, Audit and Account Establishment.

Capital cost per acre of earlier projects was less than those of subsequent years, mostly due to increased price of labour and material. For the Haveli Project the actual anticipated outlay is Rs. 36.25 per acre.

The most important single item of expenditure in irrigation is the headworks in the river. The cost of headworks is independent of the area irrigated and depends upon the maximum discharge of the river and the height and type of the gates used. The cost of a storage dam would depend on its locations, its height and its accessibility for tools and plant and branches would depend on the distance of irrigation boundary from the head works, intensity of irrigation and the nature and size of cross drainage works. The cost of construction of distributaries depends on the capacity per thousand acre of the area for which these channels are designed. The cost of distributaries in non-perennial areas is higher than in perennial areas. In colony canals water courses are constructed through Government agency and the cost is recovered in installments on an acreage basis. The drainage works are to be excluded from the capital cost of a project for the purpose of considering its financial prospects. Cost of establishment entirely depends on the number of years taken to complete a project.

In the past there has been a tendency to under estimate the cost of irrigation projects. Inspite of the heavily increased capital cost as compared with original estimates, the Punjab Canals are a financial success. Irrigation receipts constitute more than 40% of the total revenue of the Punjab. In addition to capital outlay and interest on capital, there is expenditure on establishment, maintenance and repair of

canals and further improvements in the system. The cost of establishment and maintenance are almost equal. The best criteria for economic feasibility of a project would be when the interest on the capital cost plus the annual charges for operation and maintenance are the least.

In the administrative Account of the Irrigation Department, direct receipts consist of occupier rates, sale of water, receipts from plants and other canal produce, rents, fines under Canal Act, miscellaneous and other receipts. Water rates charged for various crops per acre are uniform. The water rates were changed during 1900 to 1938 on account of increase in area of cash crops and introduction of new canals. It has been suggested that water rates should be based on volumetric basis but it involves an appreciable investment and is not practicable.

Indirect receipts consist of sale proceeds of crown waste land, rent from temporary cultivation & Malkiana from crown waste land. It has always been a question whether income from the enhancement of land revenue from sales proceeds of crown waste lands is correctly creditable to the canal projects or not. The revenue on account of indirect receipts owes its very existence to the introduction of canal irrigation and should be treated as a credit to the accounts of the project. In the Punjab Canal system the direct receipts have been estimated as Rs. 1.2 per acre:

Initial cost for development of water courses and clearing of jungle is based on flat rate over the entire area of the project. On the Sutlej Valley Project this charge was fixed at Rs.3 per acre. The main annual costs are repayment of capital costs (met by water rate receipts) and the costs on working and maintenance of the system. The return to the farmer from canal irrigation may be due to increase in land value and additional income from farm produce. The land prices have gone up to Rs.200 to Rs.400 per acre. A method should be devised to credit part of this increase to the canal project. Additional income from farm produce may be due to higher percentage of matured crops to sown area, more valuable cash cropping, and higher yield per acre.

The average water rates from canal irrigation are lower and at Rs. 4 per acre whereas the average rate of water from tube wells is Rs. 10.87 per acre. When considering the benefits of canal irrigation it has been calculated that its income is Rs. 21 as compared to Rs.8 from un-

irrigated area. The price of farm produce varies from year to year whereas farm expenses remain almost uniform. Agricultural products are governed by supply and demand. However agriculture economics is different from industrial economics by virtue of major role played by natural environments. There are seasonal variations in output of crops. The percentage ratio of the average water rate to the average value of the produce per acre varied from 6% to 15% during 1918 to 1931.

The growing population demands an increase in cultivated area and hence development of irrigation. Irrigation schemes provide vast scope for employment. The only difficulty in development of irrigation is that future schemes may not pay good returns. The standard of basing the test of productivity for 10 years is arbitrary and some of the best canal system failed to come up to this test. Full development of irrigation scheme may take as many as 30 years. It is suggested that the cost of storage schemes for supplementing the existing winter supplies should be pooled with the cost on the original projects for the purpose of financial tests. The financial requirement demands that water rates should be fixed at a level that the cultivators can reasonably afford to pay.

Note:

Paper No. 228 appeared in the Proceedings of Engineering Congress, 1939 Vol. XXVII at pages 191 to 259. It has 7 graphs and 7 other plates. Discussions are recorded at pages 259a to 259y.

Paper No. 230
Year 1940

REMODELLING DISTRIBUTARIES AND DISTRIBUTION OF WATER TO AREAS IRRIGATED BY COLONY CANALS

By

A.W.M. JESSON

Paper No. 230
Year 1940

REMODELLING DISTRIBUTARIES AND DISTRIBUTION OF WATER TO AREAS IRRIGATED BY COLONY CANALS

By

A.W.M. JESSON

Remodeling of irrigation channels was carried out during the years 1927 to 1938 in the Lower Jehlum Canal Circle and East Circle of the Lower Chenab Canal, because of unsatisfactory distribution of irrigation water. In the past, to overcome the tail shortage the only remedy was to adjust the outlets without paying attention to the hydraulic condition of the channel. Now it is the time to lay down clear and definite policy for remodeling on each canal and it should be reviewed periodically when experience shows that it is defective.

In Lower Jehlum Canal Circle the remodeling of Mithalak distributary and in Lower Chenab Canal East Circle the remodeling of Mungi, Awagat and Kheowala distributary was attempted prior to 1934. Mithalak distributary offtakes from Northern Branch whereas the latter three channels offtake from Lower Gugera Branch. The channels had generally silted up from time to time and their slopes were steepened. The headregulator crests were raised and construction of raised cill was carried out in some cases to prevent excessive coarse silt entry as the original crests were at the bed of the parent channel. This remedial measure failed and the channels continued to silt up. From time to time outlets were remodeled and the orifice outlets were replaced by Kennedy Gauge outlets and at a later stage Mr. Crumps A.P.M. outlets were adopted. The channels continued to give trouble resulting in rise in water level at head and at tail shortage. Also meter flumes and control points were introduced to regulate the flow and improve the discharge

of outlets.

In 1934 remodeling of Mithlak distributary was taken up to evolve a strategy for remodeling of other channels. A detailed hydraulic survey of the channel showed that the channel slopes were steep and it had widened exceptionally in all the reaches. It was observed that main cause of trouble was the hydraulically defective head regulator which drew excessive coarse silt due to formation of eddies in the pocket in front of the head regulator. The headregulator was remodeled and the channel bed regarded to slightly flatter slopes as steeper slopes required abolishing of control points which was not considered desirable. The channel was artificially forced to conform to the new section by forming berms with hanging brushes. The design of control points was modified with the provision of broad crested 'weirs. Mr. Sharmas modified design of A.P.M outlet with its setting close to the bed level was adopted to increase the silt induction capacity of outlets.

Hydraulic survey of Mungi distributary was carried out and longitudinal section of the channel was prepared indicating hydraulic data of the channel and outlets. The headregulator of the channel was ineffective to control excessive silt entry. A skimmer already constructed failed to serve the purpose due to silt deposit in front of exit tunnel because of obstruction due to King's Vane. The head regulator was remodeled to control the silt entry. T9 remodel the channel it was planned to construct two control points consisting of broad crested weirs which required flattening of slopes. This proposal was found unworkable. The outlets were remodeled by adopting Sharma's modified type A.P.M. outlet with its setting close to bed of the channel.

Remodeling of Awagat distributary was planned after detailed hydraulic survey. It was observed that channel section was abnormally wide and shallow and the outlets were not drawing equitable share of silt. It was planned to regard the channel to slightly flatter slopes as the headregulator was to be remodeled. Sharma's design of modified A.P.M. outlet was adopted to remodel the outlets based on their satisfactory silt drawing capacity. The section was tightened with longitudinal bushing. The distributary has operated satisfactorily and it shows that in some cases flat slopes may be adopted provided head regulator is remodeled in a way to prevent excessive coarse silt entry.

The hydraulic data of Kheowala distributary shows that the channel could not draw authorized discharge due to silting. The remodeling of the channel was planned by redesign of headregulator and introduction of meter flume and a control point. It was anticipated that the channel would work with flatter slopes as compared to existing slopes. Sharma's A.P.M. outlets were adopted to remodel the outlets. During the operation of the channel it was observed that there existed slight. silting tendency and therefore steep slopes were adopted. The remodeling experience of this channel shows that the policy of flattening of slopes did not work in this case and provision of control point for such a short channel was unnecessary.

The remodeling of channel prior to 1934 failed because the proposed remedial measures for remodeling of head regulators were not able to prevent eddy formation and also the proposed skimmer and advanced cill intensified the trouble. The replacement of old type outlets with Kennedy Gauge outlets did not improve the condition to overcome tail shortage. The main cause of failure was the attempt to deal with one aspect of trouble only. If the history of the channel had been studied and complete remodeling carried out, success would have been attained in many cases.

The planning of remodeling of a channel involves hydraulic survey which requires establishing reliable bench marks for leveling survey. The longitudinal section and cross-sections are plotted. The study of previous history of the channel provides useful information to prepare a strategy for remodeling. The remodeling of masonry structures such as headregulator is carried out first and the channel is regarded to proposed slopes. The channel is allowed to work for a full crop period before remodeling the outlets, even though it may become necessary to run the channel with higher discharges. The crest level of the outlets is set as low as possible to increase their silt induction. However, in some cases there will be constraint to this setting depending upon the availability of the working head. It is necessary to inspect the outlets and ascertain the problems of irrigators to evolve proper remedial measures. The remodeling scheme should include all works that are necessary to make the channel work efficiently.

Raising and strengthening of banks should be based upon discharge of the channel. Earthwork should be measured by bank measurements.

Strong banks and liberal berms should be provided. Artificial construction of the channel should only be restored when the section is abnormally wide. Hanging bushing spurs have not been found to be advisable and the only suitable method of berm formation is by longitudinal bushing. Silt clearance of channel is too performed to the full existing bed width. If a channel is wider than the designed bed width it must not be silt cleared to the designed bed level, otherwise the water levels in the channel would be lowered. After remodeling of the channel there may be berm formation at tail during the summer season, but this does not mean failure as the normal water levels can be restored by clearing the tail. The monitoring of outlet is to be done by observing their discharge by specially designed portable flumes. In some cases the outlets would not draw their authorized discharge due to lowering of water levels and therefore in such cases auxiliary pipes should be providing to cater for the demands of irrigators.

The suggested policy for remodeling on the Lower Chenab Canal system is based on remodeling experience and the canal operation. The full supply level of as many channels as possible in a system should be fixed at a definite level and maintained at this level by silt clearance. The water levels should not be raised unnecessarily as it will result in extra expenditure on raising of banks. Command of high patches of area with further raising of water level should not be allowed in any case. This policy will reduce the expenditure on remodeling of outlets. In some cases the channel may show scouring trend and this problem can be tackled by introduction of control points. This policy might be adopted on the Lower Jhelum and Lower Bari Doab Canals after examining the local conditions.

As a general principle every distributary should be made to draw as much coarse silt as it can take without interfering with its regime. Drastic silt exclusion with skimmer headregulator should only be permitted in exceptional cases. The experience in the West Circle of the Lower Chenab Canal shows that the tail distributaries of the Jhang

Branch Upper have silted badly. Open flume type headregulators have proved most satisfactory as most of the channels that have been remodeled are now in fairly stable regime.

The head regulators for Awagat and Kheowala distributaries were designed in such a way that the mean velocity in the flume of the regulator would be approximately equal to or slightly lesser than the mean velocity in the side segment of the parent channel. The gate is placed on the distributary side of the bridge so that the high velocity under the gate should have no effect on the velocity at the mouth of flume. The experiments were conducted to ascertain the distribution of silt in the regulators bays which showed that medium silt was distributed uniformly whereas coarse silt was less near the sides than the centre. Further research on model experiment would give us information about the silt drawing capacity of head regulators. Material times and control points should be installed at suitable location to monitor and regulate the flow. An appropriate type of structure is broad crested flume which requires less working head for its modularity. Its main advantage is formation of standing wave near the crest and wave action downstream is reduced to a minimum. Its discharge coefficient varies from 2.95 to 3.05 depending upon head above the flume and its geometric profile.

The hydraulic data of the channel and outlets should be prepared on longitudinal section with a horizontal and vertical scale of 1"= 1 mile and 1/50 respectively. Existing and proposed water levels and bed levels should be indicated in different colours. The Superintending Engineer of the circle should exercise his control in regard to timely observations and correct methodology and record keeping of hydraulic data of the channel and outlets.

Paper No. 235
Year 1940

THE FORMATION AND THE RECLAMATION OF THUR LANDS IN THE PUNJAB

By

M.L. MEHTA

Paper No. 235
Year 1940

THE FORMATION AND THE RECLAMATION OF THUR LANDS IN THE PUNJAB

By

M.L. MEHTA

The main object of this paper is to discuss the problem of salt in soils with reference to land deterioration as it affects the Government revenue and the Zemindar's income. Although the problem of Thur or Kallar was brought to the notice of the Punjab Government as early as 1908, the regular survey of affected land was carried out in 1972. It showed the problem of Thur as serious as that of water logging. Fields were considered to be damaged if their cultivation had been abandoned or their crop production had fallen below four anna crop. The presence of a white efflorescence over pulverized or swollen earth was the type of thur to be taken into consideration. It was estimated that 5 lakhs of acres had been abandoned for cultivation due to thur, the present rate of deterioration being 25000 acres per annum.

The source of the salt responsible for thur formation was believed to be the water-table. Thur appeared as a result of evaporation and transpiration at the soil crust surface. Thur studies therefore were confined to areas of high water-table. In America attention had been devoted to the addition of salt to the land by irrigation water. In order to investigate the real causes of thur formation on lands, soil surveys were carried out in the Lower Chenab Canal area where water table depth varied from 9 ft. to 40 ft. This study indicated that water-table might not be an essential factor contributing to the formation of thur. For further investigations the soil profiles were studied in fields where thur has appeared at the surface and the adjoining fields in which normal crops

were grown. In thur lands, the salt was present throughout the depth of soil crust up to 10 ft. and the main zone of accumulation of salt was 0 to 5 ft. These profiles demonstrated that both in the irrigated and un-irrigated areas salts were generally present in the soil crust. The presence of salts in the soil under un-irrigated land showed that their occurrence could not be due to addition made by irrigation water, but was a characteristic of original constituent of the soil crust. Formation of thur is dependent upon the original salt content of the soil crust.

Investigations by Puri indicate that in areas of rising water-table, if the soil crust is more than 10 ft. thick, the rise stops when the water-table touches the soil crust. This study leads to the conclusion that the water-table is unlikely to contribute salts to the soil surface when it is situated at a depth of 10 ft. Soil profiles in high water-table area, that had not gone out of cultivation, generally show a zone of salt accumulation below the surface indicating that even in high water- table areas, water-table itself is not such an important factor in thur formation as was once supposed.

The Irrigation practices in the fields under present delta are tilting the balance of salt movement towards the soil surface. As soon as the zone of accumulation of salt approaches such a depth from the surface that the concentration of salts can occur due to evaporation then the land becomes thur. It has been observed that the salt movement is seasonal and in the rabi season the conditions are favourable for appearance of salt at the surface whereas in monsoons, salt zone is depressed.

Before the commencement of irrigation, salt is distributed throughout the depth of soil crust and in the absence of irrigation water no movement takes place. With the introduction of irrigation the salts accumulate in a zone below the natural surface. The subsequent position of this zone depends upon the intensity of irrigation and the type of crop. If the quantity of water used is insufficient to balance the losses due to evaporation and transpiration and the zone of accumulation is within 10 feet of the surface then the tendency of this zone would be to move upwards. Experiments were started in the field near Jaranwala to examine the effect of irrigation pattern under various crops on the salt movement in soil crust. With the irrigation applied to cotton, salts have been removed from the surface and have formed a zone of accumulation at a depth of 6-7 ft. Rice fields are heavily irrigated where the salts have been completely removed from the soil crust.

A soil containing calcium clay is permeable to water and air, and gives good crop yield. When a solution of sodium sulphate or sodium chloride is brought in contact with calcium clay soil, Base Exchange takes place, resulting in sodium clay which is alkaline in nature. This type of soil is not suitable for the growth of normal crops. The following standards were laid down for soil classification purpose based on salt content, pH value and the yield of crop;

Type 1. Soils which are likely to give normal yield of crops have a salt content of 0.2% and pH value not higher than 8.5

Type 2. Soils which will tend to reduce the yield of crops below normal have salt content below 0.2% and pH value ranging between 8.5 and 9.0.

Type 3. Soils which are suited for limited type of cropping in the initial stages of irrigation. Their salt content is less than 0.5% and pH value between 9.0 & 9.5.

Type 4. Salt soils that can be economically reclaimed have salt content above 0.2% while pH value below 9.0

Type 5. Salt and alkaline soils which are difficult and expensive to reclaim having pH value always higher than 9.5.

The experiments were conducted at Chakanwali, Renala areas to find the methods for reclamation of the various types of soils and the financial aspects involved. It was found that the most suitable type of drainage method in high water table area is the open type drains. They kept the water table in motion which in turn removed the salts from the fields, increased soil aeration and thus allowed the normal crops to grow. The thur type of soils could be reclaimed with one or two rice crops but the rakkar type could be reclaimed in less than four Kharif seasons. It was estimated that the net cost to reclaim the land was Rs. 42 per acre and the land once reclaimed would remain fit for cultivation for a period of seven years. A financial study of commercial reclamation of 5000 acres block of deteriorated land shows that if there are no

calamities such as hail or crop disease, there will be a net profit of Rs. 30, 172 at the end of reclamation operation. At Renala it is observed that the leaching of thur land is not likely to cause any damage to the adjoining lands, and the local rise in water-table is a temporary phase.

The attack of rice borer caused reduction in crop yield and financial loss to the reclamation measure. Therefore, a new variety of rice "Sathra" was introduced in Kharif 1938 at Kala Shah Kaku and in areas of Lower Chenab Canal and Lower Bari Doab Canal for reclamation purpose. It was experienced that this variety could stand a relatively higher salt content, needed less irrigation water, and gave a heavy yield. Two rice crops could be obtained resulting in reduction in overall reclamation cost. Reclamation through rice required extra water and therefore arrangements were made to frame new warabandi and excess water was supplied at the rate of 50 acres per cusec. The Zamindars are now becoming aware of the benefits of reclamation procedures evolved through these experiments and are willing to apply them on their thur lands.

The reclamation procedure of land involves leveling of the area and construction of drainage system. The area is divided into 1/4 acre plots with the main water course in the centre. If the water table is below 6 ft. no seepage drains are provided. In April water is allowed to stand in the field to a depth of four inches. In May when salts are washed down, the Sathra variety of rice is grown. The heavy irrigation eliminates the salts from the soil crust and the alkalinity removed by the action of roots of rice crop. The carbon dioxide formed by roots converts the sodium sulphate into sodium bicarbonate and the soil becomes permeable. In order to re-establish the nitrogen balance in the soil leguminous crops like gram, berseem are grown following rice crops. In a seep water-table area if the salts have been washed either to a sand layer or with 10 ft deep soil crust, it is considered that the land had been permanently reclaimed. If there exists a zone of accumulation within a 10 ft. deep soil crust, then the reclamation should be regarded as temporary.

The difficulties encountered in the reclamation of thur lands are the non-availability of extra supply in the distributaries during Kharif season and in case the extra water is allowed to run in the channel, the tail out areas are unable to get their due share of irrigation supplies

because of over withdrawal by head reach outlets. To overcome such problems it is essential to consider reclamation of land at the early stage of appearance of thur when less water is required to reclaim the land and the use of Gibbs module in head reaches. In large areas of thur land where no field bund exists, the problem arises from interference of run off containing saltish water with the progress of reclamation operation in the adjoining lands. In soils with high salt content and low degree of alkalization, leaching is very rapid resulting in loss of irrigation water and low crop yield. The barrani hill varieties have been introduced to overcome this problem.

It is necessary to determine the salt content of soil through soil surveys to ascertain the degree of deterioration of land under cultivation. To prevent thur formation, it is necessary to reduce intensity of irrigation and increase delta with the introduction of suitable variety of rice crop. A subject which is now receiving considerable attention is the water requirement of crops and its relation with the reclamation of deteriorated lands. It is suggested that the agricultural system of deteriorated lands should be altered temporarily to include rice in crop rotations and the irrigation supplies should be enhanced.

Paper No. 245
Year 1941

RAINFALL RUNOFF

By

S.D. KHANGAR AND N.D. GULHATI

Paper No. 245
Year 1941

RAINFALL RUNOFF

By

S.D. KHANGAR AND N.D. GULHATI

The design of storage and drainage schemes requires evaluation of rainfall runoff from a given catchment area. In 1885 the design of inlet structures and syphons on the Lower Chanab Canal was based on 31 cusecs per square mile or catchment area. However with the passage of time the intensity of flood reaching these works has fallen. It is thus necessary to have a definite solution for calculation of runoff. To determine the volume of runoff for storage schemes, it is also required to know the frequency with which a rainfall of a given intensity would occur.

Meteorological department keeps record of rainfall by means of Symon's Rain Gauge. Punjab Irrigation department also maintains a number of Rain Gauge Stations. These rainfall records only supply total daily rainfall, but give no information of the duration or intensity. Since 1930, an Integrating Rain Gauge has been installed at Lahore which record variation of storm with time. A mere record of total daily rainfall without knowledge of its duration and time cannot be of much use in calculating probable runoff.

In the absence of adequate rainfall record, indirect methods for obtaining the probable form of rainfall curves have to be applied. All rainfall curves for any total rainfall will generally fall below the graph of the most intense rainfall. The graph of the heaviest storm obtained by Integrating Rain Gauge at Lahore conforms to the graph obtained analytically for the most intense rainfall.

Intensity of rainfall varies from place to place. The rainfall record at

Charapunji for different stations leads to the following conclusions:

1. That the rainfall recorded at any rain gauge is a true index of the intensity of the storm for a limited area around it, and
2. That there may be areas between neighbouring rain gauge stations that may receive no rainfall are all, or very much in excess of, or less than that recorded by any of those stations.

Various investigations have been made for correlating variation in the intensity of rainfall with area but these have not led to any definite results.

Estimation of total rainfall in an area can be obtained by closely and uniformly spaced rain gauge stations. It is recommended that rain gauge stations in the irrigation areas of Punjab should be spaced not more than 7 miles apart in either direction, and atleast 10% should be of the integrating type.

Rain gauges are installed in the open. The rainfall that can produce runoff is always less than that recorded by a rain gauge. No accurate estimate exists of the quantity which should be deducted on this account. From observations made in America it was concluded that 70% of the rainfall recorded in open, reaches the ground the amount of water held by foliage crops will vary with:

(i) The intensity of rainfall
(ii) wind action during and after the storm
(iii) Thickness and nature of foliage and kind of crop

The losses are due to evaporation, transpiration by plants and absorption into subsoil.

Thus, Runoff = Ground rainfall - Absorption and Evaporation. Buckley's and Harrington experimentally estimated that evaporation and transpiration losses are of the order of 1/100 and 1/130 inch per hour respectively. The loss due to infiltration into the soil is the principal loss. Kennedy observed by experiments that absorption losses

vary considerably for different kinds of soils. Also the absorption losses decrease with the duration of flood. In general absorption losses depend on temperature changes, packing of soil, soil moisture content, shrinkage and swelling of soil. Further work in this area is required to establish the infiltration rate.

Topography of the catchment area also affects the rainfall runoff in different ways. When the width of catchment measured from the drain to the watershed is narrow, it results in higher intensity runoff as compared with wider catchment. A steep slope has the same effect. The effect of natural or artificial pondage in the area is considerable. In cultivated areas field dowels provide immense storage capacity depending upon the height and strength of the dowels and sometimes there is no runoff.

To determine runoff for any storm the factors involved are so varied and interdependent that only a simplified case under ideal conditions can be considered. The assumptions are that area considered is small, absorption losses and velocity of storm are uniform over the catchment area, there is no vegetation, natural or artificial pondage and the quantity of water flowing over the catchment during the storm is ignored. The area is divided into a number of strips and sheet flow runoff is assumed although actual flow is in the form of small streams and there are obstructions to flow. The runoff is then determined by drawing graphs of the rainfall in strips against time. The design discharge of a drain should generally be based on intensities likely to occur once in three years. At present no graph of rainfall exists to establish the frequencies of rainfall of various degrees and therefore indirect methods have to be adopted for determining the probable runoff. In rivers, a discharge which has a frequency once in three years is about 1/4th to 1/3rd of the maximum flood discharge ever recorded. On this analogy, from the maximum intensity rainfall hydrograph, the discharge for a drain may be designed.

Corrections are to be applied to the simplified case to determine the actual runoff. The correction to absorption losses can be accounted for by a modification of the absorption line along the ideal curve. Difference in infiltration capacity of various soils can be approximated

by weighted average infiltration rate. The velocity of flow in the beginning and at the end of the storm would be less than theoretical whereas the peak flow velocity would be higher than theoretical value. It is not proposed to make any allowance for natural pondage over the catchment as the initial pondage may be nil. For the application of correction on account of cultivation, it is considered that banjar areas are wholly effective, while 10°, of canal irrigated area may be considered equivalent to banjar area, and 80% of barani and chahi area may be regarded as banjar area. Experience shows that effective area of drain in flat irrigated tract does not exceed a strip of 1.5 miles width on either side of drain. The determination of average height of -hydrograph depends upon inlet time. To solve a particular problem it is necessary to first assume a value of time, draw a hydrograph, determine height of hydrograph and then recalculate time. If the calculated time does not agree with the assumed value, repeat the process until the difference is negligible.

When considering large catchment area, the intensity of rainfall may not be uniform and the flood discharge flattens out as it proceeds down a drain. The actual flattening that occurs in a particular case would depend upon the intensity of discharge, the duration of peak, size of the channel and amount of spills form the channel. The runoff per square mile for larger area would therefore be less than that for smaller catchment. It has been analyzed that runoff varies as two-third power of the area.

It is suggested that the number of rain gauge stations and integrating rain gauges should be increased to determine more accurately the frequency of storm of high intensity. Some observations are necessary to have a good estimate of the inlet time. The observation of runoff should be extended to areas other than Lower Chanab Canal, presently being done to establish a general equation for Punjab drainage system.

Note:

Paper No. 246 appeared in the Proceedings of Engineering Congress 1941, Vol. XXIX at pages 129 to 173. This lengthy paper has 8 Plates and 18 Figures. Discussions are recorded at 27 pages from 174a to 174aa. The paper has detailed formulae and arguments to which the reader may refer.

Paper No. 251
Year 1942

THE KALABAGH BARRAGE

By

S.I. MAHBUB

Paper No. 251
Year 1942

THE KALABAGH BARRAGE

By

S.I. MAHBUB

Kalabagh Barrage was constructed for feeding the channels to irrigate the Doab between the rivers Indus and Jhelum called the Thal Project. Gross Command area by this barrage is estimated as 19.3 lac acres. According to 1940 estimates, the total cost would be Rs. 7.72 crores of which the headworks will cost Rs. 1.88 crores.

Pakki Shah village on left bank was selected as the site of headworks in. 1936 because of certainty of correct cost estimation, facility of construction, shingle bed foundation, low afflux and easy access by road as well as railway etc. Maximum design discharge for the barrage has been taken as 9,50,000 cusecs after analysis of past data. There is enough freeboard to pass a super flood of 11,00,000 cusecs with increased afflux. The maximum discharge actually encountered has been 818,510 cusecs on 29.8.29. The canal has been designed to take upto 10,000 cusecs with a pond level of 694. An afflux of 15% or 3 on the normal flood level of 693 is considered suitable, giving H.F.L. upstream of 696. Excessive retrogression downstream of the weir is ruled out because of underlying shingle bed and the smallness of pond. It is taken as 2 as against 4 to 5 for other barrages. This gives a minimum downstream level of 672.00 and a maximum cross head of 22 for which the barrage is designed.

Width between abutments is 3797 based on Lacey's relationship with an average intensity of 290 for 11 lac cusecs discharge. The waterway consists of 56 spans, each of 60' clear, with 7 piers and 2 divide walls of 25 each. Undersluices, 14 spans of 60, each are provided to facilitate the diversion of river over the completed barrage, allow the unwatering of

weir for subsequent maintenance and inspection, and help silt control into the canal by the formation of a deep channel near its off- take in which low velocities of approach could be secured. The crest level of undersluices is RL 675 while the well crest level is RL. 678' Cistern levels have been fixed at RL 667 and RL 670 for undersluices and weir portions respectively. The length of cistern was calculated and provided as 70' for weir portion and 75' for undersluices. Low cistern levels and length are provided to ensure that any hypercritical flow from the weir does not pass beyond it.

The position of standing wave is stable on a sloping glacis. More intense wave is produced on flatter glacies and the range of the trough requiring heavy thickness of floor will also be greater. A series of experiments were conducted in Research Institute and a slope of 1 : 4 was adopted on the upstream side and 1 : 3 on the downstream side. The crest width of 6' is fixed. The total length of the weir floor is 140' and that of undersluices 150'.

The river has a shingle bed for several miles above the weir site. Therefore 18" quartzite sets instead of concrete are proposed for facing/pitching in those portions of the section which will be subject to movement of shingle at higher velocities. Staggered friction blocks are suggested to be provided in trapezoidal rows. Staggered friction blocks are suggested to be provided in trapezoidal rows. Originally, three lines of sheet piles, viz; at upstream side, at downstream side and at the toe of the glacis were proposed but because of shortage of piles due to War, it is decided to provide one line of 7.5 piles on the downstream end. Cut-offs walls are provided on other locations. An exit gradient of 0.289, giving a factor of safety of 3.46, is calculated and it is considered to be quite safe for a shingle bed. The pressures under the floor are determined by reading off from the curves based on the mathematical solutions for elementary forms and are subsequently checked by the Research Institute.

The gravity section is preferred to raft design as shingle is locally available, making the gravity section much more economical. Inverted filters, flexible protection, deep pier foundations and flank walls are provided by using standard practices and designs. In the beginning, it was considered to provide all the 14 undersluices bays on the right side

as the river after leaving the railway bridge upstream hugs the right bank. Later on however half of them are provided on right side and half on the left side for ease of unwatering, repairing and river control.

Divide wall is provided in a headworks to form a deep channel and it controls silt entry into the canals. From the examination of data, a 300' long divide wall has been found to be the best for least silt entry into the canal. Silt excluders and silt extractors are the devices for the control of silt in canals and these are provided in the headworks and canals respectively. Khanki type silt excluder was found to be more efficient and hence is adopted. Silt excluders prevent the entry of coarse rolling silt and a proportion of the suspended silt into the canal, while the silt extractors draw out or eject the suspended heavy silt from the canal. A regulator is proposed to be provided at RD 3300 of the canal in order to maintain optimum water level in the reach above it and a series of five extractors are provided in this reach. The head regulator is designed to take 8160 cusecs with pond level RL 692. With the pond level at RL 694 however, it can pass 4000 cusecs extra.

A 25 roadway is provided over the regulator while a 10 Arterial Road Bridge with one 2.5 foot walk-over the barrage is finally chosen. This bridge will serve as a road link between Punjab and NWFP. Diverging type of guide banks are proposed, as these ensure a smoother entrance and reduce the chances of lateral flow. Top levels of the marginal bunds are kept 2' higher than the guide banks so as to allow for rise in the level at the guide bank noses. A T-head spur is also provided to protect the left marginal bund. Gates and gearings were manufactured in the Central Workshops at Amritser.

Two divisions, with five sub-divisions were responsible for the construction of Kalabagh Barrage. Power Division looked after the power house and workshops whereas construction of headworks, supply of material, railway and quarry was the responsibilities of Kalabagh Division. 300 tenders were received on the basis of revised form of Haveli Schedule of rates and the average of the rates tendered approximated very closely to the schedule. Land acquisition was done by special Land Acquisition Officer.

Pitching stone was obtained from Paikhel quarry but because of certain limitations, supplement supply from Sikhanwala quarry was necessary. Stone sets were acquired from Nowshehra and Abbotabad. The supply of shingle was arranged from a shingle quarry about 2 to 3 miles from the weir. About 120 lac bricks were obtained from the government kilns and about 80 lac purchased locally. To cope with power needs and the heavy repair work of the plant in use, power capacity of 900 K.W was provided. A programme of work was drawn and 1941-42 was fixed for the completion of the barrage.

Theodolites were fixed on two high pillars on both sides of weir line for checking levels. The cill girders and grooves were also aligned from these. A 10' width on the upstream, downstream and flanks for possible drains etc., was added to the designed sections for excavation. General methods of well sinking, which are good for a sandy soil, failed for the shingle bed as the usual sand grab proved to be an utter failure in this case. The method finally adopted was to unwater the well approximately to curve level as far as possible and excavate the shingle inside and outside the well manually.

Two big concrete mixers were used. Batching was done on volume basis. Slump tests were carried out every morning in order to get rough guide for the water quantity but final adjustment was generally done by trial after seeing the workability of the concrete at site. Kalabagh barrage is the first major work where almost all of the concrete was mechanically vibrated. Curing of weir floor was done by pipe line fitted with pumps at suitable intervals where large mattresses made of gunny bags were used for the curing of divide walls and friction blocks. Extensive form-work was avoided because of the use of precast shells in the weir. Ordinary wooden or brick shuttering etc., were the general types used for various other works.

To check the safety of the work, a large number of pressure pipes were installed for measuring actual pressures under the barrage floor. These observations indicated that the actual pressures were 10 to 20% higher than those theoretically assumed in the design. However, these results were doubtful as observations were taken at low head and value was not assumed correctly. Other wrong assumptions were also considered in the pressures measurement and it was decided that the weighting of the

floor or its extension was not necessary at present. However, it was stressed that no hollows should remain under the work and grouting must be done with great care and the pressures should be kept under observations.

A number of precautions, in addition to more extensive grouting, were taken. The shingle in bays 1 to 7 of left undersluices and 8 to 13 of the weir was excavated to the level upto which there was any possibility of runnel formation and replaced by pure sand. All cross walls were taken down to rest on undisturbed soil and all drainage water was passed through bajri filters to avoid piping. The order of pouring the concrete blocks was such as to do the lowest level work first. This was rigidly observed. Lowering of the sheet pile line in left under-sluices was also done.

Note:

Paper No. 251 appeared at pages 1 to 66 of the Proceedings of Engineering Congress 1942, Vol : XXX. It has 10 plates the discussions on the paper are recorded at pages 251 a to 251 w, and mainly concern formulae and assumptions used in the paper. For details the interested reader may refer to the full paper.

Paper No. 260
Year 1943

LINING OF CHANNELS

By

S.I. MAHBUB

Paper No. 260
Year 1943

LINING OF CHANNELS

By

S.I. MAHBUB

The main advantages of lining a channel are saving in irrigation water, avoiding water-logging, stability of the section and reduction in maintenance cost. Improvement of command owing to flatter slopes is also possible. Two major lining schemes i.e. lining Gang Canal with concrete lining and lining Haveli Main Line with brick lining have been completed. There still, however, remains a controversy regarding the most effective and economical method of preventing absorption losses through the irrigation channels.

Financial analysis of lined channel scheme has been made in regard to the benefits achieved from savings in water. This is based on the presumption that the area irrigated would increase proportionally with the increase in supply. It has been estimated that expenditure incurred to effect saving of 1 cusec of capacity for the three cases given hereafter would be Rs. 113,000, 65,000 and 30,000 respectively with 6% return. The categories are:

(i) Water saved utilized in Crown waste land on temporary cultivation.

(ii) Water saved utilized in Crown waste lands which are sold

(iii) Water saved utilized in areas already receiving irrigation.

These calculations do not take into account the indirect benefits derived from prevention of water-logging.

The design of lined channel is governed by the permeability, coefficient of rugosity, durability, cost of construction and maintenance. The permeability of material determines absorption losses and the co-efficient of rugosity determines the carrying capacity of the channel. Weathering is caused by disruptive action of temperature variation, alternate freezing and thawing, and wetting and drying. The alkali soil causes corrosion of concrete and this can be prevented by the application of sulphate resistant cement. Cost of construction would vary with the locality and the availability of various materials. The lined section should be structurally stable. In reinforced concrete lining the reinforcement is designed to reduce the size of contraction cracks and to prevent damage due to settlement of subgrade, but it may delay the relief obtained through local failure in small patches. This was the main cause of failure of Haveli Main line resulting from back pressure of water. The side slope of the lined channel should be the same as the angle of repose of the retained soil. The thickness of lined section would depend upon the lining material, the side slope and the existence of hydro-static pressure. There should be proper drainage system to prevent failure due to back pressure.

Concrete lining is durable if laid properly and the absorption losses are reduced by 95%. The coefficient of rugosity is low and in view of high velocities possible, the section is reduced. The construction is carried out in panels and grooves are provided to prevent cracks due to shrinkage and alternate expansion and contraction. Oil paper, crude oil, 1 : 6 cement plaster or 1 : 4 cement sand slurry are used at the top of subgrade to avoid its becoming spongy and permeable. A greater control on the manufacture of concrete is possible through slump tests.
Cement mortar lining is not very durable unless suitably protected and as such can only be used in conjunction with some other protective material. Stone masonary has a limited application mainly on account of its cost and can thus only be used where stone is locally available. Road oil lining is not durable nor it is effective and the coefficient of rugosity is high. Sodium carbonate lining has been used in water courses and small channels, but it's useful life is not more than two to three seasons.

Clay puddle lining reduces seepage losses by about 80%. The quality of puddle can be judged by its dry bulk density, which is a measure of its compaction. It has been shown that there is optimum moisture content

for each soil at which the dry bulk density obtained is maximum. This optimum moisture content is determined in the laboratory by compaction test, or approximately by formula devised by the author. The clay puddle is compacted in 6 inch layers at optimum moisture content by the use of toothed rollers.

Brick lining was used on a large scale for the first time in America in 1933. It was adopted with suitable modification for Haveli Canal in 1937. This lining failed due to inadequate compaction of the back-fill, lack of proper drainage of banks and insufficient free board. The absorption losses QA in an unlined channel are given by

$$QA = 0.0133\, LQ^{0.05625}$$

Where Q is the discharge in cusecs and L is the length of the channel reach. The experience at Haveli Main line and Gang Canal show that the absorption losses in lined canal are of the order of 1.5 cusec per million square feet of wetted perimeter. Absorption losses in a brick lined channel can be estimated from Haigh's formula:

$$K = 1.25 \times Q^{0.056}$$

Where "K" is the absorption loss per million square feet of wetted perimeter.

Haveli Canal was designed with Mannings N of 0.0146 whereas its observed value varies between 0.018 and 0.02 which is the result of sand blown in or brought in from the head and the presence of caddis worm in large number. In future a higher coefficient for brick lining say 0.018 should be adopted along with adequate free board. It is also proposed to use 10" x 4.87" x 2.75" bricks in place of tiles. The use of larger bricks means saving in mortar and low rate of expansion and contraction.

Certain precautions are required in brick lining. The salt content of earth used for brick manufacturing should be not more than 0.3%, fineness modules preferably not less than 1.2. It should be free from organic impurities and excessive silt. The consistency of mortar should be regulated by slump tests. The plaster should be allowed to set properly and the subgrade should by properly moistened and oiled.

Brick lining as compared to concrete lining does not require specialized labour, no elaborate or expensive equipment is needed, contraction cracks and buckling caused by expansion is reduced. The thickness of lining is controlled by the thickness of bricks and repairs, when necessary, can be carried out easily.

Mr. Haigh carried out certain experiments on various types of brick lining to decide on a suitable lining for distributaries. The following types were tried on Hassuwali distributary:

> Type I : 25" thick tile masonry in 1 : 3 mortar laid on cured and dry 3/8" thick cement plaster with G.I wire No 10 as reinforcement.
> Type II: 2.5" thick tile masonry in 1 : 3 mortar laid on cured and dry 3/8" thick cement plaster without reinforcement.
>
> Type III: 2.5" thick tile masonry in 1 : 3 mortar laid on green cement plaster with Maxwell fabric reinforcement.
>
> Type IV: 2.5" thick tile masonry over 3/8" thick cured and dry cement plaster with Maxwell fabric reinforcement

It was observed that based on measurement of absorption losses and the cost of construction, Type II is preferable as compared to other types.
Precast cement concrete blocks were tried to repair the damaged lining of Haveli Canal. The cost worked out to be Rs. 56/10/- against Rs. 25/5/- per 100 sqft. for tile masonry. These units have the advantage of facility in construction, structural strength and durability, low coefficient of rugosity and high degree of impermeability. The main drawback is its high cost. A slab and beam system was also tried in a short reach, but this was found impracticable. The permeability of bitumen impregnated cloth protected by Masonry was tested by the author and the losses remained under 1 cusec per million square feet. However experiments indicated that under high hydraulic pressure this type of material deteriorated with time.

The back fill material should be compacted by toothed roller at optimum moisture content before lining the channel bed. Maximum dry

bulk density would be obtained if the soil contains 70% sand and 10 to 20% clay. It is desirable to aim at compaction with a minimum of 110% of the dry bulk density of the natural soil in the locality. Suitable drain should be provided at the toe of the bank along with proper berm width and the dowel. The bank should slope outwards. In areas with high spring level, a continuous inverted filter, a system of drains or porous galleries or a system of vertical relief pipes may be provided depending upon cost and site conditions. The best form of lining section would be an arc having sloping sides, more or less at the same slope as the angle of repose of the soil. This may be possible for channels upto 2000 cusecs. For larger channels similar side slopes with flatbed are designed. In case of lining of existing canals, it is advisable to construct a new lined channel along the existing one.

Note:

Paper No. 260 appeared at pages 8 to 37 of the Proceedings of Punjab Engineering Congress 1943 Vol. XXXI. It has 7 Plates. Lengthy discussions on the paper at pages 37a to 37z and pages 37aa to 3'7ff.

Paper No. 261
Year 1943

THE CONSTRUCTION OF A MOTOR ROAD ROUND SIMLA

W.A.R. BAKER AND BALWANT SINGH

Paper No. 261
Year 1943

THE CONSTRUCTION OF A MOTOR ROAD ROUND SIMLA

By

W.A.R. BAKER AND BALWANT SINGH

The density of population in Simla is stated to be five times as great as anywhere else in Punjab. Many schemes for the improvement of civic amenities at Simla were prepared but none of them matured. The present scheme envisages improvement of water supply and sewerage system, an experimental scheme of housing for migratory coolies and the construction of a motor road round the town. The first three items are desirable from the health point of view. It is expected that the motor road would reduce the demand and the number of coolies, cut out the influx of mules and mule-men from the Hindustan-Tibet road by improving transport and lorry traffic, and encourage the development of suburbs.

There have been two suggested routes for a road, known as "Wadley" and "Dorman". Wadley route is further away from the centres of population and might be preferable for relieving the present over-crowding. Dorman route is shorter and more accessible from the principal areas, while the localities which it opens up are probably more attractive for prospective development. It was finally decided in 1941 to construct the road on the Darman alignment.

The general specifications are that the roadway consists of 18' side formation, increasing to 20' on bends, with maximum gradient 1 in 10 but short sections upto 1 in 8 gradients would be acceptable. The radius of bends is not to be less than 50' at the centre line of the road. All road bridges and culverts are to be capable of carrying the Indian Roads Congress standard loading. The road crust consists of 3" thickness of

hard sand-stone ballast over 3" of local stone soling.

Surface treatment consists of two coats of tar and chippings. Retaining walls to be built are of dry stone masonry as in other hill stations in Punjab. Where space prohibited the standard type of wall with its battered face, vertical wall with 1 : 6 cement mortar was adopted. Its cost was about double that of dry stone masonry. Maximum super elevation was limited to 1 in 12. Vertical curves were designed to provide easy riding qualities at changes of grade. Transition curves were not provided because of the topography.

Coursed rubble masonry in cement sand mortar was considered suitable for the abutments and flooring of culverts, but in view of poor class of masons available in Simla, brick arches were adopted in all cases except for certain small culverts. Principal structures include one tunnel, 3 single span bridges and 4 viaducts. The site of the tunnel was selected on account of the short length (160 ft) through the ridge at this point and on account of easy approaches. It was decided to scrap the two existing cumbersome and weak Nala Bridges and the Belvedere Bridge because these were unable to meet the specifications of the Indian Road Congress, and to build new bridges. The concrete slab of the new Belvedere bridge is curved in plan to meet the approaches and is super elevated. All the viaducts are curved in plan to varying extents, and are on gradients.

Materials used and stresses allowed were as per standard P.W.D. specifications. The controversial item is likely to be the method of construction of bridge abutments and piers i.e. a brick facing, with a filling of 1:5:12 concrete. But this construction is a natural development of the normal form of construction of brickwork in 1:6 cement mortar. Moreover solid brickwork would be completely cost prohibitive at Simla.

The estimated cost of the work is about Rs. 12 lakh. Survey work was completed in 1941 and design and estimates were under preparation to commence construction immediately. Tenders were called but contractors were hesitant to commit themselves to contracts during a period of fluctuating market except at very handsome rates. Therefore

the work was let out on work order basis. Supply of building stone also proved difficult. As long as petrol coupons for Lorries were available, quarries were worked but now as petrol coupons are unavailable, transportation of stone has become very difficult. Even mules, bullock-carts owners etc. are hesitant to replace their normal loads of potatoes, mangoes etc. with stones due to freight rate. When central P.W.D. started its works in December 1941, competition for material and labour arose and created more difficulties for the road project. These are some of the difficulties under which the project work proceeded until June 1942 when cement supplies were totally suspended resulting in damage to many unfinished culverts.

This paper has been written when the project is half-way through. It has been attempted to cover all the salient features and important design work. Progress has, for the reasons outlined, been slow, but it is expected that the road will be open to traffic by the summer of 1943. It has been endeavoured to keep down the costs, and in no way encourage any war-time tendencies for very high rates. In spite of the best efforts, not much success has been achieved in working to anywhere near peacetime rates.

Note:-

Paper No 216 appears in the proceedings of Engineering Congress 1943, Vol. XXXI at pages 39 to 54. It has 9 photographs and 8 plates. Discussions are recorded at pages 54a to 54u.

Paper No. 264
Year 1944

IRRIGATION OUTLETS

S.I. MAHBUB & N.D. GULHATI

Paper No. 264
Year 1944

IRRIGATION OUTLETS

By

S.I. MAHBUB & N.D. GULHATI

The success of an irrigation enterprise would depend upon the success with which irrigation water could be supplied whenever needed by crops. There are three main methods i.e. Sub surface irrigation, Spray irrigation and Surface irrigation. An outlet is a device at the head of a watercourse to deliver water from the canal. In Punjab about 40,000 outlets irrigate an area of about 14 million acres.

The principle underlying the distribution of water envisages that each cultivator has a uniform proportion of area irrigated in his irrigable area. There are various methods of distribution of water. Continuous flow system of irrigation is only useful to big farmers. In intermittent flow system, the entire discharge from an outlet is taken by different' farmers in turn, the turns being fixed in proportion to the irrigated areas owned by each individual. This system is conducive to economical use of water. For Punjab canals, the distribution and supply of water on demand is impracticable. There is generally no manual control on working of outlets. The internal distribution of water on the farm is managed by cultivators themselves. The assessment of water through outlet by measuring volume is not practicable due to costly measuring devices and silt and debris in water which could block the measuring device. In Punjab, the system of assessment is based on the acreage matured, and on water rates difference for different crops.

There are three main sources of irrigation supplies; rivers, reservoirs and open wells/tubewells. When the source of water is the river, water supply is limited to availability at a particular time. The irrigation supplies may be made in three ways;

(i) By continuously running the various channels with their share of the available supplies.

(ii) By running the big channels continuously with their share and the small channels in rotation.

(iii) By giving authorized full supply discharge to each distributary system in rotation.

In actual practice the procedure adopted is a combination of first and third method.

A properly functioning outlet must be temper proof with low cost, should draw fair share of silt and should work efficiently with a small working head also. The optimum capacity of an outlet should be the discharge which the cultivators can handle efficiently and also that the absorption losses in the water course and in the field are minimum. It has been found that a 2 cusecs outlet is generally the best for the cultivators in Punjab who irrigate in fields of about 1/2 acre size.

Previously temporary outlets were fixed. An earthen ware pipe of 6" standard size was allowed for 100 acres of annual irrigation. Different water allowances were fixed for different areas. The duty of one cusecs varied from 275 to 457 acres. Both rectangular and pipe outlets were in use. All outlets were closed with wooden flaps. In those days tatiling of channel and tatiling of outlets on the same channel was a normal practice. The size of outlets was changed according to whether the area irrigated was in excess or less than the prescribed proportion of the commanded area. The questions raised include setting and geometry of outlet, size of barrel, method of closing the orifice, and when a permanent outlet should be built.

There has been further development in design, manufacture and management of outlets since then. The location of outlet was fixed at the highest point with reference to adjacent commanded area. The size was fixed on the basis of normal full supply factor ranging from 250 to 300 acres. Many officers worked to obtain modular or semi modular

conditions on the outlets. The tatiling of outlets is regarded as an inefficient working of the distribution system. However remodeling of outlets in Punjab is still a problem.

Pipe outlet was adopted in Punjab for the first time on the Chenab canal. It consists of steel or cast iron pipe. Adjustability is obtained by putting the pipe of a larger size and by fitting it with a reducing socket. The pipe outlet set at bed level draws a fair share of silt. It can pass the required discharge with a small working head of even 0.1 ft. Scratchley outlet is a pipe outlet which opens into a cistern 2 to 3 ft square. Pipe outlet behaves as non-modular under submerged condition, but can be designed to act as semi modular outlet if free fall orifice conditions are secured. In 1928 a standing wave pipe outlet was developed. A rateable semi module was also developed known as Kennedy gauge outlet. This outlet could be easily tempered with due to its peculiar structure consisting of a vent pipe. The development process continued. Harvey Stoddard improved irrigation outlet consisted of an adjustable orifice connected by rectangular masonry pipe to a narrow long crested weir, which discharged into a flume. Its minimum modular head was in the range of 15% to 20% of the depth over the weir crest.

The open flume outlet is a development of the idea underlying the Harvey outlet. It consists of a smooth weir with a throat constricted sufficiently to ensure a velocity above the critical, with an expanding flume at the outfall to obtain maximum recover of head. It is not easily adjustable. Proportionality can be secured by keeping the crest of the outlet at 0.7 of the depth of the channel. The minimum modular head lied in the range of 10% to 20% of the head above the crest. Various types of these outlets were developed which include Crumps open flume, Haigh′s and Sharma′s modified open flume, Jamrao type open flume.

An orifice semi module is an orifice provided with an expanding flume. The critical velocity is exceeded in the orifice and thus discharge is independent of water level in the watercourse. Adjustability is secured by raising or lowering the roof block. Proportionality is secured when the bottom of roof block is submerged below the full supply level by 3/10 of the depth of water in the channel. The experience on channel

fitted with proportional orifice semi module shows that the channel is generally silted up. A modification was made in this type of outlet by lowering the setting of outlet close to the channel bed which improved the silt draw and made it more rigid.

It was observed that it was not possible to place the crest of an open flume or an orifice semi module too close to the bed on account of practical difficulties and limitations of available working head. Various devices such as bend outlet, Haigh's silt-extracting S.M.M.O and Gunns nozzle outlet were developed to extract more silt from the channel. Pipe cum semi module has an advantage over other devices as control of silt induction can be achieved.

The earlier attempts to design a module were made in Europe. A series of modules with moving parts were designed in India which include Vishesvarya self-acting module, Kennedy outlet module, Wilkins module, Kent '0' type module and Khanna's auto adjusting orifice distributor. These outlets have little practical utility on a large scale. Some of them are very expensive and not simple to design and construct. In modules without moving parts such as Gibb's, Khanna's O.S.M. and Ghafoor's the constant discharge is automatically regulated by the velocity of water itself. The Gibbs' module was tried in 1909 and it was observed that it was easily tempered with, was costly and had low silt drawing capacity. The other modules were still in experimental stage. To measure the volume of irrigation water through outlet Dethidge meter, Recorder cum semi-module and Patwari cum semi module were tried in the field.

In irrigation channels the discharges and the water levels vary from time to time. Such variations in discharge require proportional outlets. The needs of reclamation or seasonal variation in slope require the use of outlets of low flexibility. Rotational running presumes the use of outlets of high flexibility. Lindlay pointed out in 1923 that proportional distribution is neither necessary nor desirable. He concluded that semi modules with low flexibility can satisfy the needs of the cultivators.
It is difficult to satisfy the opposing conditions of small loss of head in the outlet and efficient silt conduction. Sharma conducted experiments on various types of outlets which showed that a silt conduction of 110

to 115% was obtained by;

(i) Crumps, 0.S.M set at 9/10th

(ii) Sharma's O.S.M. set at $8/10^{th}$

(iii) Crumps open flume set at bed level

(iv) Sharma's modified open flume set at 7/10th.

The available working head can be increased by exclusion of high areas, shifting the site upstream of control point, raising of full supply level and desilting of water courses.

The modules should be used in the case of direct outlets taking off from a branch canal. Non modular outlets should be avoided as far as possible, but where available working head is limited Scratchley type outlet may be used. Wherever possible outlets should be clustered above a control point. The outlet at the tail cluster should be of open flume type. All other outlets should have as low flexibility as possible. This can be secured by A.O.S.M set at bed level or open flumes fitted with roof block. Wherever the section of bank is heavy and other outlets cannot be set at bed level, the pipe cum semi module should be used. On new channels the use of temporary pipe outlet would give definite data on which final construction of outlets could be carried out.

Note:

Paper No. 264 appears in the Proceeding of Engineering Congress 1944 Vol. XXXII at pages 1 to 98. This paper consists of 7 Chapters and five appendices. It has 20 Plates showing the details of various outlets. Discussions are recorded at pages 99 to 125. Interested reader may refer to the original paper.

Paper No. 267
Year 1944

CHIEF CONSIDERATIONS AFFECTING THE DESIGN AND USAGE OF RAILWAY SLEEPERS IN INDIA

By

S.L. KUMAR

Paper No. 267
Year 1944

CHIEF CONSIDERATIONS AFFECTING THE DESIGN AND USAGE OF RAILWAY SLEEPERS IN INDIA

By

S.L. KUMAR

Object of the paper is to bring to the notice of engineers in general and railway engineers in particular the fundamentals of the design and usage of the various types of sleepers in India. An ideal sleeper should be able to distribute load over the ballast evenly in addition to maintaining the correct gauge between a pair of rails. It should be strong and stiff enough to function as a beam with' adequate lateral strength to resist the track distortion under the influence of lateral flange forces. Resistance to creep and adequate bearing area are additional desirable features of a sleeper. It must also be light in weight to facilitate transportation and should be resistant to corrosive effects of the environment. A good sleeper should have the least number of fittings. Basically sleepers are either rigid (one piece) or semi-rigid (the double block).

Wood, steel, cast iron and plain or reinforced concrete are common materials used for sleepers. Wood sleepers are generally preferred over metal sleepers whereas concrete sleepers are rarely used. Metal sleepers are more frequently used in some countries. Apart from technical and economic considerations, prejudices appear to influence selection of a particular material for manufacturing sleepers. In a few countries use of metal sleepers is relatively common. Metal sleepers are more susceptible to environmental attacks than wooden sleepers which have added advantage of better insulation and quieter movement of trains. Excessive weight, higher risks of their damages in case of derailments, and higher maintenance cost are among the disadvantages of the metal

sleepers.

Design of a sleeper is influenced by theoretical as well as practical considerations. Theoretical aspects include behaviour of track and sleeper under load, dispersion of sleeper load through ballast, sleeper spacing and strength of track, relationship between sleeper spacing and axle load, sleeper spacing in relation to the rail joint and impact, effect of sleeper dimensions on its load bearing capacity, effect of the width of rail bearing upon sleeper stability and the lateral strength of track. The designer must treat a sleeper as a beam on elastic supports. The pressure underside a sleeper spreads at an angle of 45 degrees. Sleeper section is a function of its spacing which differs for various axle loads. A sleeper placed close to a rail joint increases the useful life of rail by preventing its repeated bending. Design of sleeper and rail fastenings have an important effect on the lateral strength of the track.

Practical design considerations vary for different types of sleepers. For cast iron sleepers, the weight of the plate, shape and bearing area of sleeper and the effect of tie bar on the lateral stability of a sleeper deserve due consideration. The weight of cast iron sleeper is generally believed to contribute to the stability of the track against wave motion. Its shape practically enhances its ability to hold ballast. Salient practical features of a steel trough sleeper include its wasted shape and provision of baffle which significantly contribute to the compactness of the sleeper. A wood sleeper should be capable of seasoning without excessive splitting, be amenable to treatment, should have sufficient compressive strength and adequate hardness to withstand rail abrasion. Average life of different types of wood sleepers in India is for deodar 18 to 21 years, chir 16 to 18 years, fir and kail: 14 years and other untreated soft wood sleepers 12 years. In America and Britain average life for a treated wood sleeper is over 25 years. Causes of low life of wood sleepers in India are the inadequate section, insufficient treatment and improper protection against mechanical wear, spike killing of wood and defective system of sleeper replacements.

Economic benefits of using wooden sleepers can be enhanced by relaxation of the existing sleeper specifications, exploring type of wood not being tried at present, use of half round sleepers of sal, teak and chir etc. Standardization of two to three different sections of sleepers for any

particular gauge and accelerated seasoning and care in handling can also contribute to the economical use of wood sleepers. Economic considerations alone decide the type of sleeper to be used. The relative merits of any two sleepers should be determined purely from economic considerations based on their initial prices and on their probable lives in the track under identical conditions of service. For this purpose a common criteria of annual cost of a sleeper has been adopted. The annual cost of a sleeper is the sum of the interest charges on the initial cost, the depreciation charges and the maintenance charges.

Total track mileage in India is equally served by wood and metal sleepers. Use of metal sleepers is advocated for economic reasons and due to non-availability of quality wood sleepers in India which, in the opinion of the author, is not justified. The past record shows that popularity of a particular type of sleeper changes with time. In the beginning good wood for sleepers was in abundance but due to indiscriminate cutting of trees and non-development of other forests, wood has become short, and attention was attracted by metal sleepers. During the last 15 years suitable types of steel and cast iron sleepers have been evolved. With proper efforts to implement author's recommendation, wooden sleepers can regain their lost place.

Paper No. 290
Year 1949

STUDIES IN LYSIMETERS

DR. A.G.ASGHAR,
H.S. ZAIDI
M.A. QAYYUM

Paper No. 290
Year 1949

STUDIES IN LYSIMETERS

By

DR. A.G. ASGHAR,
H.S. ZAIDI
M.A. QAYYUM

The paper has been divided in three parts: (i) Influence of the pellicular zone on the proportion of surface application reaching the sub-soil by Dr. A.G. Asghar; (ii) Construction of lysimeter and building up of soil profile by Dr. A.G,. Asghar and M.S. Zaidi; (iii) Influence of the surface application on the sub-soil water table under different crops by Dr. A.G. Asghar, M.S. Zaidi and M.A. Qayyum.

The surface application such as irrigation water and rainfall percolates the subsoil and generates vertical moisture gradient resulting in redistribution of moisture content from surface to the water-table. Similar is the effect of high water table as the moisture travels upwards due to evaporation and transpiration at the surface. The Author divides the moisture distribution in two zones as against three suggested by Mr. Taylor. Experimental evidence shows that field capacity zone, recognized by Taylor as a constant moisture content region, is merely a point on the distribution curve. The zones above and below the point representing the field capacity may be termed as pellicular and capillary zones respectively. The surface application will cause accretion of water table provided the pellicular zone moisture content is raised to field capacity moisture and in case the surface application is insufficient the pellicular zone will be re-established before draining water from the water table, with the redistribution of moisture content depending upon the texture, compaction and nature of the soil crust.

For the pellicular zone to be raised to field capacity at the beginning of

any period, the total capacity of the zone to retain moisture during the period must be equal to the total moisture removed by evaporation and transpiration. To make use of this capacity, the surface application must coincide with the period when the pellicular zone has been established. Since the exhaustion of this zone is slower, its total capacity with less moisture is greater for more frequent and small surface applications as compared with those having shorter frequency and longer duration.

The experiments conducted on soil columns to study the effect of surface application on moisture movement through the soil were not applicable to actual field conductions due to small pipes and lack of provision for measurement of water reaching the water table, It was concluded that for irrigation applied to the soil surface a part of water is retained by the soil profile, another part is lost due to evaporation and the remaining water adds to the subsoil water table.

For the first time a study was undertaken in November 1942 to observe the moisture distribution in soil profile under natural condition by allocating a 40 ft. x 40 ft. un-irrigated plot at Kot Lakhpat Lahore. The observations included measurement of irrigation water, rain fall and moisture contents at various depths ranging from 1 ft to 16 ft, with the estimation of field capacity and pellicular deficiency. It was concluded from the analysis of the results that the pellicular deficiency attains a maximum level during the dry period and reduces in Monsoon period due to rainfall. In areas under irrigated crops, pellicular deficiency is maximum before a watering. Irrigation water causes accretion of water table in the soil crust having low pellicular deficiency and even a high delta of irrigation water may not contribute to the water table in case of high order of pellicular deficiency. It requires further study of moisture movement on un-irrigated and irrigated lands under principal crops to fix the delta of a crop and to avoid deposition of sodium salts.

PART-II

The study of moisture movement through construction of a Lysimeter started at the end of 18th century in regard to addition to subsoil water table and effect of prolonged leaching to the soil nutrients for plants. Lysimeter studies progressed in Cornell Agricultural Experimental Station, New York State Agricultural Station and New Jersey Experimental Station. The experiments lacked in true representation of natural soil profile in respect of density, moisture content and the free water table. The Land Reclamation Laboratory, Lahore took a further step to continue the studies by constructing the Tank Lysimeters and the Iron Lysimeters.

Tank Lysimeters consisted of a set of four tanks 20'x20'x20', the walls being constructed with specially designed interlocking blocks, filled with cement concrete and finished plaster surface. At the junction point of the four tanks, arrangements were made for water table observation. The beds and sides of the tanks were coated with a thick coat of cotton to make them water tight. A three feet layer of coarse sand under 9" deep water was kept in each tank. The observations of moisture content and bulk density were made on the excavated soil compacted by sheep foot rollers in 6" layers, in the tanks. A tubewell was installed to irrigate the tank lysimeters.

A set of twenty Iron Lysimeters were constructed, each with 34 inches diameter and 15 ft. height having holes at 120 degrees at every foot from top. A double wall room with brick platforms in 5 rows was constructed to house the lysimeters. In order to protect the soil from rainfall runoff, galvanized iron collars were provided. The soil profiles were transported from Lahore and Sheikhupura on LTC, Lyallpur on LCC, Montgomery on the LBDC and Sargodha on LJC and compacted to natural field conditions of moisture and density. The bottom of lysimeter was filled with 3" bajri and water table gauge was fixed in the sand column. The water level was maintained at 13.5 ft from the exposed top surface of the soil column.

The experiments were conducted on Tank and Iron Lysimeters to study the effect of surface application on the water table under various crops.

After initial changes of moisture content the water table was maintained at 11 ft. depth from natural surface. The tank lysimeters were designated from D to G to keep track of observations and subsequent analysis.

In tank D, reclamation through crop rotation was applied by leaching through rice crop followed by gram, berseem and sugar cane. The analysis of results show an addition of 29.4% of surface application to the water table during first reclamation rice, 48% during second reclamation rice and 28.2% during berseem. The sugar cane added 12.4% of the surface application to the water table.

In tank E. ordinary rice, wheat, maize and cotton were grown in rotation and it was observed that ordinary rice makes an addition of 24.5% of surface application to the water table; maize and wheat cause depletion in the water able. In tank F, cotton, sugar-cane and maize were grown. The results of tank F were affected by heavy rainfall. Out of 22'02 inches in surface application 19.02 inches were of rain fall and 8.83 inches were added to the water-table.

The tank G was kept as control tank and a total evaporation of 55.8" was observed for a period of 3 years, covering three Kharif and three Rabi seasons. Considering the control tank as representative of fallow condition 55.8 inches of moisture was lost through evaporation and transpiration. The results indicate that for a given soil temperature of 27°C and presence of both sodium sulphate and sodium chloride in equal proportion, the average evaporation of 18.6" is capable of depositing 650 tons of salts annually. The salts are likely to remain at the surface if not washed down by heavy irrigation or rainfall.

The cultivation of rice year after year increases the yearly accretion rate. There is depletion of water table during growth of cotton, wheat, gram and maize. The extent of accretion to the water-table depends upon the previous type of crop grown. Heavy rain during short period greatly contributes to the accretion of water table. Sugar-cane can be recommended as a crop for partially reclaimed fields.

The experiments on iron lysimeter, designated as 1,2,3 were carried out in the same pattern as for tank lysimeter. The results support the

observations made in tank lysimeter in regard to accretion or depletion of water table with a slight difference in actual values. This fact points to the role that the size of lysimeter plays in moisture movement. The mechanical composition of the soil has direct bearing on the moisture movement and a given amount of surface application would not affect the water table for different soils to the same degree. The irrigation practice, therefore, should depend upon the knowledge of soils of a particular area.

Paper No. 294
Year 1951

OBSERVATION RECORD AND ANALYSIS OF PRESSURE PIPE DATA OF WEIRS ON PERMEABLE FOUNDATIONS

DR. MUSHTAQ AHMAD

Paper No. 294
Year 1951

OBSERVATION RECORD AND ANALYSIS OF PRESSURE PIPE DATA OF WEIRS ON PERMEABLE FOUNDATIONS

By

DR. MUSHTAQ AHMAD

Pressure pipes are installed on a number of weirs and irrigation works like level crossings and siphon etc. to monitor uplift pressures due to seepage flow through permeable foundation. The method of observations and their record keeping has an appreciable effect on analysis and interpretation of data. The basic data of a hydraulic structure like construction drawings, proposed upstream and downstream water levels, subsoil data and presence or otherwise of springs and cavities provide a useful basis for the analysis of the problem. The observations during operation of the structure include the recording of water levels in pressure pipes, temperature of water and subsoil, flow water levels along the structure, scour and spring levels if any. Upstream and downstream water levels are measured by gauges whereas the water levels in pressure pipes are recorded by means of bell sounder or by Mecabe water level indicator. The temperatures are recorded by the use of maximum and minimum thermometers.

The record of a set of observations is likely to contain errors and accuracy of observations lies in the degree of skill of an observer and the technique to avoid sources of error. The errors usually result from installation of pressure pipes in a way different from their specified design, human element in observations, and mal-functioning of the pipes due to choking of strainer, leakage of pipe or inability of the pipe

to respond accurately to change in water levels. Observations during unsteady flow conditions also introduce errors in measurement. The abnormal functioning of the pipes can be tested by observing the rate of fall and rise in water levels. The observations should he made fortnightly or atleast monthly, and under conditions when the water levels have remained steady for at least 24 to 36 hours. The records of data should be kept on standard forms.

The purpose of pressure pipe observations and analysis is to take care of safety of the hydraulic structure, to avoid damage to the floor of structure by excessive uplift pressures and to safeguard against undermining and piping. The hydraulic gradient along the seepage path at any point depends upon the percentage ratio of head above the downstream water level to the head across along the structure the stability of weir floor at any point depends upon the balance of forces due to uplift pressure, the weight of masonry and that of water over the floor, if any. The condition of safety against uplift pressure is that the weight of floor under dry as well as submerged conditions in a worst case, must exceed the upward force due to seepage head. At the tail end of the structure the soil particles can easily be displaced if residual seepage uplift force is more than their submerged weight. The hydraulic gradient at exit is called critical or flotation gradient, if the vertical component, of the uplift is sufficient to lift the soil particles. In an ordinary structure critical gradient is generally not possible to occur on sand foundation. However, some factors like a scour hole extending towards the cutoff toe, presence of local surge, non-homogeneity of substrata, sudden change of head and high spring levels can lead to critical exit gradient, and Mr. Khosla proposed factors of safety for various types of sub soils ranging from 1/3 to 1/7 as against 1:1 under theoretical critical conditions.

The presence of a cavity or loose contact of floor with subgrade can initiate undermining or piping. One of the purposes of analysis of pressure pipe data is to detect the presence of such cavities or loose contact points. The effect of cavity as shown by hydraulic model studies is to steepen the exit hydraulic gradient immediately above and below the cavity, whereas it becomes almost horizontal along the cavity. Presence of a cavity close to the end of floor provides on easy path to emerging subsoil flow lines. This in turn steepens the exit gradient

beyond permissible safe limits and may result in progressive start of undermining leading to sand boiling and complete failure of the hydraulic structure.

Safety of the weir structure on permeable foundation can be monitored by comparison of hydraulic gradient from pressure pipe data with the theoretical values obtained by employing Khosla, Bose and Taylor methods. The complex weir section with a number of sheet piles is divided into elementary forms to apply the above method for determining uplift pressure at key points. These values are then corrected for the mutual interference of piles, the thickness of the floor and the slope of the floor. This method is based on the assumptions of two dimensional seepage flows in a homogeneous deep stratum, absence of any silt blanket upstream or downstream of the floor, and that of any temperature gradient in the direction or against the direction of flow. Any variation of these factors can cause a departure of the observed data from theoretical values. A complete understanding of actual seepage flow conditions and the ways to eliminate discrepancies in observed data is needed for the purpose of comparison. Three dimensional seepage flows is predominant in case of cross flow through weir bays due to different water levels across weir bays or due to change in design along weir length. The effect of this condition on observed data can be determined from hydraulic model studies in case of serious doubts about the safety of the structure.

The available evidence regarding horizontal stratification indicates that in case permeability increases along the depth, the difference in hydraulic gradient from the structure on homogeneous subgrade is not as great as in case of vertical stratification. The structures lying on sand underlaid by an impermeable clay layer of finite depth in which the sheet pile pierces the clay layer will result in downstream uplift pressures independent of the upstream water levels. For a pileless floor resting on sand of finite depth underlain by an impermeable clay stratum of infinite depth, the uplift pressures have been found to be less under downstream half than in case of the infinitely deep sand provided the floor length is atleast four times the depth of sand strata; otherwise it is not appreciable. Sufficient experimental data is not available for the effect of mixture of sand and clay on uplift pressure. A model of Kalabagh weir on a foundation of sand and shingle mixed in different

ratios shows that uplift pressures are generally higher than those of homogeneous sand foundation.

Model tests to study the effect of silt blanket indicate that the uplift pressure immediately below the silt blanket decreases with an increase in thickness of silt blanket. The effect of a silt blanket is similar to the presence of cavity in that it flattens the hydraulic gradient between measured points. In such a case complication crops up in the analysis of pressure pipe data and it becomes essential to eliminate the effect of silt blanket. Elimination of effect of silt blanket can be achieved by the application of graphical method or by formula method. The graphical method does not clearly indicate regions of flatter gradient as compared to normal gradient. The formula method follows from the graphical method by plotting a graph between uplift pressure and proportional relative position of pipes. It should however be noted that accuracy of this method depends upon the reading of the first pipe.

The study of the effect of temperature on uplift pressure shows that in the absence of silt blanket the temperature difference between the upstream and subsoil water of about 15^{o} C can cause a change in uplift pressure by 5%. The effect of temperature gradient is minor as compared to the silt blanket effect and may be masked by observational errors.

Statistical analysis should be performed before comparative study of accumulated observed pressure pipe data with the theoretical values to rectify the observational error. All those values should be rejected which are negative, or more than twice the general run of values, or less than half of the general run of values. The values of uplift pressure which differ from mean value by more than three times the corresponding standard deviation should also be rejected. The average percentage of mean to theoretical value is applied to all the actual values and the results are compared with the theoretical values. The plot should indicate that region of the structure which is unsafe against the uplift as well as the period of the year during which the structure is likely to be unsafe.

Paper No. 300
Year 1951

ENGINEERING PLANNING FOR INDUSTRIAL DEVELOPMENT IN PAKISTAN

By

I. A. ZAFAR

Paper No. 300
Year 1951

ENGINEERING PLANNING FOR INDUSTRIAL DEVELOPMENT IN PAKISTAN

By

I. A. ZAFAR

The diminishing trends in export of raw material will be an index about their utilization within our country and a yardstick to measure our industrial development. Though we have already made an encouraging start in this respect, there is still need to eradicate all possible handicaps which impede the healthy growth of industry. Proper engineering planning in setting up factories is a major contribution to overcome not only many bottle necks but also for efficient running of the manufacturing process. The problems facing industrialization are typical of our conditions and quite different from the developed countries. Therefore we can best learn to improve from our own experience. Generally speaking time factor is of great importance in setting up an industrial project because of the quick turnover and increased dividends desirable on capital.

Various stages coming within the purview of engineering planning can be roughly classified as follows;

(i) Selection of site, and engineering and resource survey

(ii) Economic and financial appraisal

(iii) Drafting the schedule of requirement for the machinery

(iv) Planning of site development

(v) Construction of transport facilities prior to receipt of machinery

(vi) Synchronization of the building execution with receipt of machinery

(vii) Scope for future development and improvements

(viii) Testing performance and output of the machinery as connected with construction of various buildings.

When selecting a site following points should be kept in mind. Cost of land, utilization of any existing facilities and proximity of any villages for labour, water supply, existing sewerage system and waste disposal, availability of electric power. Other similar considerations will result in reduction of the overall cost of construction.

As local stuff has to enter the market in keen competition with imported products, the financial appraisal is essential to determine the production costs for the present and the future. It will assist in planning to keep this as low as possible and to have a better hold in the market than imported items. Cost benefit relations and amortization value of the project must be established.

Before purchasing the machinery, it is essential that detailed requirements should be known. Delivery period and facility for supply of spares, maintenance and servicing may be considered for selection. Layout of the factory units should be carefully planned. Factory units include gate house, weigh bridge, parking, offices, welfare building, sanitary facilities, canteen, first aid centre, power house, material stocking buildings and silos, loading and unloading platforms and test laboratories etc. If volume of raw material and of finished products justifies a rail sliding it should be provided. Treatment of industrial waste should be given due consideration. The predominant requirement for industrial waste control is to check pollution of streams or complications caused to the town sewage disposal system than the profits expected by the recovery operation.

In the new industrial projects the structure should be studied simultaneously with the mechanical layout and operating procedures. Opinion of the insurance companies may be sought regarding design of such building units as doors, windows, partition walls, stocking godowns for inflammable products, fuel oil handling structures etc. Type of materials which are available and are most suitable for the construction should be determined. The design of the factory should be such that day light as well as electric light ventilation and unobstructed working space are provided for. Human efficiency, capacity and mental alertness is best at a particular temperature and relative humidity. Therefore opinion of machine manufactures can be sought as to which operations require air conditioning. Specialized service of consulting engineers may be obtained for engineering planning and erection of industrial concerns, with the following advantages:

(a) Eliminated, of delays in manufacture and deficiencies of fabrication and supply machinery.

(b) Saving of time in writing tenders and checking

(c) Economy of construction,

(d) Selection of reputed contractors with proper tools and plant to undertake the job

(e) Availability of materials of best quality and specifications for construction.

It requires considerable experience and years of handling of industries for framing any recommendations for guidance of private or government enterprise. To render help to guide those who venture to set up industrial concerns it is necessary for some organization to collect all sorts of statistics and data on various lines so that whatever guidance is sought is readily available without ambiguities.

This paper does not claim authority on the subject nor is it backed by any large scale field experience. But the author believes that an

engineer can apply his mind to any technical problem in the sphere of our national development and by utilizing his power of observation can produce commendable results. The author also believes that by a scientific approach, by reading plans, and by discussion with the non-technical people or entrepreneurs, the engineers can grasp the problem. The knowledge and experience of the engineers has to be propagated and shared for the benefit of many others beset with similar problems. While recording our appreciation of a problem, our ideas may not be perfect and our recommendations may not cut much ice but we certainly would be doing a service to humanity.

Note:-

This paper appeared at pages 129 to 189 of the Proceedings of Engineering Congress Vol. XXXVII, 1952.

Paper No. 307
Year 1952

CONSTRUCTION ASPECTS OF BALLOKI-SULEIMANKI LINK

By

S. ALLAH BAKSH AND MUZAFFAR AHMAD

Paper No. 307
Year 1952

CONSTRUCTION ASPECTS OF BALLOKI-SUMEIMANKI LINK

By

S. ALLAH BAKSH AND MUZAFFAR AHMAD

Sutlej Valley canals have been experiencing serious shortage of river supplies in Kharif sowing and maturing period, ever since their construction. During the periods of acute shortage i.e. April to June and September, surplus water is available in the river Chenab. The Anderson Committee in 1935 therefore recommended transfer of surplus Chenab waters to Sutlej River through feeder canals linking the Chenab with the Ravi and then the Ravi with the Sutlej.

Various alignments for link canal from the Ravi to the Sutlej were examined, and finally Balloki Suleimanki Link alignment was selected keeping in view the following factors:

a) shortest possible route

b) Alignment of the canal through Crown-waste land to avoid acquisition of fertile private cultivated areas.

c) To minimize the reaches in which the water table is high

d) To dig the canal through low-lying country to reduce water-logging.

e) To reduce the number of crossings for the existing channels and roads.

f) To avoid grave-yards and religious buildings.

Keeping in view the above factors some curves had to be introduced on the alignment of the link canal. The canal is in heavy cutting upto RD 106,200. In the head reach upto RD 68,000, subsoil water level was above the designed bed level. The link consists of both unlined and lined parts. Unlined portion upto RD 0-73250 was designed as per Lacey theory. For a discharge of 15000 cusecs, section adopted consisted of bed width of 300' and depth of 13'. Side slopes above subsoil water level were kept 1,1 and below it 3 : 1. Longitudinal slope was kept as 1:10,000 with Lacey's silt factor 1.0. Lined section has bed width of 115.8' and depth of 18', bed slope of 1:8000 and side slopes as 2.1. It can be seen that there is an increase of 5' in depth and reduction of 184.2' in bed width at the junction of unlined and lined reaches. It meant an abrupt drop of 5' and instead of giving a fall, the bed width was reduced and the depth increased in stages in the transition reach from RD 73250 to 78,500. This was achieved in three cascades.

The Link has unprecedented features such as that the depth of cutting at places exceeds 40' and that the bed line in the head reach requires digging below sub-soil water level to a maximum of 8.27'. It was decided that the dry earthwork should be done by donkey labour and the wet and slush earthwork by machines. Donkey labour was to do excavation upto 2' to 3' above water level from where machinery was to take over. The wet and slush earthwork as per original estimate was 18 crore Cft, but the excavation program aimed at reducing it to the very minimum. To achieve this a cunette 50' wide and 2' deep below design bed was dug by machines near the centre line of the canal, after the dry earth was removed from the top by donkey labour. This method proved effective and the sub-soil water level was considerably lowered. About 4.5 crore Cft of earthwork, originally considered to be wet was dug out by donkey labour without paying wetness allowance. Thus out of a total estimated quantity of earthwork of about 87 crore Cft 71 crore cft was done by the donkey labour. Earth-moving equipment worth Rs. 1 crore cft was imported from abroad but the total work done by these machines was only 10% of the work done by the donkeys during the same period. Comparison of cost between machinery and donkey labour shows that work done by the former is cheaper.

Major masonry works to be constructed were 14. These included 3

aqueducts for distributaries, 5 V.R. Bridges, 2 DR Bridges, and 2 AR Bridges and a railway bridge. Generally structure crossings were made at right angles to the centre line of the Link, but AR bridges were sometimes skewed to avoid bends in important highways. However in the case of Railway Bridge an "S" curve was introduced in the Link to make a right-angled crossing for the railway line over the canal.

Selection and timely arrangement of materials is a pre-requisite for every large project. Cement was arranged from Wah works. Sand samples were collected from various places and their fineness module and cost was worked out. Kalabagh sand was used only on a few important works. Wazirabad sand was used for all concrete works whereas Ravi or Beas bed and pit sand was used for all masonry works. Reinforcement steel weighing 677 tons was arranged from local market. To ensure quality control, sample of bars were got tested from Punjab Engineering college Laboratory at Moghalpura. Results revealed that steel was of good quality. Brick requirement was met by operating departmental kilns. For shuttering, centering, etc 7855 cft of timber was used. To meet the requirement of water 7 tubewells were sunk but sweet water could not be found even upto a depth of 250'. Canal and tubewell water was therefore mixed for use on masonry works and lining.

Dewatering of the foundations was carried out according to the following methods as per site requirements.

a) Pumping units worked by electric motors and diesel engines.

b) Hand Pumps or contractors Pumps for small sites

c) Bailing out water with hand

Major dewatering problem was encountered at the Head Regulator which had to be built on the bank of the river by the side of a running main canal (LBDC). An area of 450' x 450' had to be dewatered with a head across of 21' from the river pond level. Pumping sets were of 2 cusecs capacity.

At various sites wells were sunk to provide stable foundations for structures. Various methods employed for well sinking were Jham Grab worked by bullocks by using excavator; open excavating, by steam winches and by water jets as the site conditions warranted. Various

difficulties were met during well sinking. First was maintaining the verticality of the wells. This was ensured by sinking alternate wells by halves, standing wells giving an indication of verticality. Second difficulty was that the wells got stuck up when they were being sunk though clay strata and any further loading seemed ineffective. In such cases resort was made to method of well sinking by open excavation. Another source of trouble was that the false work collapsed and fell into the well and filled it up with water and slush from outside. It was thought better to have thicker falsework, and 1:8 cement sand mortar instead of mud mortar.

Wooden scaffoldings were used in the case of the Railway Bridge. Angle iron cribs served as scaffoldings at all such works where centerings were made of angle iron cribs. Where earth centering was used, scaffolding was also afforded by filling earth side by side with the construction of piers. Pipe scaffolding was used for tall structures. For many works, formwork and shuttering were required. M.S. sheet formwork is the best but non-availability in such large quantities prevented its use. Deodar wood formwork lined with GI sheet was used on head regulator. Formwork consisting of brick work in mud mortar lined with impervious coat of cement mortar and properly white washed was also used. At places combination of wooden formwork and brickwork were used.

Placing of reinforcement in deep and narrow beams was tackled by placing it in steps. Similarly proper care was employed in preparing mix design and then its placing, compaction and curing. Centrifugal concrete mixer of the tilting type was used for mixing concrete and vibrators worked by air compressors were used for its compaction. Constructions, expansion and contraction joints were provided at suitable places.

Note:-

Paper No. 307 appears in the proceedings of Engineering Congress 1954, Volume 38 at pages 166 to 222. It has 18 photographs and 13 plates.

Paper No. 315A, 315B
Year 1956

STUDIES ON SOME HYDRAULIC FEATURES OF THE DESIGN OF TAUNSA BARRAGE

By

DR. MUSHTAQ AHMAD, ABDUL LATIF AND
CH. MUHAMMAD ALI

Paper No. 315A, 315B
Year 1956

STUDIES ON SOME HYDRAULIC FEATURES OF THE DESIGN ON TAUNSA BARRAGE

By

DR. MUSHTAQ AHMAD, ABDUL LATIF AND
CH. MUHAMMAD ALI

This paper is in two parts. In part I, (315A) alignment and location of the headworks, and in Part II, (315B) estimation of maximum and minimum discharges and levels in the Indus at Taunsa barrage are discussed.

PART - I, Paper 315 A

A number of investigations were undertaken by the Irrigation Research Institute to determine the best location of Taunsa barrage on the river Indus. It was planned to study the location of different weirs on the alluvial rivers to assess their working in relation to the site selection of headworks. The usual practice in Indo Pakistan Subcontinent is to construct the barrage outside the main channel in a bye-river temporarily closed, or in an abandoned course of creek which is dry in winter and the river is diverted to pass over the weir after its completion.

The barrage site should be such as not to lose command and is sufficiently near the commanded area. In case such requirements require the weir siting near an existing bridge or gorge, the site below the existing control point is preferred. Other factors to be considered are minimum haulage distance for construction material, easy diversion

after construction and attainment of approach to the barrage after diversion which may be good for sediment exclusion from canals.

The main factors which determine the behaviour of river flow through the barrage site are;

a. general direction of the river or the river axis
b. The river loops in the vicinity of the selected site.

During the floods the river spills over the banks and if the angle between the weir axis and river axis is large the general river approach will be oblique to the upstream noses of the guide banks. Whenever the river axis makes an appreciable angle with weir axis, shoals would appear by the side of right or left guide bank according to the river axis on the right or left of the headworks axis. Location of the barrage on one side of the centre of Khadir will increase spill area on the other side of the weir thereby enhancing the tendency to form shoal along the guide banks on that side. Islam, Sidhnai and Balloki headworks are the examples where oblique approach has caused such problems. The most suitable position of weir when constructed dry is below the outer side of a convex bend upstream of which the river is straight for some distance.

An angle of 10 degree has been recommended for Taunsa weir between headwork axis and the river axis. The location of barrage is on the outside of the convex bend above which the river is straight for some length. The weir is located towards the left of the Khadir axis in the left arm of the river which was closed by bunds leaving enough waterway to pass the floods during construction period. This site was also suitable due to the proximity of road and railway link. The greater spill area on the right which would result from the asymmetric placement is proposed to be corrected by training works.

PART - II, Paper 315 B

The estimation of maximum discharge, maximum and minimum water levels and their limits of fluctuations due to retrogression and accretion cycles are important from design point of view. It is not possible to estimate the magnitude of the maximum possible flood on a large river to any great degree of accuracy. The following methods were used to

get estimate of probably maximum flood at Taunsa:-

1. Empirical Formulae
2. Probability Method
3. Estimation based on record of floods at Kalabagh & Ghazighat.

A number of empirical formulae are available for estimation of maximum floods. In fact simultaneous existence of all the meteorological and hydrological factors responsible for heavy rainfall and runoff that contribute to make a record flood cannot be represented by a simple empirical formula. These formulae give only indicative but not reliable results on which design can be based without complementary studies. The maximum flood at Taunsa from Karpov and Kanwar Sain curves could be 22 lac cusecs.

There are two probability methods; Basic and Yearly Flood method. The results of analysis by probability method depend on the data input which require large number of authentic discharge observations. For Kalabagh site the data was analysed for periods 1923 to 1940, 1950 and 1952. From probability curves it was found that flood equal to or greater than 20, 13 and 11 lacs can occur on 1 in 1000,100 and 50 years respectively at Ghazighat. The observed data is not enough to make the above results reliable.

The maximum flood recorded at Kalabagh was 8,19,000 cusecs on 29.8.29. An approximate relationship between the maximum annual Kalabagh discharge (x) and the corresponding Ghazighat discharge (y) in thousand cusecs has been developed as under

$$y = 1.8495 x = .868$$

The average net decrease in discharge per mile was determined from this and the computed discharge at Taunsa Barrage site came to 7,61,000 cusecs. If the maximum yield from Suleiman range is taken as 1 lac cusecs and allowing a further margin of 1,40,000 cusecs for the bursting of Shayok dam and other contingencies an estimate of maximum discharge of 10 lac cusecs at Taunsa is reasonable. It is emphasized that a discharge above 10 lacs cusecs may occur but it

would be a rare event. A sudden on-rush of an abnormal flood would still find the waterway inadequate and therefore a margin of flexibility is provided with a factor of 1.25, and designing for maximum discharge of 12.5 lac cusecs.

In alluvial channels the same discharge can pass at different levels at various times. There are infrequent cycles of accretion and retrogression. There occurs a charge in regime of river after construction of a barrage causing extra retrogression downstream, which has to be provided for in design. The estimation of maximum and minimum accreted and retrogressed levels for Taunsa barrage site was based on Foy's method with some modifications. The nearest gauge site was at Ghazighat for which the authentic discharge gauge observations were available for a period of 9 years. The author's method gives the difference of level between accreted and retrogressed levels equivalent to about 2.4' at a discharge of 10 Lacs cusecs as compared to about 9 feet at the low stage. These results are in conformity with the general trend of envelope curves of discharge and gauge in different years. The arbitrary assumptions for arriving at the discharge gauge curves are still open to further improvements. At present this method of analysis is a rational way of getting the discharge rating curve for testing the performance of weirs.

Note:-

Paper No. 315 A & B appear in the proceedings of Engineering Congress 1956, Vol. 40 at pages 1 to 18. It has two parts with 8 figures and 5 tables.

Paper No. 329
Year 1957

THE PHENOMENA OF LOSSES AND GAINS IN THE INDUS RIVER SYSTEM

By

S.S. KIRMANI

Paper No. 329
Year 1957

THE PHENOMENA OF LOSSES AND GAINS IN THE INDUS RIVER SYSTEM

By

S.S. KIRMANI

A correct estimation of river supplies, available for irrigation and other uses in the Indus River System, largely depends upon a proper understanding of the phenomena of losses and gains. Presently no relationship is available between river flows and losses/gains in this system. This paper describes the development of a new relationship with sound theoretical background and detailed analysis of the historical data.

Indus River System covers an area of about 348,000 square miles. It comprises the main Indus and its six major tributaries: Kabul on the right bank and Jhelum, Chenab, Ravi, Beas and Sutlej on the left bank. The Indus valley is composed of alluvium and depth of alluvium ranges from 5000 to 10,000 feet. The rivers pass through vast alluvial plains. Longitudinal slopes in Punjab become very flat like 1 foot per mile which further decrease to 0.5 foot per mile in the lower reaches. All the rivers of Indus system have characteristics of changing their courses and this often made it impossible to locate the sites of many old rivers.

Snow is the source of water for head reaches of most of the streams. In the sub-mountainous regions precipitation averages from 30 to 40 inches and decreases to 15 inches in Punjab and 5 inches in South. The plains are therefore classified from semi-arid to arid zones. Local rainfall is not consistent in terms of quantity, incidence and duration and mainly concentrates during the monsoons (June to September). The average summer temperature in the plains is 95"F with maximum upto 120", whereas average winter temperature is 60" with minimum

occasionally reaching freezing point.

Extreme variations in flows are the typical feature of the rivers of the Indus basin and the normal summer discharge may be as high as 20 times of the winter minimum. Water level normally rises in the start of April with the melting of Himalayan snows, reaching a maximum during July as a result of the monsoon rainfall, falling off in September and hitting its lowest during October to March months. Gross area of the Indus Basin in Pakistan is 131 million acres, 75 million acres of which is cultivable. Only 27.5 million acres are used for crops, of which 90% produces one crop per annum. Actual irrigated area of 21 million acres represents 76% of the cropped area.

Every river in the Indus basin has a phenomenon of losses and gains with an ultimate effect on water for irrigation. River loses water during high flow partly by percolation through the porous bed and bank formations and partly by evaporation and transpiration within the river valley. Regeneration occurs during low flow by the returning of water from river bed and bank formations. The losses during the months of April, May and early June determine the available river supplies for sowing of the Kharif crops. Rabi crops in some areas depend almost entirely on the regeneration from mid October to March. These losses and gains occur in great magnitude and their advance forecast is essential for an efficient and equitable distribution of the river supplies.

Basic factors causing losses include absorption, evaporation, and consumptive use of vegetation in the river valley and channel and bank storage. The gains result from percolation of ground water, return flow from channel and bank storage, rainfall and unmeasured inflows. These factors depend upon many subsidiary factors. Wetted perimeter, depth of water, soil conditions, rate of change in discharge river stage, degree of saturation. Shape and size of river, rainfall etc. are some of the important subsidiary factors.. The problem becomes more complex when gain factors operate simultaneously with the loss factors. Prediction of their combined effect in such cases may turn out to be quite misleading. All the dependent variable factors can be expressed as a function of the river flow. A reliable loss equation can be worked out by considering the loss factors alone neglecting the cumulative effect of all gain factors. Similarly true gain may also be worked out by ignoring

the loss factors. Apparent loss and gain expressions, given by the Author, however, depict cumulative net effect of all these factors. These equations reveal that the losses are not simply a direct function of the concurrent river discharge Q and, therefore, method for Proportionality, according to which losses are a direct function of the river discharge, is basically incorrect.

Bank storage is the main source of regeneration. Bank storage is recharged either by the river in high flow or by the ground water in the doab or by both. Extensive studies were carried out to find out a precise role of the above two causes. The studies established the fact that river flow and not the ground water is the main source of recharge and that there is a definite relationship between the quantity of river flows and the magnitude of regeneration. Valley storage is substantial in alluvial rivers like the Indus System because they have wide and shallow valleys. The effect of valley storage on the river flows can be found out by the use of the Stage Storage Method of Flood Routing.

In establishing the phenomena of the losses and gains in the Indus River system certain basic principles were formulated which have led to the development of important hypothesis. More important of these principles are direct proportionality of losses and concurrent flows, gains and antecedent flows, gains and drop in the river stage and inverse proportionality of losses and antecedent flows, gains and concurrent flows and gains and time elapsed since the previous high flows. For a better understanding of the losses and gains phenomena, effect of the causative factors on the losses and gains must be considered over a sufficiently long period. For a systematic study the flow hydrographs were divided into 5 periods: two rising periods from April to July, two falling periods from August to October and a low flow period from October to March. The above are the general periods for the eleven reaches considered on the Western Rivers. This method of division of hydrographs is in accordance with the method adopted for the study of channel losses in the Upper Colorado River System of USA.

The effect of concurrent and antecedent flow was measured by the volume of flows the magnitude of individual peaks within the period and by the magnitude of rise and fall in river stage. The effect of time elapsed since the previous high flows was measured in terms of the

number of days from the centre of gravity of the high flow mass to a fixed reference data of the low flow period. Method of multiple linear correlations was used for the establishment of different relationships because it provides a simple, practical and useful tool in the analysis of hydrological data. The independent X variables considered in the analysis are concurrent flows, the antecedent flows, valley storage and the time elapsed since previous flows whereas Y is the dependent variable and may be loss or gain. The correlation analysis was carried out for all the eleven reaches on the three Western River i.e., Indus, Jhelum and Chenab and for three reaches on the Eastern Rivers Ravi and Sutlej. The analysis has shown that concurrent and antecedent flow is important factors contributing to losses and gains. The amount of release from the valley storage in falling period has also an effect on pins. The analysis also established the fact that gains in the low flow period are also influenced by the time lapsed since the previous high tows. These results are in agreement with the theoretical principles and confirm the validity of the hypothesis.

Statistical measures such as the co-efficient of correlation, standard error of estimate, etc. show the significance of the relationships and for this study these parameters are in the range of acceptable limits. However these co-efficients can be improved further by including other relevant factors like precipitation, temperature, unmeasured flows etc. All the 53 formulae for losses and gains for the eleven reaches on Indus System give the same consistent relationships. Consistency of the relationships provides a more reliable measure of their significance than indicated by mathematical procedures. A comparison of the estimated values of losses and gains with the actual values in all the 31 years of available data was carried out. The estimated values of losses and gains conform closely to the actual historical values in 75 percent of the cases, whereas the remaining 25 percent cases involved extraordinary value of loss or gain resulting from unusual rains or floods. The degree of agreement between estimated and historical values is quite satisfactory. The formulae are quite adequate for the water studies as these studies assume no change in factors like rainfall and unmeasured flow etc. The future forecasting based on the formulae may have some errors because of the absence of effects of above noted factors. These formulae do, however, provide a guide for extrapolations beyond the observed range as against blind guess work.

A comparison of the results with those given by the proportional method and actual values have established a better reliability of the frmulae than that of the method of Proportionality. This study has shown that both concurrent and antecedent flows have a significant effect on the magnitude of losses and gains. Magnitude of releases from the valley storage and the time lapsed since previous high flows also influence the gains. Evaluation of the established relations shows that they are dependable equations for estimating as well as forecasting.

Paper No. 339
Year 1958

DEWATERING OF FOUNDATIONS

By

Dr. NAZIR AHMAD
ZIA-UL-HAQ

Paper No. 339
Year 1958

DEWARTERING OF FOUNDATIONS

By

Dr. NAZIR AHMAD
ZIA-UL-HAQ

Dewatering is an important problem frequently encountered in engineering construction. The advantages of dewatering by tubewells or by well point method have been discussed and compared with open sump well pumping in this paper. Methods to estimate infiltration into an excavation pit and to plan dewatering by tubewells have been given. Seven different sites dewatered by tubewells as per laboratory suggestions have been discussed.

In Pakistan dewatering is usually done by an open or sump system of dewatering. The method involves a square caisson with perforated side walls and with a plugged bottom a few feet below the level to be excavated. Water seeping into the caisson through the side holes is pumped out. Dewatering can also be done by method of well point system not commonly used in Pakistan. For a proper planning of dewatering it is essential to have knowledge of the weight of submerged soil, the velocity of flow causing boiling of soil, movement of fine particles through sand pores and permeability of the soil formation. Quantum of seepage from a formation can be estimated only if its permeability is known. Permeability can be determined in the laboratory or by Theim or Theis methods. If the site to be dewatered is beyond the influence of a line source, then full area is assumed as enclosed by s single well of that diameter and a number of formulae have been given to compute the discharge likely to be pumped. Darcy's formula can be used where tubewell formula is not applicable. If the site is long compared to its width then a number of tubewells are installed and seepage is calculated giving due consideration to mutual

interference. Usually a low level water table exists in addition to a high level source which necessitates due consideration to the feeding source and its level in estimation of infiltration. Seepage can also be estimated by plotting flow lines from source to sink when more than one free water source exist at different levels. Infiltration is worked out by using Darcy's relation.

Deep turbine or submersible pumps are especially suitable for dewatering because they can operate continuously during the construction period at a properly selected location. High initial cost and the limited availability are their disadvantages. Centrifugal pumps have low depression head of 15 to 20 ft. which requires them to be lowered as operation continues for deep dewatering. Another important component of tubewells is strainer. Its positioning and length is decided according to the deepest level to be dewatered and the discharge desired. As dewatering operations are essentially temporary arrangements, relatively inexpensive strainers should be used. Continuous pumping without any break is essential to create and sustain maximum depression head.

Some important dewatering sites involving different methods may be considered. At Chichoki Hydel site water table was lowered from R.L. 688 to RL 648. Permeability was estimated to be 0.002 ft/sec. The infiltration calculated through formulae came out to be 20 Cs from a planned dewatering area of 200 x 300 ft. Twelve tubewells were proposed with strainers starting at Rs. 650 and ending on a clay layer at RL 600. Submersible pumps were used and desired level was achieved in about two weeks of pumping. Water table was maintained at designed level by using only 10 pumps and discharge actually pumped was nearly equal to that given by the formula. Interference of wells was in the range of 13.5% 25%.

For the Ravi Syphon, area to be dewatered was 1800 x 350 ft. Water table was to be taken down to R.L. 670 from R.L. 700 and R.L. 710. For lack of permeability test, its value was assumed as 0.002 ft/sec. A mean seepage of 17 Cs was worked out by the formula. A four feet thick clay layer was encountered at R.L. 684, which proved to be very helpful in reducing infiltration from the river. The laboratory suggestion was to install 40 wells. Centrifugal pumps were suggested and used. The clay layer made the dewatering possible with only 26 wells,

pumping 16 Cs.

The Shadiwal Hydel site is downstream of fall at R.D. 426 of Upper Jhelum Canal. The area to be dewatered was 250 x 150 ft. water table was to be lowered from R.L. 741.7 to R.L. 701. At R.L. 650 there was an impervious clay layer. Permeability of formation was found out to be 0.000B to 0.0016 ft/sec. The infiltration was calculated as 16 Cs. Twelve wells were suggested, but at the site dewatering was carried out by four settings of well points.

At Gujranwala Hydel site, concrete raft was to be laid in the bed of the Upper Chenab Canal. The F.S.L. in canal was at R.L. 755. Permeability was found out to be .0013 ft/sec. by infiltration tests watertable was to be lowered from R.L. 745 to R.L. 710. Ten tubewells were suggested for an expected infiltration of 21 Cs.

At Punjnad Headworks laboratory permeability coefficient of sand specimen was found out to be 0.0005 ft/sec. Infiltration was estimated to be 8.2 Cs from Dercy's relation. Water level was to be lowered from R.L. 336 to R.L. 316. Nine tubewells each with two strainers were suggested. The executing agency did not stick to the dewatering plan suggested by the laboratory and dewatering could, therefore, be done to R.L. 319 only, with actual pumping discharge less than that calculated.
The site of Right Embankment of Guddu Barrage is near the river Indus. Area to be dewatered was 500 x 100 ft. The water table was to be lowered by about 30 ft. to R.L. 218. Permeability was found to be 0.001 to 0.0001 ft/sec. Expected seepage of 14.4 Cs was calculated for which 12 tubewells were suggested. Dewatering was successfully completed as planned.

For out-fall of Chichoki Hydel project, the area to be dewatered was 250 x 200 ft. The calculated seepage was 6.7 Cs. A system of four tubewells each having four strainers was suggested. Nine tubewells were used to give a total discharge of 8.1 Cs, which was nearly equal to the calculated one. Small variation from the computed discharge was attributed to the greater length of strainers.

The well point system consists of a number of wells made of 1.50 inch

diameter G.I pipe about 21 ft long with a 3.5 ft long filter at the end. Each well is pushed in sand by water pressure exerted through the same pipe. Spacing of wells is about 2.5 ft. All wells are connected to a common pipe. This pipe is then connected to a reciprocating pump run by diesel or electricity. A distinct advantage of tubewell system is that it can be indigenously built whereas all equipment of well point system has to be imported. Skilled manpower experienced in installation of well point system is not available in the country. A cost comparison of dewatering at Chichoki (with tubewells) and Shadiwal (with well point system) shows that cost incurred on well point system was Rs. 10,52,000 whereas expenditure on tubewells was only Rs. 189,024. Fuel cost was nearly the same on both. The well point system is reusable at any other place without repair, whereas only pumps can be reused in case of tubewells. Another disadvantage of the well point system installation is that it interferes with other activities at the construction site. Both methods are quite easy to work Sump system of open pumping is defective because it causes reduction in the weight of formation, loss of compactness and the bearing capacity of the soil. Tubewells and well points are free from these defects. In Punjab, the formation comprises fine and medium grade with one or two clay layers appearing within 100 ft. The tubewells can be lowered upto the clay layers. Level inside the well must be kept 10 to 20 ft. below the level to be dewatered. The strainer should also be a few feet below the level to be dewatered.

Paper No. 351
Year 1961

DESIGN OF ALLUVIAL CHANNELS AS INFLUENCED BY SEDIMENT CHARGE

By

MUSHTAQ AHMAD & CH. A. REHMAN

Paper No. 351
Year 1961

DESIGN OF ALLUVIAL CHANNELS AS INFLUENCED BY SEDIMENT CHARGE

By

MUSHTAQ AHMAD & CH. A. REHMAN

Seven new link canals with discharges varying from 6500 to 21700 cusecs, designed on Lacey's approach, and with slopes of 1/8000 to 1/10,500 are being constructed as a part of Indus Basin Project as a consequence of the Indus Water Treaty. The heavy withdrawals will create turbulance and disturb the desired silt exclusion. Mailsi canal, a scouring channel, silted up by 2' above the design bed due to change in river regime. Similarly in the first 20 miles of M.R. Link the average silt deposit is 4' above the designed bed. Among the canals taking off from sukkur designed originally at a slope of 1/120000, right bank canal silted up increasing the slope to 1/7300 and left bank canal retrograded flattening its slope to 1/19800. A steeper slope is required by a channel to transport an excessive charge entering into it and necessitates a higher pond level. River approach and slope in vicinity of the Head Works should be controlled to prevent river regime from changing, otherwise headworks will have to be remodeled periodically.

Previously the author had analysed LCC channels and Laurson flume data, and derived functions relating to silt intensity for course and medium sands with hydraulic perimeters. The scope of present paper includes lower as well as higher regime of flow. The data of LCC stable channels with discharge varying from 40 to 10000 cusecs, river Ravi discharges from 10,000 to 100,000 cusecs and experimental flume data from Albertson, et al has been analyzed to obtain two more functions. One of these functions relates silt intensity with discharge, slope and sand dia and the other to shear stress with silt charge. These relations can be used to determine the silt carrying capacities of existing channels

and also for designing new channels. The calculated silt carrying capacities of designed links indicate that slopes of these channels cannot be physically steepened to make them carry the silt charge entering into them. The silt charge in excess of their capacities has, therefore, to be ejected. The ejector channels can also be designed with the help of the derived functions. This paper emphasizes the need of giving proper consideration to the suspended sediment for the design of new channels.

In 1896 Kennedy produced the velocity depth relation from the analysis of data of UBDC system.

$$V = 0.84\, D^{0.64}$$

Lindley analysed the data of LCC system and produced in 1919 the following relations;

$$V + 0.95\, D^{0.57}, \quad V = 0.57\, B^{0.355} \quad \text{and} \quad B = 3.80\, D^{1.61}$$

In 1927 Lacey produced equations in which depth was replaced by R, his silt factor f takes the form of Froude number and is equal to 0.75 V2/R.

The value of 'f' used in the design of a channel depends on the size of bed material. There is no explicit relation given by Lacey for silt transporting capacity of channel. However channels with steeper gradients are presumably capable of carrying greater silt charge. Inglis incorporated the effect of silt charge in his formulae but he did not determine the constants used in his equations. Bose made statistical analysis of data obtained from LCC stable channels in 1933 and presented a set of relations. Blench advanced the concept of bed and side factors to modify Lacey's equations without including silt charge.

The data of stable channels checked with slope relations of Lacey and Bose in Irrigation Research Institute showed divergence of ± 10-20%. Lacey explained this divergence with the help of his shock theory. The equations have been examined afresh and compared with silt charge observations made during 1940, 1942 and 1943 and data from other

large channels. The variation of the coefficients in the following equations has been studied.

$V = 16\, R^{2/3} . S^{1/3}$ $\qquad$ $V/R^{2/3} . S^{1/3} = K1$

$S \times 10^{3} = 2.09\, d^{3} / Q^{0.86}$ $\qquad$ $S^{0.21} . Q^{0.86} / m = K2$

Plottings show that K1 varies from -17 to + 14% and Lacey's coefficient of 16.02 is applicable for a channel carrying silt load of 0.33 gr/litre. Bose coefficient K2 shows a divergence of -16.4 to +36.5% indicating that K2=2.09 may be applicable in case of channels transporting 0.3 gr/litre. These plottings also reveal that the divergence from mean velocity is due to silt charge.

The data of LCC and other channels and Laurson flume data was tested for silt carrying capacity as a function of $(V^3/R$ and (V^2/S) respectively. It was found that following equations were valid for silt charge upto 2gr/litre;

(i) $\quad C = 0.1 + 0.1\, (V^3/R - 1.5)^{4/3}$

(ii) $\quad C = (8g\, V^2 . S)^{2\ 1.25}$

Field data from LCC and other channels and published data of renowned hydraulic engineers namely Simons, et al has been plotted against 7 different variable functions. Plots for function V^3/D, $8gV^2/S$ and VS show wide divergence. The relation in terms of tractive force and silt diameter ranges from 0.14 to 3.0 for regimes of C = 1ppm to 30.000 ppm suggesting that form roughness and silt intensity are similar problems.

The plots of functions $q^{2/3}.s$ and $q^{2/3}.S/_W$ indicate individual trends for each range of different diameters. The plot of C against the function $q^{2/3}. S/W^{1/2}$ gives a representative curve that covers a wide range of silt charge from 'ppm to 40000 ppm. The equation of the curve is;

$$q^{2/3}.S/Wq^{1/2} = 0.5 + 5C^{2/3}$$

Where C is silt intensity in grams/litre, q is discharge intensity in cusecs, S is slope per thousand and W is fall velocity of mean size of bed material. The above formula is similar in its form to Meyer Peters bed load formula. If the left side quantity of the above equation is greater than 5, it can be reduced to:

$$C = 0.1\ q.S^{2/3} / W^{3/4}$$

Conclusion derived from this equation are;

(i) For two channels with same bed silt size their carrying capacities are:

$$C1/C2 = al/q2\ (S1/S2)^{3/2}$$

(ii) For channels with same value of $S/W^{1/2}$, silt carrying capacities are in ratio of their discharge intensities

$$C1/C2 = q1/q2$$

(iii) Also when C1/C2 and W1/W2 are both equal to 1,

$$S1/S2 = (q1/q2)^{3/2}$$

Lacey's slope relation $S= 0.39f^{5/3}/q^{1/3}$ gives the above conditions for channels having same value off

(iv) Two models with different distortion vertical and horizontal scales, may give similar results if scale ratio of silt intensities is same in both of them.

Note:

Paper No. 351 appears in the Proceedings of Engineering Congress 1962, at pages 1 to 18. There are 4 tables and 13 figures/graphs. For various formulae and their derivation the reader may refer to the original paper.

Paper No. 352
Year 1961

ARTIFICIAL CUT-OFF AT ISLAM HEAD WORKS

By

KHALID MAHMOOD & ABDUL BASIT AKHTAR

Paper No. 352
Year 1961

ARTIFICIAL CUT-OFF AT ISLAM HEAD WORKS

By

KHALID MAHMOOD & ABDUL BASIT AKHTAR

The paper deals with the development of a Horse-shoe bend in the Sutlej River at Islam Headworks which was straightened by an artificial cut-off in apprehension of unfavourable consequences of an unpredictable natural cutoff. The knowledge of meandering behaviour of alluvial channels is essential for effectively dealing with problems like the one under discussion. In this regard, series of experiments conducted at the Vicksburg Station U.S. Army Corps of Engineer had revealed that;

(a) Bank erosion was responsible for meandering when other factors of changing flow conditions were absent.

(b) An irregularity in a straight alluvial channel imparts curvature to the streamlines, setting up a circulatory current which would eventually develop a meander. The question, whether varying erodibility and other irregularities cause meandering or a straight channel develops circulatory current which creates irregularity, is debatable.

(c) The meandering once initiated progressed indefinitely with the bends consistently migrating downwards. A stage was reached for bends with large lengths where resistance to the flow became greater in the bend than that along the bar opposite to it, and the channel, therefore, started cutting through the bar to form a chute. The channel maintains a constant length by chute

development for a constant discharge.

(d) The meandering pattern depended upon the hydrograph of flow. Low, high and medium flows mainly attacked respectively the upstream end, the downstream end, and the middle part of the concave bend.

(e) If material at the downstream end was less erodible than that at the upstream end, a horse-shoe bend would form (with flow directed up-valley), a phenomenon often witnessed in natural rivers.

Artificial cut-offs have been used for river training. The first regular scheme for cut-offs for the river channel improvement for flood control was executed on Mississippi River in U.S.A. The river length of 680 miles was reduced by about 25% with the help of 2 natural and 14 artificial cut-offs, a gauge reduction of 12 to 13 ft for comparable flood discharge had also resulted. Other instances include cut-offs executed to eliminate horse-shoe bends upstream of the river structures which threatened the retired embankments. Cutt-offs have also been used to rectify the aggrading river channel downstream of the headworks.

It has been noticed that any artificial or natural cut-off results in availability of excessive energy which brings about violent changes in the river by eroding river banks upstream and aggrading the river channel downstream. Straight cut-offs create a river path in conflict with the meandering behaviour of the river. The meandering tendency ultimately endangers the agricultural lands etc. at unpredictable places, and these problems were still experienced when some artificial cut-offs in Europe were excavated to full river size. The consequences were as detrimental as those resulting from natural cut-offs.

Study of river survey plans for the years 1929 to 1961 indicate a tortuous course of the river Sutlej Upstream of Islam Headworks. Five cut-offs took place from 1929 to 1959, out of which only one was artificial (1959), the other 4 were natural. Increased tortuosity is an effect of construction of the Islam diversion weir on an alluvial river. The two possible reasons are;

(i) obstruction offered by the weir to the downward journey of the meanders and

(ii) deposition of coarser material on upstream which increases tortuosity,

It is believed that tortuosity (river length divided by straight line distance from a given point to the weir) varies directly with the sediment load. The existing problems on the upstream of the headworks are a sand shoal at the nose of the Right Guide Bund, Southern creeks downstream of the G.H. Spur, and the horse-shoe bend. The development of the horse-shoe bend may be attributed to presence of erosion resisting patch on downstream of the bend which obstructed further migration of its lower arm. The thalweg-neck distance ratio of the loop had increased to 4.2 which was an obvious indication of a likely development of a natural cut-off in flood season with serious possible consequences due to the close proximity of the Headworks. It was, therefore, decided to create a natural cut-off during the low flow season of winter 1958.

Capacity of the cunnete should be limited to 8 to 10% of the river discharge' The cut should be made tangential to the river course at entrance and exit, and the entrance should be bell mouthed with double the width in first 300 to 600 ft. in order to avoid shock losses' A mild curvature should be provided in the alignment to increase the tractive force for enhancing transport sediment capacity. The best cunnete section is the one, which provides a good scouring velocity, and is made as deep as possible with the available means. The cunnete should be such that $Rc/Lc^2 > Rr/Lr^2$ (R&L stand for hydraulic mean depth and length while the subscripts c & r represent cunnete and river respectively). Elimination of a full 'S' curve should not be aimed at by a single cut, and the upper bend must be eliminated first. In case multiple cut offs are required, it is a good practice to start from the lower end.

The length of the cunnete was 5700 ft with a constant bend width of 30ft. A side slope of 1.5:1 was provided in 3500 ft length of the head reach and 1:1 elsewhere. The bed levels at the head and tail were fixed as 447 and 446 respectively. For a river discharge of 60,000 Cs the discharge in cunnete was estimated to be 2500 Cs with velocities of 4.5 & 6.0 ft/sec. at head and tail respectively. The mean value of the tractive force at head and tail worked out to be 0.18 & 0.34 lb/ft^2. which

was sufficient to erode fairly compact and compact sandy clay (sand content less than 50%) at head and tail respectively.

The cut was opened at 9.30 AM on 8.7.59. Discharge upstream of the headworks was 55040 (minimum required for generating scouring velocity in the cunnete) at the time with an expected rise in the river upto 8000 cs over next couple of days. At the time of breaking the check bund, the head across the bund was 3.1 ft. The cut passed 2500 Cs with an estimated velocity of 7 ft/sec. within an hour of its inception. A rapid development of the cut followed. The percentage of the river discharge passing through the cut increased from 25.4% on 22.7.59 to 72% on 26.7.59, whereafter, a moderate increase was observed till 100% flow through the cut on 12.8.59. The main river course was closed during sharply falling flood. The rapid development of the cut can be attributed to proximity of the headworks, erodibility of the strata, and a rather higher loop-neck ratio of 4.4 against a recommended value of around 2.0 for the artificial cut-off. The pond level initially kept at 453.95 to accelerate the development of cut was raised by 0.9 ft to control the silt entry into the canals. The rapid development of the cut resulted in deterioration of the river downstream. However, two creeks intersecting at RDs 2200 & 3500 absorbed part of the silt load. It may be, however, emphasized that for full utility of the cut off, it also requires essential bank protection.

Longitudinal section of the Mailsi canal, off-taking from right flank of Islam Headworks, observed in the following months indicated a gradual rise of the bed level as a direct consequence of the increased sediment load due to the artificial cut, and the channel during the period upto October 1959 showed a marked tendency of meandering. Meandering in turn affected the structures starting with damage to the first bridge at RD 20180, where oblique flow and excessive scour was observed. Similar conditions were experienced on other structures and stone dumping was resorted to. Excessive sediment entry in Mailsi Canal accompanied with scour at the structures has continued till the time of writing of the paper in 1962, in addition to the other observations made already.

Paper No. 365
Year 1963

THE ENGINEERING PROFESSION IN PAKISTAN

By

S.S. KIRMANI

Paper No. 365
Year 1963

THE ENGINEERING PROFESSION IN PAKISTAN

By

S.S. KIRMANI

The engineers working for the Government Departments have made outstanding contributions to the profession by establishing extensive networks of surface irrigation, road communications, power stations, transmission lines, and innumerable other public facilities. The vast infrastructure created after construction has brought out the need for systematic operation and maintenance of complex engineering works. Day to day functions of the engineers were unavoidably influenced by rather inflexible government rules and procedures which restrict discretion and individual judgment in pursuit of a uniform policy for treatment of all problems. The motivation for doing something new has been seriously impaired and a practice of playing safe has emerged in Pakistan ever since Independence.

The engineering profession in Pakistan after having been confined to the government departments for a long time is now acquiring new dimensions. A strong drive for economic advance, establishments of major public corporations and gigantic Indus Basin Programme are the three main factors responsible for the shifting trend. The opportunities in an expanding profession have brought out new challenges at a time when lack of unity in the profession has undermined its competence.

The expanding job opportunities for engineers also resulted in some problems like controversies and conflicts. The public corporations had to borrow engineers mostly from government departments in numbers, of course, too inadequate to implement large engineering projects. Involvement of a number of foreign and local consulting firms was a

natural consequence. Repeated discussions comparing competence and practices of departmental engineers with those of local and foreign consulting firms and the comparison of individuals of differnent organizations hampered the optimal use of engineering man-power available in the country. It was not fully realized that such controversies would impair the image of engineering profession in the eyes of public as well as the government.

Development of engineering competence is not possible without paying due attention to engineering man-power. Men represent the most important M of the five essential M′s in the engineering practice. The remaining being for Methods, Materials, Machines and Money. Competence of an individual engineer depends on two qualities: his "inherent ability" and his "attitude" towards profession. Development of professional attitude is the key factor for sound engineering ability. Motivation, Enthusiasm, discipline, and Participation are the prerequisites of a professional attitude. Motivation increases with financial returns, recognition, and personal satisfaction. A motivated engineer can do wonders with unrelenting enthusiasm as a driving power. Self-discipline helps one to manage his time and energy for optimal output. It also enables him to develop clear thinking and an in-depth look at the point of view of others, and to rectify his own weaknesses through systematic work. Participation is an effective way of contributing your best and of sharing experiences with other professionals. Persons with apparently average qualities sometimes perform far better when afforded opportunities to participate and accept responsibility.

Development of engineering competence has been affected by class system in service, service rules, lack of opportunities, and inadequate communications. The prevalent class system in service is similar to Hindu Cast System as it links the future of young engineer with his status at the time of his birth in the organization. Service Rules guarantee promotion on the basis of seniority. This tends to kill the incentive for creative work which inherently is associated with some risk of failure. Great advances in engineering are made by those who decide to try something new rather than playing safe using shields of established methods and practices. Lack of opportunities and challenges has been instrumental in employing many engineers on jobs of routine

and repetitive work which soon causes their professional knowledge to become stale and obsolete. Communication is essential for continued education which was never so important as it is today in the world where knowledge becomes obsolete at an unprecedented speed. Engineering societies through periodic publications help in continued education.

Lack of professional behaviour in application of technical knowledge and not the insufficient technical knowledge is responsible for many problems of Pakistani engineers. In addition to the quality of his engineering work an engineer is also judged by his professional behaviour by the people he comes across. Engineers must recognize the fact that their 'unscrupulous criticism of other engineers affects the profession at large Engineering profession, unlike many other professions, affords a much greater flexibility in doing a given work' No engineer should be criticized because he did a job in a manner different from another engineer. The impression that no two engineers agree stems from the lack of understanding of the engineering profession.

The Pakistan Engineering Congress is truly responsible to protect ethical standards which govern the profession. The Congress which is expected to take a leading role in addressing the problems faced by the profession is today weak and ineffective. Senior members seem to be interested more in individual security than in the welfare of the profession. The constitution of Engineering Congress should be amended to include the following:

i. Establishing and maintenance of education, ethics and professional practice.

ii. Promote unity among engineers and engineering organizations.

iii. More effective role of the senior members of the profession and the Executive Council in advancement of the profession. The Executive Council should advise members of the profession and engineering organizations in matters of technical disagreements.

iv. Effective liaison with other engineering societies both inside and outside of the country.

v. The profession must wake up to recognize the stakes. Individuals as well as organizations have to accept responsibility to jointly face and resolve the problems faced by the engineering profession.

Paper No. 377
Year 1965

A STUDY OF THE EFFECT OF SUSPENSION PARAMETERS OF RIDE INDEX OF A RAILWAY VEHICLE AND RESULTS OF TRIALS ON THE PAKISTAN WESTERN RAILWAY

By

M. Z. MOZAFFAR

Paper No. 377
Year 1965

A STUDY OF THE EFFECT OF SUSPENSION PARAMETERS OF RIDE INDEX OF A RAILWAY VEHICLE AND RESULTS OF TRIALS ON THE PAKISTAN WESTERN RAILWAY

By

M. Z. MOZAFFAR

The riding characteristic is the most significant of the factors that make the railway journey attractive at a competitive price. The design of coaching stock bogies in the Pakistan Western Railway needs a re-examination to improve their riding characteristics on the track. The Author has described the trials conducted and the results obtained by the joint effort of Pakistan Western Railway and Messrs Linke-Hoffmann Bucch, a German firm of carriage manufacturers.

The sum total of the measures which improve the wellbeing of a passenger and reduce the fatigue during a journey is termed as riding comfort. Riding comfort depends upon car body vibrations, running noises, dust nuisance, temperature, lighting and general aesthetics. The paper focus is on the most significant factor of car body vibrations. Transmissibility ratio i.e., the capacity of the running gear (the bogies) to transfer shocks and impacts in the vertical and horizontal directions into bearable type of vibrations determines the running quality of a vehicle and is a function of the ratio of the frequency of the existing forces to the natural frequency of the suspension gear of the car body. Rail joints, wheel eccentricity, rail surface irregularities, shaking action of the wheel pair, track gauge variations, and lead alignment of track etc. generate forces that give rise to vertical & lateral oscillations. The

natural frequency of the suspension gear of the car body is dependent upon specific deflection of the springs, vehicle mass, system inertia and link arrangement.

Car body movements are due to oscillations along the three principal axes i.e. bouncing along the Z-axis, fore & aft movements along the longitudinal axis (X-axis) and lateral oscillation along lateral axis (Y-axis) and the rotational movement about these principal axes, known as nosing, rolling and pitching respectively. The oscillations are coupled in simple combinations to avoid complexity of vehicle dynamics, If all of these oscillations and combinations are taken into consideration then only digital computer can solve the resulting numerous simultaneous equations at the design stage of the vehicle. Presently, therefore, the natural frequencies of bouncing, nosing, swaying, lateral and rolling oscillations are considered in simplified relations for the design purposes. With the help of natural frequencies, the spring characteristics and swing link proportions are chosen in such a way that resonance does not occur at operational speeds and vehicle is not unduly sensitive to vertical and lateral impacts. The determination of damper characteristics for reducing amplitude of oscillations within acceptable limits at resonance, without adversely affecting the riding quality of the vehicle at speeds above that of the resonance speed, becomes easy with the determination of natural frequencies, Lateral and nosing frequencies, combined oscillations (swaying) and sinusoidal motion of wheels secured to common axle, dampers ratio of amplitude and vertical oscillations are determined by using the current theories on bogie design.

Rolling Stock Test Department of Reisch Bahn at Berlin-Gruncwald and Dr. Eng. Sperling studied various mathematical terms like spring stiffness damping factor etc. in relation to human sensations. The object was to establish a mathematical relationship in terms of amplitude and frequency of lateral and vertical vibrations and an index value specifying the running quality of a coach. Study of human reactions has shown that sensation of discomfort is twice as great in the case of lateral oscillations as for the vertical ones for a particular frequency. The same study established that a frequency range of 4-8 C.P.S. produces the maximum discomfort whereas frequencies above 50 C.P'S. get filtered by the human body, and cause no discomfort. According to the German

Federal Railways Standards, the running quality evaluation mark ("Wertziffer WZ") of 3 to 3.25 is the lower limit of running quality in case of passenger coaches and 4 to 4.25 for goods stock. The evaluation mark of 1 is very good whereas 5 is dangerous for operation.

Theoretical design must be tested in actual conditions because of the numerous variables involved. With the objective of a new suspension arrangement having a ride index value of 2.5 to 3, the German firm M/s. Link Hofmann Busch designed five different types of suspension arrangements for the light weight integral type semi tubular coaches. In order to test all of the five alternatives, the firm also supplied three sets of prototype bogies in which any of the suspension arrangement could be incorporated by suitably changing the springs of the secondary suspension. Harder primary suspension was classified as A, and the soft one was designated as B. The five combinations of suspension arrangements were named as Al, B1, A2, A3 & B4. Combinations 1, 2, 3, and 4 were achieved by suitably changing the springs of the bolsters. A3 and B4 bogies had higher specific deflections which made them unfit for lower class coaches.

The actual tests were carried out between Kissan and Renala Khurd Railway Stations for an overall period of 3 months and 9 days. The object was to determine the effect of change of suspension parameters on the running quality and then to select the best possible combination for good riding characteristics on the P.W.R. track. The first few trials eliminated most of the combinations of spring gears and coach No. WGNT 4307 was used mostly for this purpose. For measuring WZ values, Inductance Type Accelerometers, Amplifiers Bridge Type (single channel) and Analog Computer were the main instruments used. Resistance Strain on Gauge, Amplifier Bridges Type W (six channels), Magnetic Tape Recorder and Three Channel Recorder were used for the strain measurements. 142 trial runs on the test track were conducted. The suspension arrangements with a WZ value higher than 3 at 60 m.p.h. were eliminated. The test results were in close agreement with the theoretically anticipated predictions.

The detailed analysis of the test results revealed that on the existing tracks of Pakistan Western Railway, all coil spring bogies with suspension arrangement characteristics corresponding to proto-type A2

for the lower class and proto-type A3 for the upper class coaches are the best choices. The two suspension arrangements have axle springs of the same characteristics. Choice of the two arrangements simplifies maintenance and achieves standardization. A2 specifications are harder helical coil axle springs with helical coil spring at the bolster with a total static deflection of 0.56 cm per MP (0.217 in/ton) and distribution ratio of 30:70 between the primary and secondary springs. A3 has same primary springs as of A2 with a total deflection of 0.83 cm per MP (0.322 in/ton) distributed in the ratio of 20 : 80 between the primary and secondary springs.

Condition of wheel tyres has a great influence on the riding quality of a coach. Heat treated tyres, recently introduced on the Japanese National Railways can be used in Pakistan as these may be proved to be longer lasting. Composite brake shoes can also be useful with regards to maintenance as their use results in imparting a polished surface to the wheel tread which has a good effect on ride index value. The economic comparison of cast iron brake blocks with composite brake shoes can only be done by actual experiments. The rubber fittings and shock absorbers may pose some maintenance problems because of climate and dust and can reduce the efficiency of shock absorbers. A close monitoring of the performance of the rubber fittings and shock absorbers in early stages of the introduction of A2 and A3 bogies is necessary. Adequate facilities for shock absorbers maintenance are also required for smooth running.

Paper No. 390
Year 1967

STRUCTURAL INVESTIGATIONS OF SUKKUR BARRAGE ARCHES

By

CH. MAZHAR ALI

Paper No. 390
Year 1967

STRUCTURAL INVESTIGATIONS OF SUKKUR BARRAGE ARCHES

By

CH. MAZHAR ALI

Extensive cracks were noticed in concrete arches of Sukkur Barrage in April 1964. A high level committee was appointed to investigate the damages and submit their recommendations regarding remedial measures. The structural investigation of the arches as a part of total investigations is the subject of this paper.

Formerly known as Liyod Barrage, the Sukkur barrage is the first barrage on the river Indus. It was constructed in 1932 and is situated about 225 air-miles North-East of Karachi. Sukkur barrage project completed at a cost of Rs. 20 crore is considered to be the World's largest single unified irrigation scheme. The barrage feeds seven canals which irrigate 69 lac acres of cultivable area. The irrigation channels off-taking from Sukkur Barrage hold a key position in the rural economy and prosperity of southern West Pakistan.

The right undersluices, the central weir portion and the left under sluices are the three main divisions of the barrage. In all there are 66 bays, each having a clear span of 60 ft. The left and the right undersluices consist of 7 and 5 bays respectively, whereas the remaining 54 bays of middle weir portion are divided into 6 compartments of 9 spans each, separated by abutment piers. The lower level road bridge and the higher level gate bridge decks are supported on reinforced concrete arches.

The Gate Bridge on the u/s side and the Road bridge on the d/s side are the two separate bridges supported by the piers. Both of these bridges transmit their load to the piers through reinforced cement concrete arches. The Gate Bridge rests on two separate series of arches on u/s and d/s sides having width of 8'-3" and 5'3" respectively with a springing level of RL. 219.0 and a rise of 15'. The Road Bridge arches, 25'-3" wide, have a springing level of RL. 201.0 and a rise of 10'. River training works, constructed to eliminate serious silt trouble on the right bank canals, reduced the flood capacity of the barrage from 15 to about 9 lac cusecs. Stone masonry voussoir arches, proposed for both the bridges in the original design in 1919, were changed to reinforced cement concrete arches in 1928 just before actual construction, in view of a higher strength of reinforced concrete, its suitability for longer spans, and quick and economical construction. Design of the intrados for stone arch profile was retained for both the bridges to avoid delay in construction.

The three sets of concrete arches were designed in 1919 for certain loads and temperature variation. For the Road Bridge, live load of 100 lbs per sq. ft with an impact factor of 13% in addition to the computed dead load and a steam roller load of 15 tons was taken as the design load. A graphical arch analysis yielded a maximum compressive stress of 316 psi in masonry at crown and 311 psi at joint of rupture. The Gate Bridge design load consisted of the dead load, a live load of 45 psf with 13% impact factor, 3 ton travelling crane with a laden weight of 15 tons and an impact factor of 33%. This lead produced maximum compressive stresses of 315 psi at crown and 313 psi at joint of rupture in the upstream arch and 316 psi and 313 psi respectively in the downstream arch at corresponding locations. During review in 1928, the original loads were considered inadequate and live load over the road bridge was changed to a succession of 16 ton lorries per 10 ft traffic lane. This case of a fully loaded arch with the heavier axle at the crown was considered to be the worst condition of loading. A static load of 40 ton gate standing on 4 bearers and a rolling crane load of 7 tons passing over the arches was accepted as the worst loading for Gate Bridge. A temperature variation of + 30°F was assumed for design. The reinforcement in concrete arches was provided with a concrete over of 22".

Crown moments, thrust and shear for loads and temperature effect were calculated by using summation equations for symmetrically loaded arches. One half of the arch was considered for analysis by putting live loads symmetrically about the crown. No effort was made for investigating an economical design. Concrete for arches had a ratio of 1:1.5:4 for cement, local lime stone, and stone metal without sand. The construction was continued in all seasons, without the help of vibrators, and field as well as laboratory tests.

Cracks were first reported in the barrage piers in 1949 and in cement concrete arches in December 1950. The Cementation Company, a British firm, was engaged to repair these arches by guniting but soon after the repairs the cracks reappeared in a number of arches in March, 1956. The Cementation Company made no serious attempt to treat the cracks and accept any responsibility, and advanced reasons which were not considered to be convincing. In view of the importance of the barrage, the need for extensive field and laboratory investigations was evident. The arches of the Road as well as Gate Bridge were tested for different loadings and with different assumptions which do not form the subject of this paper by the author. The design assumptions originally made were checked and shortfalls in original design procedures were taken due notice. The Author has included the detailed results in his paper for the interested readers.

Sukkur Barrage concrete arches are not of any regular shape and arch section is also not symmetrically reinforced. These arches are also fixed at the ends and 6 different equations are required for stress analysis of the structure. As these arches follow a complex shape and cannot be analyzed by usual integral equations in addition to three equations obtained from moments and reactions. The solution of these six equations yielded many interesting results, the most significant being the increase in secondary stresses due to temperature and shrinkage with the thickening of the arch.

The first step in design check was aimed at comparison of design conditions with original computed values. The Road Bridge was further checked for present day loading conditions which, of course, represent higher loads than those originally assumed. The study yielded some significant findings like lack of thoroughness and precision in design,

ignorance of sizeable shrinkage and rib shortening effect, incorrect splay effect, use of summation equations for computing M and T values without any appreciation of their basis, etc. Live load for maximum effect was placed on the arch, on the analogy of maximum beam moments, by temperature influence, were increased to 70% of the critical moment values whereas live load contributed only 7 to 12%.
An isolated radial crack in an RCC arch is only a warning and is not a sign of failure. A through crack breaks the continuity of the arch and gives relief to the secondary effects of temperature and shrinkage. Consequently moments will become far lower than those when the arch was continuous and un-cracked. Therefore the possibility of a sudden collapse may easily be ruled out. The arches, therefore, are not required to be replaced and may be rendered safe by suitable repairs. However a thinner arch having half the thickness of the existing section and half the quantity of steel of the existing arch would have been superior and stronger.

Structural investigations of Sukkur Barrage have brought out some errors and omissions of basic nature which point to the need for a Centralized Design Office where proper analyses by expert engineers would help in eliminating such serious drawbacks. Unfortunately the Central Design Office of the Irrigation Department at Lahore, after thirty years unmatched performance, was abolished in 1962 during Reorganization of the Department. The present trend of relying on foreign Consultants to the extent of allowing them to direct our policies regarding sensitive technical decisions is fraught with danger. The trend is also likely to hamper the much needed growth of the engineering profession in Pakistan. The engineering talent in the country needs guidance and conducive working environment for which the Government as well as the Pakistan Engineering Congress should identify their roles and take immediate steps to check the worsening situation.

Paper No. 413
Year 1974

PUNJAB HEADWORKS AFTER 1973 FLOODS

By

MUHAMMAD ASLAM CHOHAN

Paper No. 413
Year 1974

PUNJAB HEADWORKS AFTER 1973 FLOODS

By

MUHAMMAD ASLAM CHOHAN

Punjnad Headworks was constructed at a cost of Rs. 1.93 crones during the year 1928-32 below the confluence point of Chenab and Sutlej and was originally designed for discharge of 4.50 lac cfs. A higher discharge of 5,49,106 cfs in 1929 necessitated remodeling of the Headworks. An annex weir of 14 bays of 60 ft span each was added on the right to increase the design capacity to 7 lacs cfs. In August 1973, a highest ever recorded flood of 8,02,516 cfs was experienced due to combined high flood conditions in all the three rivers: the Ravi, Chenab and Sutlej. It caused breaches in the left and right marginal bunds at various locations which was mainly responsible for immense losses to human life and property, irrigation system, roads and railways. To save the important city of Rahimyar Khan, cuts were made in the main railway line, Sadiqia branch and other distributaries to spread the flood water in comparatively less developed areas. The flood water damaged the entire d/s floor of Abbasia canal regulator, the divide wall and bays No. 9 to 12 of Punjnad canal head regulator due to the swirling action of backflow.

The failure of the protection bunds was caused by many factors. Flood levels at the barrage exceeded the design levels dub to accretion in the river bed. The modularity of Annex weir (bays 34-47) was poor due to river approach and it passed 163000 Cs against the designed capacity of 250,000 Cs. Shifting of the Sutlej and Chenab Confluence close to the weir made the left half of the weir relatively more active while it caused masking of the Annex weir on the right. An exceptional rate of rise of flood water along bunds, their inadequate sections to cater for hydraulic

gradient, and inherent weaknesses like clods and insufficient compaction common to bunds constructed by donkey labour contributed to initiation of leakages at a discharge of 5 lacs Cs. The bunds had also remained dry and un-soaked for a decade. Severe wind & rain storm almost for the entire duration of the peak flood stage generated wave action, produced radial cracks along the bund slopes, and hampered the watching and repair operations. After the floods, various remedial measures were considered. To prevent shrinkage cracks, wetting channel along the downstream side of the bund was proposed. A board of chief engineers decided that the existing bunds having numerous weaknesses should be used as one bank of the wetting channel while the main flood embankment should be constructed afresh on the landside by properly compacting the earth at optimum moisture content. The section of the bund should have stable slopes and should fully cover the hydraulic gradient line rather than following the practice of fixed upstream and downstream slopes regardless of the type of soil. A second defence bund behind the left marginal bund was also proposed, with the area between the two embankments suitably divided into compartments by cross blinds.

Additional waterway is required to pass a 9 lacs cfs flood for a probability of 100 years. Unsatisfactory performance of existing annex was against providing yet another annex. The only alternative is the provision of a spillway regulator between RD 5-9 of the Right marginal bund. Further, during floods of 1955 and 1956 a deep channel alongside the junction groyne and close to the barrage had formed due to short length of about 300 ft of this groyne. A slight error in regulation could cause damage to the weir. Irrigation Research Institute proposed correction of the river approach by construction of two "Y" shaped spurs along with the extension of junction groyne. This proposal was not workable due to large river depths and consequent very heavy cost of spurs.

The primary task after the floods was to restore the irrigation supply in the Punjnad and Abbasia canals. The only option was to utilize the undamaged bays 1-8 of Punjnad regulator to pass limited supplies to both canals. The damaged regulator portion was cordoned off by stone bund. In October, with a low pond level, the two regulators were segregated with an earthen bund to permit improved independent

working of these canals. Repairing the head regulators by construction of coffer dam to cordon the working area was found to be impracticable as it would render the regulator nonoperational for a period of 4 to 5 months. The other solution was to construct a diversion channel for perennial discharge only and regulate the supply into the canals by temporary regulators. Diversion channel was considered to offtake from left guide bank, join the Abbasia canal at RD 925 and outran in Punjnad Canal at RD 1350. A small pacca structure having stone pitched approaches was considered desirable to check the excessive flows during winter freshets. The temporary regulator of Panjnad canal was designed as a stone weir, when on test running with 500 cfs, settlement was noticed along the side stone pitching, provision of deep sheet pile line cutoff was made. During subsequent operation, deep scour and side erosion occurred but the structures were protected by Killabushing, and dumping of stones and concrete debris.

The working area for repairs to head regulators had to be enclosed with coffer dams in order to remove debris and to keep the differential head within safe limits. These coffer dams were constructed at a distance of 325' below the Panjnad and Abbasia regulators respectively. Another coffer dam was constructed at a distance of about 500' to 700' upstream of the weir line in the left pocket.

Keeping in view the damaged structures, scoured bed, and the existing layout, the structures were found to be marginally unsafe using Khoslas' theory. However the structure were safe based on Lanes' Creep theory. For the past 40 years of operation of these structures no damage due to undermining or even appearance of springs had come to notice. A logic way to repair the structures would have been to provide deep cut offs at the end of impervious floors for protection from damage by scour, but it was not practicable due to stone launching. The other alternative was to provide cutoff depth based on exit hydraulic gradient and to provide deep intermediate sheet pile line to prevent scour from travelling upstream.

In case of Panjnad canal regulator, 16 ft. long intermediate sheet piles were driven. The floor thickness due to this reason was increased from 3.5' to 5.0' resulting in 1:6 glacis slope in bays 9 to 12. A cantilever type R.C.C. divide wall was constructed between bays 8 and 9 to avoid cross

flows. Foundation for a future divide wall was constructed between bays 11&12 so that one bay of Panjnad regulator can be added to Abbasia canal to meet the future demands of irrigation water. Concrete block floor along with inverted filters with improved grading was repaired. At Abbasia regulator a deep sheet pile cutoff of length 10'-8" was provided, at the end of impervious floor. Also 16 ft long intermediate sheet piles were driven to check the scour. Variable depth (8' & 16') sheet piles were driven around downstream wells of divide wall. A previous apron 34 ft. wide, 3 ft. thick was provided downstream of concrete block zone.

The design of divide wall was modified and a solid gravity type stone masonry divide wall was constructed with an independent soft foundation. The transition from solid divide wall to the normal bank of two canals was achieved by flared out brick masonry wall. The working area was dewatered successfully by 5 No open pumps. The subsoil consisted of fine sand, and shallow tubewells up to 50' depth with a pumping discharge of 25 to 30 cfs were found appropriate for the job.

To repair the floors, cracks were opened and anchor bars were embedded in the old concrete with a recess on both sides of crack line and sealed with epoxy concrete. Cement sand grouting was clone by drilling bore holes to fill the cavities inside the floor and in the substrata. Concreting of new floors was done in panels of manageable sizes and construction joints with water stops were provided to check the seepage flow. Uniform mix of 1:2:4 was adopted for concreting to attain the recommended cube strength of 2500 to 3000 psi at 28 clays, having water cement ratio 5.5 to 6 gallons/cwt and slump ranging from 1.5" to 2.0". Piezometers consisting of Pvc pipe and surrounded by well graded shrouding materials were installed at various locations of the head regulators to monitor the uplift pressures.

The structures were to be completed before the advent of Kharif season i.e. 15th April 1974. Therefore all the activities were carefully planned by detailed project planning. All the works were carried out in accordance with the project schedule and were completed successfully.

Paper No. 456
Year 1981

CONSTRUCTION OF RIVER TRAINING WORKS ON THE LEFT BANK OF RIVER RAVI FROM BABU SABU TO CHUNG, NEAR LAHORE

SYED MANSOOB ALI ZAIDI

Paper No. 456
Year 1981

CONSTRUCTION OF RIVER TRAINING WORKS ON THE LEFT BANK OF RIVER RAVI FROM BABU SABU TO CHUNG, NEAR LAHORE

By

SYED MANSOOB ALI ZAIDI

Sinuous course traversed by the river Ravi in plains from Madhopur to Sidhnai Head Works keeps changing and new loops and short circuiting of the old ones takes place. According to Ingles and others meandering is triggered due to slope being either more or less than regime slope, sediment charge being in excess of its carrying capacity, and the irregularities in the channel section. It is generally accepted that meanders mover downstream the river maintains a limiting length of channel and width of Khadir for a particular alluvium. If certain conditions are stable the river is likely to attain a permanent regime; but since hydraulic conditions in river channels are never stable the permanent regime is never achieved.

A loop threatening an important salient is not to be left to itself lest it should erode the salient much before a natural cut off develops. Such a situation calls for providing an assisted cut off to afford protection to the important salient. Also earthen protrusions into the river belt with a protected leading nose called spurs can arrest erosion. Later Mole Head spur, Tee Head spur, Hockey spur and sloping spur developed. The J Spur developed by IRI in recent years is quite effective for most of the locations. A bund constructed in 1959-61 to protect Sharakpur Town on the right side of Ravi was extended in 1972 to tie it with Shahdara

Distributary. During 1973 floods, due to breach in the extension the river flows entered the rear of the bund. Sharakpur Town was once more under threat. In 1974 and 1975 the river was diverted to its pre 1973 course and two spurs were constructed without model tests, to hold the river away from the bund. But in 1976 the river started eroding its left bank opposite the villages form Jhuggian to Chung. The off shore plains opposite many villages were completely eroded off. Part of Meharpura and Manowal were washed away in the 1973 floods and developments of embayments towards Multan Road and erosion of fertile cropped land and properties continued. This posed a threat to the National High Way, important industries and developing townships. A fresh river survey and model studies were ordered to evolve training works required to hold the river in its prescribed course. A model was setup according to the fresh surveys on 1 : 200 horizontal scale and 1 : 25 vertical scale at Field Research Station Nandipur. River stretch between 1'5 miles upstream and 18 miles downstream of Shandara bridge including existing works was represented on the model. The training measures as indicated by the model study include;

(i) Cunnette No.1 (5500' long, 100' wide and 10' deep) across the neck of the river v. bend facing Kharak and Meharpura

(ii) Bye pass road cum flood embankment connecting the existing bund road with Muridwal and Multan road

(iii) J spur No.1 with 4363' length of shank tied to RD 5500 of the new extension bund road

(iv) L bund (mole spur) tied to new extension bund road neat. RD 10600

(v) J spur No.2 with length of shank 4562', abutted to river bluff near Niazbeg after development of the proposed cunnette and shift of the river towards Multan Road

The measures constituted a package solution for providing adequate protection against floods. If these works were not to be completed in one season, the cunnettes and J spur near Niazbeg may be constructed

in the 1st season and the remaining works in the 2nd season' The works required to protect Multan Road and villages comprise cunnettes No.2 and 3, J Spur No. 3 crossing the river and abutted to the high bluff of the river near Chung and a mole spur tied to the downstream side of this J spur. These measures also formed a package solution to be constructed in one season. The design of apron for J spurs catered for the maximum scour likely to occur on training works constructed along river Ravi. After these recommendations, detailed survey was conducted for various training works and the estimated cost came to Rs. 75.66 million.

The project was taken up by the Lahore Development Authority under a Directorate established with staff from Irrigation Department. Tenders for execution of 1st phase were received and after evaluation the work was allotted to Mechanized Construction of Pakistan (MCP). The 1st phase included construction of 6500' long Tie Bund in extension to Lahore Protection Bund from Babu Sabu to the edge of the first channel of the main river, a J spur anchored to Tie Bund and an assisted cut-off across the neck of first loop of the river.

The construction of Tie Bund, shank of J-spur No.1 and excavation of cunnette NO.1 were started. The intake of the cunnette was bell mouthed and its width reduced from 300' to 100' in 500' length. The average depth of 11' of the cunnette was based upon winter water level. The cunnette was completed in May while the other two works in July, 1979. The cunnette developed rapidly and took about 90% of the river discharge in June. As a result of execution of above works, Mian Mir storm water drains No. 1 and 2 and regulating structure at Banu Sabu were re-located. A bridge and a culvert had to be constructed.

After flood season of 1979 the work on the first component of Phase II i.e. J spur No.2 was started. Cunnetts No.2 and 3 were completed before December, 1979. The work on J spur NO.2 and the remaining portion of Tie Bund were completed in May next year. The Diversion Bund in the Main River Channel and two small bunds were constructed in the old river creek. River discharge of 1200 Cs was diverted into cunnette No.2 by closing the final gap of 70' in the Diversion bund. Within 4 weeks of commissioning, these cunnettes enlarged to about 2.5 times the initial capacity. Due to diversion the deep channel near Chung was abandoned by the river and was later closed to complete the work of J spur No. 3.

The mole head spur emanating from RD 2500 of this J spur was also completed simultaneously. The mose of L Bund at RD 10600 of Tie Bund was converted into a stone protected mole.

These works were completed in time and stood the high flood on 17.7.80. All the cunnettes developed to the desired extent to carry the discharge. During the construction of these works compaction and soil testing was done by a soil Laboratory set up at site.

Apart from protection afforded to villages Kharak, Meharpura, Shadiwal, Hanjarwal, Niazbeg, Mohlanwal and Chung 8000 acres of fertile land was saved from floods. Lahore, Multan Highway, industries alongside, developing townships and future housing colonies between the Road and the Tie Bund also stood protected.

Note:

Paper No 456 appears in the Proceedings of the Engineering Congress, Vol. LVIII, 1982, at pages 324 to 371. There are 4 tables, 10 photographs and 9 figures. The interested reader for further details may refer to the original paper.

Paper No. 459
Year 1983

A STUDY ON DIESELISATION OF SIBI KHOST SECTION WITH GEU – 15 & GMU – 15 GROUP – IV DIESEL LOCOMOTIVES

By

MIAN GHIAS-UD-DIN

Paper No. 459
Year 1983

A STUDY ON DIESELISATION OF SIBI KHOST SECTION WITH GEU – 15 & GMU – 15 GROUP – IV DIESEL LOCOMOTIVES

By

MIAN GHIAS-UD-DIN

Pakistan Railways provides the only transport link fur passengers and goods between Sibi and Khost through a 133 Km long railway track section. Transportation operations have suffered because of deteriorated condition of XA class locomotives group V leading to loss of revenue. A study has been undertaken to see if dependable heavier axle load diesel locomotive with lesser rail betiding moments can replace existing XA class locomotives which produce greater rail bending moments with low axle loads. The appropriate choice would be to consider the GEU-15 and GMU-15 diesel locomotives under Group IV which are already touching Sibi shed. This would require an increase of permissible section speed from 40 to 60 KMH to achieve the optimum operation speed.

The German Railway design practice has been adopted to find the rail bending moments under static axle loads of the locomotives. The track section material is taken as 100 lbs rail with mono block prestressed concrete sleepers of Pakistan Railway design. From the analysis it was found that under static axle load XA class steam locomotive produces a maximum rail bending moment of 2.0064 ton cm as compared to rail bending moments of 1.7577 & 1.7787 ton-cm produced by GEU-15 & GMU-15 diesel locomotives respectively in the worst case. By the

commissioning of diesel locomotives, increased speed will have dynamic effect which can be analysed through the application of impact factor and speed coefficients to the static axle load. The practices of Indian, Belgium and West German Railways along with Pakistan railways has been evaluated and compared to find out the permissible speed factor. From the experience of Indian Railways on test/trial of Rajdhani express a speed factor of 28.6% was adopted for a locomotive speed of 60 KMH. A rather small rise of 7% in the speed factor has been estimated when the speed increased from 40 KM I I to 60 KMH. Therefore a section speed of 60 Km per hour would be permissible provided the track is well maintained to specified track geometry parameters.

Under the prevailing financial crisis, the objective of introducing the diesel locomotives by Pakistan Railways could be achieved by intensification of maintenance efforts rather than the alternative of replacements of existing infrastructural elements involving major investments' In the first stage of first phase, induction of G 15 and GMU-15 diesel engine would be made for transportation along Sibi Khost section. The increased maintenance effort would include increasing maintenance funds and equipment by 20% initially subject to further review, ultrasonic testing of rails to replace defective rails, replacement of excessively worn out rails along sharp curves exceeding wear limits as prescribed in Way and Works Manual, and intensified monitoring of bridges by Bridge Branch' The existing speed restrictions of bridges would continue. Maintenance efforts for embankment and protection works would also be intensified by improving the current revenue budgetary allocations.

In the second stage action plan would be initiated to undertake improvements in track structural works. Track on all major bridges would be welded by providing suitably designed expansion joints accompanied by the provision of fittings and fasteners in accordance with International Standards to cater for expansion. Check railing would be provided on sharp curves. Joint leveling and delogging operations would be introduced throughout the section and their measurements would be made by "Funicular Rule". Rail joints would be maintained by special efforts along the entire section.

In phase II of the programme, maximum sectional speed would be raised from 40 to 60 KMH by the realignment and redesign of sensitive track curves considering super-elevation, deficiency of cant and transition. Special efforts would be made to clear the obstructed view and to prevent boulders falling on the track by removing overhanging boulders and rocks. Execution of pending protection works and clearance of catch water drains would be undertaken.

The technical evaluation and recommendations made in this paper have been accepted and approved by the Railway Board. They have further been implemented with full achievement of targets.

Paper No. 460
Year 1983

IMPROVEMENT OF BEARING CAPACITY FOR FOUNDATIONS OF KOTRI GAS TURBINES EXTENSION PROJECT

By

MOHAMMAD RASHEED CHAUDHRY

Paper No. 460
Year 1983

IMPROVEMENT OF BEARING CAPACITY FOR FOUNDATIONS OF KOTRI GAS TURBINES EXTENSION PROJECT

By

MOHAMMAD RASHEED CHAUDHRY

The 50 MW Kotri Gas Turbine Extension Project of units 5 and 6 was prepared by WAPDA. After its approval by ECNEC, it was completed by 1.5th May 1981. The contractors (M/s Marubenil, the Consultants (M/s EPDC) and the manufacturers of major equipment (M/s Hitachi) were all from Japan. The estimated cost of the project was Rs. 201 millions with a Foreign Exchange Component of Rs. 108 millions.

The contractor was also responsible for investigation and design. He drilled three bore holes, upto 15 meter depth by rotary method, at locations selected to help in design of foundations. The SPT tests were performed at 1 meter intervals upto the depths of 4,9 and 14 meters respectively in bore holes 1,2 and 3. The SPT values varied from 48 to refusal. Chalky lime stone was encountered during drilling necessitating strength tests on rock cores. The crushing strength of lime stone for 15 out of 18 samples averaged at 2.5 kg/sq.cm proving it to be a very weak rock. The investigated subsoil was divided into three layers. The top layer upto 1 meter thickness was brown clayey silt, and the middle layer varying from 3 to 4.5 meter depth from ground level was brown weathered limestone. The bottom layer was brown chalky limestone with the exception of 0.6 meter thick seam of shale at a depth of 7

meters in bore hole No. 1 and 1.5 meter thick layer of chalk at a depth of 10.5 meter in bore hole no. 3. No water samples were collected for testing because the bore holes were dry.

The bearing capacity of 2 and 2:5 kg/sq.cm was recommended for depths 1.5 and 2 meters respectively. Considering the possible interaction of rain water during heavy rains with weathered limestone, a low bearing capacity 0.5 kg/sq. cm (approximately 0.5 T/sft) was estimated while the computed bearing pressure under the turbine load was 1 Kg/sq.cm. The contractor proposed to provide a shallow foundation consisting of 1.35 meter thick R.C.C. with compressive strength of 4000 psi over 150 mm thick lean concrete laid at a depth of 1.1 meter below the ground level. The author as the representative of WAPDA as the employer rejected this proposal because of anticipated low bearing capacity of soil under possible wet conditions.

Alternative proposal of the contractor for providing 12 number 22 inch dia., 3.9 meter deep cast-in-situ concrete piles under each Turbine foundation was also rejected because of additional cost and more execution time required. On Author's suggestion weathered rock 1.35 meter below the R.C.C. foundation was excavated and replaced with uniformly graded Bholari sand mixed with 2% cement by weight with a maximum slump of 1.5 inches. The cost comparison of the three types of foundations showed piles to be the costliest and sand as the cheapest. Additional weight of 1:3:6 lean concrete under contractor's first proposal was another disadvantage of that proposal.

A 2% cement sand mix was selected on the basis of achieved maximum dry density of 1.74 gm/cc in the trial tests carried on 1%, 1'5% and 2'% cement sand mixes filled in 3x3x2 ft. deep pits reflecting the actual site conditions. The selected mix was also good for avoiding liquefaction. Chemical tests indicated a low sulphate content of 0.03 to 0.15% in the subsoil allowing for the use of normal Portland Cement. Low content of Calcium Oxide (0.03 to 0.16%) and of Magnesium Oxide (0.01 to 0.03%) was found to pose no threat to the durability of the concrete.
The cement sand mixed in the standard concrete mixer was placed in 150 mm thick layers compacted with vibrating plate compactor with a minimum target of 85% relative density according to USBR Relative Density Test. The achieved relative density varied from 87 to 100%.

The compaction was started from the outer ends and moved towards the centre providing overlaps and covering the whole area. The moisture content was controlled continuously with the speedy moisture testing equipment. The entire operation was carried out round the clock in three shifts. There was a saving of Rs' 282,000 as compared to the pile foundation alternative.

The plate load test confirmed the increase of foundation bearing capacity from 0.5 T/sft to 5 T/sft. The improved bearing capacity is expected to be long lasting with a life longer than the life of the gas turbines.

Paper No. 472
Year 1984

CONSTRUCTION OF KHAIRWALA DRAINAGE PROJECT

By

SYED MANSOOB ALI ZAIDI

Paper No. 472
Year 1984

CONSTRUCTION OF KHAIRWALA DRAINAGE PROJECT

By

SYED MANSOOB ALI ZAIDI

At the time of construction of the Lower Chenab Canal, subsoil water-table was at a depth of about 60 to 80 feet. With the passage of time and development of irrigation, water table gradually rose leading to water logging and salinity. At present about 54% of the total area of 3,33,400 acres has a subsoil water table depth of less than 5 feet. In order to provide relief to this area, the Irrigation Department conducted surveys and prepared the Khairwala Drainage Project for construction of 124 miles of trunk drains. The project area consists of alluvium, mainly unconsolidated sand and silt with small amounts of clay and kankar. The average annual rainfall is from 10 to 16 inches out of which 80% falls during the monsoon season. Thus once one of the first agricultural lands are now faced with the problem of salinity, water logging, rainfall runoff, and flooding.

Salinity is due to shortage of irrigation water on the one hand and high water table on the other hand. Water logging is the result of seepage from the irrigation system and infiltration of rainfall runoff due to lack of surface drainage. Similarly, flooding is caused by the accumulation of rainfall runoff for lack of storage capacity in the soils due to high water table and due to obstruction to flow and absence of proper surface drainage. The average annual loss due to flood damage in the area is estimated as Rs. 6.31 million and due to reduction in crop yields as Rs. 31.4 million.

It was therefore proposed to construct the Khairwala main drain, and the Dijkot and the Nasrana branch drains. In addition to these surface drains, 96 tubewells of 2.5 cfs capacity each were to be installed by WAPDA in low lying pockets of Dijkot and Nasrana sub basins. Detailed field investigation and surveys were carried out to arrive at the drain alignments and capacities, design of drain sections and construction parameters. Drain design computations were based on Manning's relationship.

The drain excavation process was divided into two phases for the Dijkot and the Khairwala drains. The first phase had the excavation of the upper dry earth while the second covered wet excavation below the subsoil water level with draglines. The project included construction of bridges, aqueducts, inlets etc. Three railway bridges, were designed and constructed by the Pakistan Railways. Six AR bridges, 10 DR bridges and 49 VR bridges were envisaged, many of which have been constructed. Four falls have been constructed. Problem of well sinking and lowering of subsoil water was the main challenge. Lenses of impervious hard clay prevented sinking of wells in most cases and it became a serious problem to dislodge them. The structures were provided with pressure pipes to monitor their safety. Another special feature was the laminated construction of weir crests. The main advantage of this feature is that the slopes of these drains can be conveniently increased later by lowering the crest. The drains cross the irrigation channels at 3 points. These are trough type, the trough conveying the irrigation water over the drain. The design discharge capacity of troughs has been kept as 10 to 25% over the present requirements for future development. A trough type structure had to be constructed to provide safe crossing for the main Sui Gas Transmission, and their structure had to be provided with proper anchorage to safe guard against vibrations. About 200 inlets for rainwater were proposed to be constructed at the depression sites on the three drains. Almost all the inlets are barrel type. These have been provided with double acting flap valves to stop reverse flow from the drain into the depression.

Dewatering was the main problem during construction stage of the project. At first the working area was enclosed with a ring bund and connected with roads through a specially made track. The lowering of

subsoil water level had to be done against a constant static head of water standing around the bund in addition to the sub surface flow from a sandy aquifer with a high yield. This required heavy pumpage over a long duration.

The search for quality bricks was a serious problem, as there is no control over the manufacturing process of bricks. Steel products of low quality and uncertain properties are being produced by the manufacturers. Deformed steel with working tensile stress of 18000 psi has been used on all structures. Coarse aggregate was brought from Margalla Hills or from good quality Sikhanwala stone. Clean pit/river sand was used for general purpose but for reinforced concrete in deck slabs, coarse Haro sand was added in the ratio of 1 : 1 to the pit sand. Normal portland cement was used in all the structures. To achieve quality control, departmental specifications were followed and site offices were set up on every component work where relevant plans and other record along with inspection registers were also maintained. A complete soil laboratory was setup for monitoring of compaction of soil and determination of soil characteristics. In addition to field checks steel and concrete were tested at the central testing laboratory of the Punjab Engineering University at Lahore. Bar charts were used for preparation of work schedule and monitoring of progress. Triangular diagrams were used for monitoring of subworks. Information on financial utilization was contained in monthly progress reports.

Another problems faced during the execution of project was the land acquisition. The existing procedures caused delays in execution due to prolonged litigation in the civil courts.

From the experienced gained on the project following suggestions have been made:

(i) Land acquisition process should be simplified. Either the project Engineer should be delegated with the powers of land acquisition Collector or a proper land acquisition cell headed by an officer having full powers of District Collector may be created and attached to each major project. The land acquisition Act and relevant rules may be modified on the lines

of the housing Act.

(ii) Ways and means including necessary infrastructure for control over manufacture of important construction material is needed. To ensure quality of bricks revival of departmental kilns is necessary. Manufacturers of steel products should specify structural properties of their goods.

(iii) Funding of project should correspond phasing of expenditure as deviation results in delays and increase in cost due to escalation and higher interest charges.

The annual average benefits of the project are estimated as Rs. 37.71 million. The benefits are already apparent in the upper Dijkot Drain and Nasrana sub basin, where pools of stagnant water and marshy areas are dis-appearing rapidly.

Note:-

Paper No 472 appears in the Proceedings of Engineering Congress Vol. LX, 1985 from pages 167 to 178. There are 8 tables 12 Photographs and 10 figures. The interested reader for further details may refer to the original Paper.

Paper No. 480
Year 1985

TRANSPORT OPTIONS FOR DEVELOPING COUNTRIES

By

MIAN GHIAS-UD-DIN

Paper No. 480
Year 1985

TRANSPORT OPTIONS FOR DEVELOPING COUNTRIES

By

MIAN GHIAS-UD-DIN

Man is greatly dependent on the transport industry for socio-economic growth and development. This industry overcomes physical barriers and facilitates socio-economic contacts and promotes better understanding and nation-wide unity. Productive sectors which are major components of GDP and GNP are closely linked with transport. The main objectives of Transport Planning are safe movement of the passengers and goods, and facilitating increase in the economic growth, rate of production and per capita income. It is, therefore, essential to determine the share of national resources that must be devoted to achieve the goals of projected transport needs.

The overall transport development policy should seek to improve various modes of transport, encourage increase in commercial activities, provide access to backward areas, encourage more private investment in transport sector, and upgrade transport facilities at the international terminals. The entire transport sector should be developed as a part of a Transport Master Plan detailing the time cum resource framework on yearly and five yearly bases. Policies for the development of national transport system should cater for domestic and international demands in passenger and goods movement, priority of investment on projects of higher and quicker economic returns, and long distance transport of passengers and goods for specific reasons. The Master Plan should have a viable undertaking of evolving a transport system for a span of 15 to

25 years and a promise of ideal transport service in the country after its complete execution.

In developing countries of Asia and Africa Roads and Railways are main surface transport modes. However, with the exception of Indian Railways and a limited success in some of the West Asian oil producing Gulf States, Iraq and Iran they have failed to fulfill their role as a rapid mass transport mode. Air transport has severe limitations for cargo transport as well as passenger movement due to cost and space considerations. The choice of developing a mass transport mode is thus confined to roads and railways.

The Railways have some inherent technical and economical characteristics such as an independent track, operation at prescribed timings, fast running multiple unit trains, long life tracks, powerful rolling stock and low direct costs. Average costs in case of Railways decrease substantially with increase in traffic and haulage, whereas the road costs remain static.

Road transport essentially involves the use of small capacity vehicles which results in problems like congestion, air pollution and frequent accidents. The railway transport is free from the handicaps inherently built in the road transport system. Obviously the only option available to the developing countries is to rely on the railways for most of the transport needs. A properly designed road transport system can be effectively used to supplement the railway transport. The ultimate solution lies in adopting a multimodal mix predominated by railways for long distance and bulk hauls. Waterways and coastal shipping can fill in the gaps in peculiar situations. Air transport should be developed for international travel and as an elite luxury transport within the country.

Each mode of transport should be developed as a part of the total integrated plan with close coordination at policy making levels. Experience of the developed nations may be shared. An important fact should, however, be kept in view that the foreign experts generally render their advice based on experience of their own countries which may not be applicable fully to the developing countries. There are

frequent examples of loss of timely progress resulting primarily from slow decision making without any accountability. Closer communication and rapport with the scientists and technologists may be sought by the transport professionals. The developing countries have plenty of opportunities for a better economic growth which in turn would indicate a bright future for the transport industry.

Paper No. 484
Year 1985

REMODELLING MARALA BARRAGE AND LINK CANAL FOR SILT CONTROL

By

MOHAMMAD ASLAM CHOHAN

Paper No. 484
Year 1985

REMODELLING MARALA BARRAGE AND LINK CANAL FOR SILT CONTROL

By

MOHAMMAD ASLAM CHOHAN

Marala Headworks was constructed during 1905-12 on the river Chenab, just below its confluence with Jammu Tawi and Munawar Tawi Nullahs. The Upper Chenab Canal, off taking from this shuttered weir, served the districts of Sialkot, Gujranwala and Sheikhupura and outfalled in the river Ravi at the right flank of Balloki Headworks for augmenting the flow of Lower Bari Doab Canal. It was the first linkage of the waters of two rivers. Marala Headworks was remodeled from time to time and had a length of 4318 feet, and was designed for a flood discharge of 7,30,000 Cusecs. After Independence, as a sequel of Indo-Pakistan water dispute, Upper Chenab Canal was remodeled during 1949-53 to increase its capacity from 11694 cusecs to 16500 cusecs to increase transfers to Balloki. Construction of Balloki-Sulemanki link of 15000 cusec capacity was taken up in 1951. Marala Ravi Link Canal, off- taking above Marala Headworks and outfalling into river Ravi at some distance upstream of Ravi Syphon, was constructed during 1953-56 with a design discharge of 22000 cusecs to meet the requirement of Balloki-Sulemanki Link. The regulator of this link was located 300 feet upstream of the U.C.C. regulator. Excessive silt deposition in the Link Canal soon after its commissioning in 1957, necessitated the extension of divide wall of Marala Headworks by 312 feet during 962-63.

Marala Ravi Link, a non-perennial unlined channel had been designed on Lacey's theory for a discharge of 22000 Cs, Lacey's silt factor of

0.98 and silt carrying capacity of 0.7 grams/litre. After two years of its commissioning the channel had heavily silted, its bed width had increased, the head regulator was rendered non-modular and the discharge capacity was reduced from 22000 Cs to 15000 Cs. The sediment concentration entering the canal frequently exceeded its sediment carrying capacity because disallowing the river waters with excessive sediment would involve repeated closures of the canal. Jammu Tawi joining the river just above the left undersluices of the old weir, was also responsible for inducing higher silt charge into the link. Apart from the extension of the divide wall, shutters of the weir in the first two bays were replaced or modified to raise the pond level, spurs with stone pitched noses were constructed in the canal head reach to restore the design bed width, and training works were constructed along left upper marginal bund to shift the confluence of Jammu Tawi upstream to control silt entry into M.R. Link. All these measures proved futile. The other off-take, the Upper Chenab Canal, however, faced no silt problems.

With the conclusion of Indus Water Treaty between the two countries, India got exclusive right for use of waters of the rivers Ravi, Beas and Sutlej. To feed the canals of these rivers, a net work of Links, some Barrages and two Dams were constructed by Pakistan (Termed as Replacement Plan Works) by securing financial help from the World Bank and the friendly countries. The newly established water rights between Pakistan and India warranted either completed remodeling of existing Marala weir or construction of an entirely new structure. After a detailed examination, it was decided to construct a new barrage 1100 feet downstream of the existing weir. The construction of the barrage was started in 1965 and completed in 1968.

The only way of increasing the silt transport capacity of the Link was to steepen its slope. The crest of the regulating bridge cum fall at RD 237,500 was lowered by 4.34 feet, and it was expected that retrogression smoothly travelling upwards would generate a steeper slope of 1 in 8333. The retrogression, however, travelled only for a lengh of 5 miles due to presence of hard clayey strata.

The barrage has its own share of problems. It has been felt that the pocket is somewhat wider than required for the length of existing divide

wall. River supply turning round the upstream end of the divide wall enters the link canal more or less directly without being sufficiently influenced by the still pond effect of the pocket.

Experience has shown that when the undersluices are operated for flushing, the silt deposited in the paved portion gets washed while that on the Kacha bed takes much longer to get washed. During winter opportunity for flushing is sometimes provided by a sporadic freshet, otherwise the discharge remains well below that required for effective flushing operation. Provision of skimming platform or silt vanes in front of the regulator can help to exercise some control on the silt entry. Other additional measures like raising crest of the regulator, incorporation of rising cill gates, possible raising of normal pond limit or a combination of these may be helpful in exercising a better control on the silt entry into the link canal. Knocking down of crest of old weir from RL 800 to RL 795 will have healthy effect on silt entry into the pocket.

Due to continuous entry of fine to coarse sand into the link, it has gradually silted upto RD 237,320. Total volume of silt deposit in this reach was estimated at 252 Mcft in 1960 and has now increased to 421.5 Mcft. The link has acquired a steeper slope of 0.17 per 1000 feet. Siltation process combined with running of the link with low discharges has caused widening of the channel and its meandering, resulting in drastic rise in its full supply level. The value of Lacey's silt factor for the existing discharge and existing slope is 1.30 and nearly the same value was obtained when silt factor was determined from mean silt diameter. Silt ejector could not be introduced in the link because of the absence of any old river creek/course on the left downstream.

The Author has proposed some corrective measures for the barrage. The performance of left undersluices can be improved by extension of existing divide wall by 400 feet to eliminate partial pocket effect. Construction of silt vanes to form a silt excluder in the pocket can enable heavy silt charge at the bottom layer of flow in the pocket to pass below the barrage. Compartmentation of pocket into three zones will permit flushing operation at low discharges at more frequent intervals. Pavement of Kacha portion of the pocket will expedite the process of flushing. The old weir crest should be removed because it

creates harmful shoaling effect on the upstream side. Modifying the crest level of existing head regulator of the link including raising of crest level with provision of cill gates will reduce silt entry into the link. The operational pond level can be raised to R.L 813 without endangering the safety of the barrage.

Steps can also be taken to improve condition of M.R. Link. The proposals given by the Author include regarding the bed from head to RD 90190, desilting of the channel, provision of stone pitched profile walls, strengthening of banks particularly the left bank in filling reaches and raising the decks of bridges. The rough cost estimated to implement the above proposals (price level 1985-86) amounts to 194 million rupees.

Paper No. 492
Year 1986

COMBATING HIGHER SULPHUR IN THE COAL AT LAKHRA POWER PLANT

By

GHULAM MURTAZA ILIAS

Paper No. 492
Year 1986

COMBATING HIGHER SULPHUR IN THE COAL AT LAKHRA POWER PLANT

By

GHULAM MURTAZA ILIAS

Brick kilns are the main consumers of over one million tonnes of the coal produced annually in Pakistan. Among the significant industrial units, 15 MW thermal plant of Wapda at Quetta is the only concern using coal as a fuel coal. The occurrence of coal at Lakhra in Dadu District of Sind province has been known for the last 100 years. The mining of coal started in 1960 after Geological Survey of Pakistan estimated the coal resource over an area of 250 Sq Km to be 240 million tonnes. A feasibility study conducted by Wapda in 1985-86 through American Consultants established the presence of 174 million tonnes of in place and 123 million tonnes of recoverable coal reserves. The Lakhra Coal, classified as liginite, has a sulphur content of 7.65% as the most significant of its impurities.

Tests performed in USA on a sample of Lakhra coal to determine its combustion performance indicated a severe slagging potential, medium to high fouling propensity, substantial corrosion and high erosion/abrasion capability. The boiler design for such a coal requires a conservative approach. The washed sample tested to find the change in its combustion characteristics, revealed a reduction in over all heat content by 25%. The burn-test also indicated that the slagging and fouling properties became rather worse. Using washed coal was therefore uneconomical in the boiler designed on a conservative approach for unwashed coal.

It is highly desirable to remove sulphur as it causes corrosion in the boiler and also contributes to environmental pollution. There are several methods of eliminating sulphur such as coal washing before burning, chemical treatment during combustion, chemical treatment after combustion and fluidised bed combustion technology. In case of Lakhra iginite coal washing is not a justifiable treatment because washing not only reduces the overall heat content by 25% per tonne of delivered coal but also worsens the slagging and fouling properties of the coal. Chemical treatment during combustion is normally applied to coal with low sulphur content to neutralise sulphur dioxide produced during combustion. Lime and coal in pulverised form are mixed in a pre-determined ratio and fed into the furnace, where sulphur oxides are - consumed in chemical reactions. Chemical treatment after combustion or Flue Gas Desulphurization (FGD) is suitable for high sulphur coal. This process also involves the use of lime. Lime and sulphur ratio has to be carefully determined for each type of coal. The process operates by allowing contact of flue gas with alkaline slurry or liquid or dry powder which absorbs sulphur oxides. Western countries are employing this process even for low sulphur coals but this technology cannot be used in Pakistan because of prohibitive costs.

In 1985 Wapda conducted a feasibility study for installing a 300 MW power plant at Lakhra. The possibility of installing 700 MW thermal plant was also considered, but with a bigger plant, the sulpur dioxide emission exceeded the limit of 1000 tons/day at 85% plant factor specified by World Bank which was one of the donor agencies for the project. The only options left were either to have FGD equipment for the bigger plant or to reduce the size of plant to keep the Sulphur dioxide emission below the prescribed limit. It was finally decided to have 3x50 MW unit based on Fluidized Bed Combustion (FBC) Technology. The utility application of this technology has started in recent years and is especially suited to high sulphur coal. The purpose of FBC is to trap most of the sulphur dioxide in the furnace. Three types of systems are atmospheric FBC, Pressurized FBC, and Recirculating FBC. Lakhra Power Plant will be designed on Atmospheric Fluidized Bed Combustion system (AFBC). The problems of slagging and fouling for which Lakhra Coal has high propensity are virtually eliminated. A sulphur trap of 90% can be achieved with a calcium/sulphur ratio of 2.5.

However some years of operating experience are still needed to identify the problems associated with this technology that holds a good promise for the poor quality lignite at Lakhra.

The feasibility studies for installation of a power plant at Lakhra have led to following conclusions:

1. In view of the potential emission of sulphur dioxide, the use of conventional boilers should be avoided as far as possible.

2. Generation from Lakhra lignite should be based on FBC technology.

3. For the projected power plant of 3x30 MW to be commissioned in 1990-91 AFBC boilers of 50 MW size will be suitable. The proposed plant provides an optimum approach within the framework of World Bank's environmental guidelines. Bigger FBC units may be chosen for future projects.

4. Another power plant of 2x50 MW being installed by private sector will be operated by 3x35 MW AFBC boilers. The energy available from the power plant will be sold to Wapda. Other provinces of Pakistan can also adopt this technology to consume domestic coal for generation of energy. The challenge of existing power shortage can successfully be met if both the private and the public sector participate in the national programme of resolving the power crisis.

Paper No. 493
Year 1986

ESTIMATION OF MAXIMUM DISCHARGE FOR THE DESIGN OF HYDRAULIC STRUCTURES

By

DR. MUSHTAQ AHMAD

Paper No. 493
Year 1986

ESTIMATION OF MAXIMUM DISCHARGE FOR THE DESIGN OF HYDRAULIC STRUCTURES

By

DR. MUSHTAQ AHMAD

Correct estimation of maximum flood discharge in a river, alongwith determination of maximum and minimum water levels is essential for safe and economical design of large hydraulic structures. Intensive rainfall in the upper catchments during monsoon period is generally the primary cause of devastating floods. The size of catchment area and the rainfall intensity are the two major factors which influence the magnitude of flood discharge. Many methods are available for estimating the maximum discharge for the design of hydraulic structures. The choice of a particular method depends on the available records of hydraulic, hydrologic and meteorological data and the type of the hydraulic structure. The following discussion briefly covers various methods employed for estimation of the maximum discharge.

The available empirical formulae for estimating the maximum discharge are based on analysis of maximum flood records of rivers having- catchment areas of particular characteristics and hydro-meteorological conditions. Each of these formulae gives better results when applied for catchment with conditions more or less similar to those for which it was derived. The catchment formulae developed by Dickens, Ryes, Fanning, Myers, Inglis, Kipling, Karpov and Kanwar Sain are more reliable relations for predicting maximum discharge. For small hydraulic structures like culverts, syphons, aqueducts etc. One should carefully select a formula essentially meant for relatively small

catchment areas.

Another category of formulae includes as a variable the rainfall in the catchment and thus provides a more rational approach. We can estimate the peak discharge with the help of Chamier's or Craig's formula if catchment parameters viz its area, length, width, velocity of run-off, run-off coefficient and average intensity of rainfall are known. The run-off characteristics of the catchment area and the rainfall intensity enter the computation to provide a more rational approach. For available data either Richard's method or Unit hydrograph method can be applied. In case of a given catchment if the length, slope, coefficient of run-off, maximum rainfall in inches and duration of storm are known, we can use Richard's formula to arrive at the maximum discharge. Richard's approach involves determination of rainfall coefficient R which is a function of intensity of storm I at the point of maximum rainfall and the duration of storm in hours. He calculates the rainfall coefficient from duration of storm T and time of concentration (t) and assumes that a storm of unit intensity is spread equally over the entire catchment area and the duration of the storm is the same as the time of concentration at the point where maximum discharge is to be determined. The coefficient of run-off K is ratio of rainfall to surface run-off. A unit hydrograph can be developed from the known total rainfall and the losses in the basin due to absorption and retention. The net rainfall can be determined by either index 0 method or trial and error method. The index 0 represents the rainfall that does not appear as run-off and represents the losses in the drainage basin. In trial and error method, rainfall excess is found by successive assumption of different rates of retention till the computed excess equals the storm run-off.

With the help of unit hydrograph drawn for a given catchment corresponding to a given storm, one can compute the maximum discharge that the catchment can yield. It is essential to select a design storm of particular frequency and magnitude for applying it to a given catchment unit hydrograph for estimating the maximum design discharge. Unit hydrograph can be developed from the analysis of rainfall run-off record or isolated unit storm or a major storm. In case no run-off records are available, data of other similar basins of different sizes and characteristics is used to construct a synthetic hydrograph which helps in determining the maximum design discharge. The

synthetic unit hydrograph can be constructed using one of the methods namely Synder's method, Linslay, Kohler and Paulhus method and Soil Conservation Services (S.C.S.) method. S.C.S. also recommends an approach based on Probable Maximum Precipitation. This approach requires a design storm arrangement and design rainfall for generating "probable maximum precipitation" for estimation of direct run-off.

There are also some statistical methods available for estimation of maximum discharge from the flood records by frequency analysis. Out of many methods used for estimation of maximum discharge, Hazen's, Fuller's and Gumble's methods are the ones more commonly used. In Hazen's method annual maximum discharges are first arranged in chronological order and in descending order of magnitude. These are expressed as ratios of mean flood and variation (d) is computed for each value. Squares and cubes of variation are calculated to determine the Coefficient of Variation Cv and the Coefficient of Skew Cs by applying standard formulae. The coefficients help in finding the probable floods expected at different frequencies. Fuller's formula is applicable if catchment area M is known. Maximum discharge is the mean of yearly maximum flows in cusecs for data of N years. The probable maximum flood, likely to occur in N years can be determined by substituting values of maximum discharge, N and M in the formulae. In Gumble's method the probability of occurrence of a flood of magnitude X is function of a factor which depends on the inverse of the standard deviation of the X values. For values of P assumed as 0.1, 0.5, 0.8, 0.9, 0.95, 0.98, 0.99, 0.995, 0.998 correspond to the values of return period as 1.1, 2, 5, 10, 22, 50, 100, 200 and 500 respectively. The value of dimensionless parameter Y and the probable discharge of magnitude X can be determined against the assumed values of P. The plot is made on a special graph paper in which ordinate represents the flood flow and the abscissa the dimensionless parameter. Since the flood frequency analysis is based on data alone which may not include all the physical factors contributing to the maximum flood, it is preferable to use the method with a factor of safety for maximum recorded flood. The probable frequency of a flood of given magnitude can be determined by applying laws of probability using either Basic Stage method or Yearly Flood method.

In order to determine the maximum design discharge capacity for a

hydraulic structure such as a bridge, one can apply Manning's equation. The expected mean operating slope corresponding to a selected flow stage near the maximum Flood Mark and the exact geometry of cross section which has to pass the design discharge must be known. By assuming a modest value of Manning's roughness coefficient for proposed cross section or different values of the roughness coefficient for different compartments, the maximum discharge for the design of the proposed structure can be worked out.

Paper No. 511
Year 1987

SUB-SURFACE PIPE DRAINAGE CONSTRUCTION METHODOLOGY

By

JAVED SALEEM QAMAR

Paper No. 511
Year 1987

SUB-SURFACE PIPE DRAINAGE CONSTRUCTION METHODOLOGY

By

JAVED SALEEM QAMAR[1]

For sustainable agriculture land drainage is a necessity. This is true for humid, sub-humid, arid and semi-arid regions where agriculture is practiced. The drainage required may be both surface and sub-surface. An important objective of a drainage system in irrigated agriculture is to keep water table below root zone and maintain salinity in the root zone within acceptable limits for maximum production. Agriculture land in the Indus basin requires surface and sub-surface drainage. Surface drainage is achieved through open drains which outfall into the rivers. The problem of sub-surface drainage is being countered on a large scale since early sixties. Many areas have been provided with sub-surface drainage and there is still lot of agriculture land which requires drainage. Indus basin is an alluvial plain having good aquifer. In majority of the areas sub-surface drainage has been achieved through installation of tube wells. In useable ground water area the tube well effluent is used for supplemental irrigation. In the saline ground water area the tube well effluent is disposed in the drainage system or mixed with irrigation water. In certain areas either the aquifer is not available or ground water quality is too hazardous for disposal in the drainage system. In such areas sub-surface pipe like drainage is technically more feasible.

Faisalabad SCARP is located in Faisalabad, Jaranwala and Samundri Tehsils of Faisalabad District. The Project covered about 355,000 acres of which 295,000 acres were canal commanded. About 77 percent of the Project area had water-table within five feet from land surface. Due to high water-table about 33 percent of the area had developed salinity

[1] Chief Engineer (Water) WAPDA, Faisalabad

and sodicity. Ground water quality was marginal to hazardous. Although the area is underlain by reasonably good aquifer yet poor ground water quality makes it undesirable to pump saline effluent and pollute flows down-stream.

The problem of water logging and salinity was proposed to be controlled by installing sub-surface horizontal pipe drains apart from improving surface drainage and adopting on-farm water management practices. Sub-surface pipe drains were installed in disastrous area extending over 130,000 acres. The sub-surface drains are of perforated corrugated PVC pipe of 4 to 15 inches dia. The drains are generally placed at 6 to 11 feet below land surface with the help of trenching machines. Four inches thick gravel envelope is placed around the pipes to prevent fine soil particles entering the pipe and improve flow conditions in the close vicinity of the pipes.

The project design was based on two layered model and takes into account the anisotropic condition of the soil. Anisotropy is accounted for through transformation. However the drainage equations are looked upon as over simplified models of a very complex reality. The design of individual drain in a sump system is site specific. The system of design adopted is extended lateral which is typical for an arid, semi-arid irrigated area having relatively high hydraulic conductivity.

The major construction equipment for sub-surface drainage used in the Project area was trenching machines.

Components of a drainage unit

(i) ***Pipe drains:*** The laying of pipe drains is either singular, composite or extended lateral system. The lay-out of sub-surface drain in the Project area is based on extended lateral system.

(ii) ***Man-holes:*** Man-holes are located at junction of laterals or change of pipe size.

(iii) ***Sump:*** Sump is a masonry well which is sunk through conventional system. Sump is plugged at the bottom with RCC slab placed under dry condition which is

achieved through de-watering. Pump house is built on the top of the sump. Automatic electric pumps are installed in the sump.

(iv) ***Discharge box:*** Discharge box is a masonry structure constructed close to sump well. The discharge pipes of the pumps empty the effluent in the discharge box.

(v) ***Disposal channel, open & buried:*** Discharge channels are of rectangular masonry or concrete pipe drains which carry the sump effluent from the discharge box to open earthen drain under gravity.

(vi) ***Drain inlet structure:*** To facilitate entry of effluent brought through disposal channel under gravity into the earthen surface drains, inlet structure is constructed.

(vii) ***Baffle wall:*** Baffle wall is a circular masonry cylinder 4½ inch thick masonry plastered on both sides constructed in-side the sump well providing an annular clearance of 12 inches between the sump and baffle wall.

Laser plane control system

Laying of pipe drains to desired line and grade is controlled through laser plan control system. For trench depths of over 3 meters double laser system is needed.

After laying of drain pipes, the receiving trench is backfilled.

The purpose of consolidation of back fill in pipe trench to ensure that the longitudinal cracks, fissures resulting from the trenching process do not provide functional path to irrigation water and possible migration of soil particles into pipe through gravel.
The density of the consolidated back fill material is envisaged to equal to the dry density of the original un-disturbed soil adjacent to the tubing.

Polyvinyl chloride (PVC) or polyethylene was used to manufacture drain tubing. The manufacturing process includes blender, extruder, cortugator, perforator and coiler. PVC resin is mixed with other

additives in a blender. The blended compound is fed into the barrel of the extruder. The blended compound is plasticized inside the barrel and extruded through the die by means of two screw conveyors.

Gravel envelope

Silty soil without a protective filter around a pipe drain is likely to lead to failure due to choking of slots. The ultimate selection of the envelope material depends on the stability, texture and the permeability of the base material in the vicinity of the drain. Two main functions of the envelope are (i) to act as a filter to prevent soil particles from entering the drain and (ii) to improve permeability around the pipe drain and facilitate the flow of water towards and into it. The criteria generally followed for the design of gravel filter are the one advocated by U.K. Road Research Laboratory (RRL), U.S.A. Soil Conservation Service (SCS) and U.S. Bureau of Reclamation (USER). The best design is usually evolved after due monitoring of the field result.

Paper No. 522
Year 1989

ALLUVIAL CHANNELS REDESIGN PROCEDURE

By

M. NAIMETULLAH CHEEMA, M. HASNAIN KHAN
AND
TAHIR HAMEED

Paper No. 522
Year 1989

ALLUVIAL CHANNELS REDESIGN PROCEDURE

By

M. NAIMETULLAH CHEEMA[2], M. HASNAIN KHAN[3]
TAHIR HAMEED[4]

The Authors in this paper have compared Sediment Transport concept and Tractive Force method with Regime Theory as alternative design approaches for Punjab canals. Commonly accepted relations of Regime Theory have been tested on 678 data sets observed by Alluvial Channels Observation Programme (ACOP). Variations in Lacey's silt factor have been studied as useful indicators of behaviour of an existing channel. Alluvial Channels Redesign Procedure (ACRP) has been developed for channels upto 1000 Cs discharges. ACRP uses established relations of Regime Theory without the silt factor, and can be used for existing as well as new channels.

Lacey's set of equations, accepted by Central Board of Irrigation (India) in 1934, despite some severe criticism, continues to be the basis of design of alluvial channels in Pakistan. Main reason of criticism on Regime Theory, the origin of Lacey's relations, is its pure empirical basis. Sediment Transport concept and Tractive Force method are being considered as possible design alternatives.

This paper compares Sediment Transport concept and the Tractive Force method as alternative design approaches for irrigation channels in Punjab. The Authors have used 678 data sets (for discharge upto 1000 Cs) observed by Alluvial Channel Observations Programme (ACOP)

[2] Deputy Director Designs, Irrigation & Power Department, Lahore
[3] Assistant Design Engineer, Irrigation & Power Department, Lahore
[4] Assistant Design Engineer, Irrigation & Power Department, Lahore

for the analysis. Some of the relations given by Lacey, Bose and Simon-Albertson have been tested on the observed data with the prime purpose of developing criteria for identifying hydraulic problems in the existing channels.

Sediment Transport Approach

In recent years a lot of work has been done to understand the mechanism of sediment transport in alluvial channels. Numbers of formulae have been developed all over the world to predict sediment carrying capacity of the channels. Validity of such equations for different field conditions can, therefore, be rightly questioned.

The Authors have selected five of better known sediment transport relations to give the reader a feel of difference in their predictions. The relations are:

1. Shields (1936)----------(REL. 1)
2. Einstein-Brown (1950)----------(REL. 2)
3. Engelund-Hansen (1967)----------(REL. 3)
4. Ackers-White (1973)----------(REL. 4)
5. Karim-Kennedy(1981)----------(REL. 5)

The formula developed by Dr. Mushtaq, in 1962, has not been included in comparison because it shows an appreciable decreasing trend in prediction (in terms of PPM) towards the tail reach. Behaviour of this formula, therefore, does not provide a common base of comparison with the above noted formulae. It may be mentioned that a fully fledged research effort is required before one can declare a formula preferable over the rest.

Predictions for Punjab Canals

Comparison for a discharge, say 1000 Cs. shows that:

i) Bed load predictors (Shields & Einstein-Brown) differ by 51% and 132% for the two sediment sizes.
ii) Total load predictors differ by 11% to −40% with respect to

Engelund-Hansen.

iii) Formulae predict 3 to 5.5 times less due to increase in d50 from 0.1 to .3mm.

It is clear from above data that the sediment load predictors differ significantly and their predictions are highly sensitive to sediment size.

Practical Design Aspects

A design approach directly based on incoming sediment must resolve number of issues related with peculiar functioning of Punjab Canals. Following discussion briefly covers practical design aspects of sediment Transport approach in relation to Punjab conditions:

1. Carrying capacity of a channel has to be compared with the incoming sediment to verify the adequacy of design. There is a wide range of variation of sediment inflow in a typical Punjab Canal over the year. Figure 4 shows ten daily discharge and sediment observation for B.S. Link for the year 1985 (Ref. 11). The highest figure of the sediment charge (3920 PPM in August), is over six times the lowest (630 PPM in January). Variation of this magnitude in the incoming sediment is common in Punjab Canals. First practical problem is selection of a representative sediment charge for design. It may be mentioned that the design slopes, as worked out by sediment transport relations, are quite sensitive to the sediment charge. The formula given by Engelund-Hansen, for instance, requires slopes of 1:8612 and 1:5665 for carrying sediment concentrations of 100 PPM and 200 PPM respectively for a discharge of 500 Cs.

Tractive Force Method

Tractive Force Method has resulted essentially from the study of forces that cause initiation of motion of the particles composing the channel perimeter. Theoretically the movement of particles will take place when the disturbing force (caused by the water past these particles) exceeds the resisting forces of cohesion and gravity. The design procedure involves equating the unit tractive force with the permissible tractive force estimated from curves / relations based on type of soil, its voids

ratio, particle size and the content of sediment in water. Depth of flow thus computed and the assumed side slopes are used in the Manning's formula to determine the bed width.

The Tractive Force method essentially aims at prevention of scour, where as in case of Punjab canals the common problem is sedimentation. Topography and command constraints seldom allow the flow velocity to exceed a limit where scour would become a predominant design consideration. The very nature of the method makes it an inadequate choice as a design alternative in Punjab.

Regime Concept

The basic difference of the Regime theory from the above noted approaches is that it considers the alluvial periphery of the channel, the fluid and sediment flowing in it as a single whole. The other two approaches take into account the individual effect of the contributing factors in accordance with the Laws of Fluid and Soil Mechanics.

Regime of a channel reflects a range of favourable conditions and not just one and only one combination of discharge, slope, and geometry of the channel cross-section.

Observations on channels which have run long enough to attain regime indicate significant variations to confirm the above fact. Variations in slopes and velocities in different parts of year along with their effects on the cross-section of the flow are matter of common observation. The design approach (ACRP) proposed by the Authors in section 4.8 determines the slope for the regime condition. Steepening of the slope to a certain extent does not, however, disturb the regime.

The fact that larger discharges attain flatter regime slopes necessitates correlation of the lower end of the "regime range" of the Froude Number with the discharge. Based on the lower value of the Fr from the discharge group of the data where fvr>fsr, the Authors have developed the following relation from regression;

$$LFr = 0.195Q^{0.025} \quad \text{------- (22)}$$

The above formula gives the minimum recommended value of Fr for a typical canal. For instance for discharges of 20 and 1000 Cs., Fr worked out by above formula is 0.181 and 0.164 respectively.

Alluvial Channels Redesign Procedure (ACRP)

Flow chart in Figure 1 explains in detail the Alluvial Channel Redesign Procedure (ACRP) developed by the Authors for channels up to 1000 cs. discharge. ACRP can be used for problem identification and redesign of existing as well as design of new channels. The controlling relations used in ACRP are;

$P = 2.8\,Q^{1/2}$ ------- Bose

$V = 16R^{2/3}\,S^{1/3}$ ------- Lacey

$Lfr = .195\,Q^{-0.025}$ ------- Authors;

For an existing channel reach, the relevant procedure of the flow chart may be summarized as follows;

1. If
(i) fvr > fsr and
(ii) LFr < Fr < 0.3, the channel reach has no hydraulic problem.
2. If above conditions do not exist the channel reach must be redesigned.

The following design procedure is applicable for both new as well as existing channels.

STEP-1 Assume a design slope from considerations of command and Topography.

STEP-2 Compute

$P = 2.8\,Q^{1/2}$

$A = 0.286\,Q^{0.5}/S^{0.2}$

$B = P - 2.385D$

$D = (P - (P^2 - 6.94A)^{0.5})/3.47$

$$Fr = V/(gA/Tw)^{0.5}$$

$$LFr = .195\ Q^{-0.025}$$

STEP-3 If Fr < LFr then
(i) Increase the slope, if possible, and go to Step 2.
(ii) Line the reach if slope cannot be increased.

STEP-4 If Fr > LFr but less than 0.3, the design is complete.

STEP-5 If Fr > 0.3 and the existing channel show an objectionable scouring trend, then select a flatter slope (by introducing a fall in the reach) and go to step 2. An existing channel, as an isolated case, may show scouring trend with Froude Number greater than Lfr but less than 0.3 which would require flattening of the slope. For new channels Fr may be kept well below 0.3.

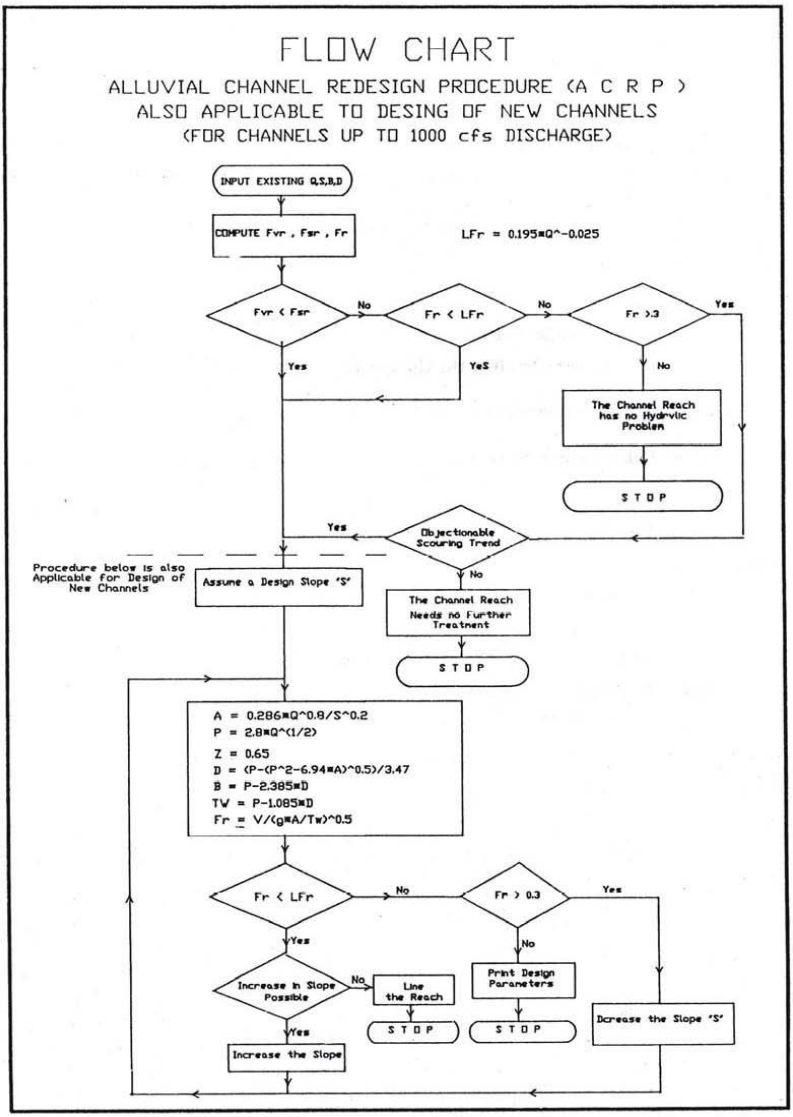
Higher limit of the Froude number is not as important for Punjab canals as the lower limit (LFr) is. The available slopes do not generate high velocities.

Conclusions:

The discussion presented in the paper leads to following conclusions.

1. Sediment transport relations predict widely varying transport capacities of canals and show a poor correlation with sediment data observed in Punjab.
2. Huge variations in the sediment inflow over the year and lack of research work to account for operating conditions of Punjab Canals make the Sediment Trans port concept an inadequate choice as design approach at its present stage of research.
3. Tractive Force method does not effectively deal with the more common problem of sedimentation in Punjab canals. It does not, as such, qualify for a reliable design alternative.
4. Regime Theory is a dependable design tool for Punjab canals. There is a need for improvement of design equations being used by Punjab Irrigation and Power Department.
5. For an existing channel, variations in different forms of Lacey's f serve as useful indicators of the channel behaviour.

6. Alluvial Channel Redesign Procedure (ACRP), developed by the Authors, uses accepted relations of Regime Theory, without the disputed estimations of Lacey' T and produces results quite comparable with approaches given by Lacey and other researchers.



Paper No. 541
Year 1992

REMODELLING OF BAMBANWALA CROSS REGULATOR (RD 133296 U.C.C.)

By

ENGR. USMAN AKRAM

Paper No. 541
Year 1982

REMODELLING OF BAMBANWALA CROSS REGULATOR (RD 133296 U.C.C.)

By

ENGR. USMAN AKRAM

General introduction to weir design is followed by introduction to the specific remodelling work highlighting the problems, analysis of the existing structures under different theories / approaches to prove the inadequacy with the existing structure. Next, design requirements are explained and followed by various possible remedies coming to the adopted remedial measure, i.e., remodelling of the existing structure for safety against limiting head across. Further, various options for remodelling were examined before deciding upon Extension of the upstream floor and providing a cut-off at the upstream end of the extended floor, besides constructing a right side encasing wall and a baffle at the end of the downstream floor of Bambanwala Cross Regulator.

The Paper is a role model to compare, analyse and quantify the effects of various design options on relevant parameters i.e. exit gradient, uplift pressures, floor thicknesses, creep coefficient etc in concluding the design adopted. Following are noteworthy:

i. Deepening downstream cut-off and thickening of the downstream floor is fraught with the risk of opening up of

joints between old and new masonry and reducing benefit to zero.

ii. Deepening upstream cut-off whereby uplift pressures reduce under the upstream floor; which are however, already more than counter balanced, by the load of water on the floor. Uplift pressures under the downstream floor reduce nominally and thickening of floor would still be required.

iii. Extension of the upstream floor, although a costlier proposal increasing length of creep for safety against uplift pressures under downstream floor, piping and Khosla's exit gradient was adopted in the instant case.

The remodelling work on Bambanwala Cross Regulator of the Main Upper Chenab Canal (U.C.C.) could obviously be carried out in the annual canal closure period of 18 - 19 days. Knowing this, the work had to be properly planned under Critical Path Method (CPM) besides the traditional Bar Chart and pre-closure activities identified, time lines prepared and actions taken up on time.

The Paper goes on to describe the day and night watch on quality / quantity / progress of the work. The author has very honestly explained the accidents on the work suggesting remedies to avoid such mishaps i.e.

i. Breach in the bund across U.C.C. just upstream of the site to stop seepage water and resultant flooding of the working pit causing damage and delay towards restoring working conditions requiring extension of the closure period.

ii. Overturning of the right side encasing wall 2 feet above floor level into the canal, which the Paper attributes to high backfill pressure of the saturated clayey material as a result of seepage from the headed up supply upstream of the cross bund in U.C.C.

All said and done, the work was completed on and U.C.C. opened to the initial supply at end of the extended closure period.

The summary and conclusions drawn at the end of the Paper provide very useful information and help to all officers of the irrigation departments towards execution of future similar works.

Paper No. 543
Year 1992

DRAINAGE OF IRRIGATED LANDS OF PAKISTAN

A CRITICAL REVIEW

By

DR. NAZIR AHMAD

Paper No. 543
Year: 1992

DRAINAGE OF IRRIGATED LANDS OF PAKISTAN

A CRITICAL REVIEW

By

DR. NAZIR AHMAD

Irrigation Engineering developed in this country during the last 200 years was mainly based upon certain assumptions, actual construction, checking the results and then developing an improved design. This paper is discusses some aspects of drainage, groundwater, tubewells construction and water saving devices currently on ground certain aspects of the design, problems of the operation and maintenance, their cost and present proposals about their disposal have been discussed. These tubewells were designed by Foreign Experts with an assumed useful life of 50 to 100 years.

Another aspect discussed in this paper is the disastrous effect of withdrawal of water of high salinity and its spreading on agricultural lands of the Punjab.

In this paper two more aspects of assumed increased water resources are discussed. Some details of water saving by lining of distributaries and minors is given together with the expected saving of water, including some information about drainage by tiles, design of filter and some suggestions about alternative cheaper method of land drainage.

A chronology of some salient events in this field follows:

In 1950, Pakistan requested the World Organizations to help solve problems of water logging and salinity.

In 1959, WAPDA came into being and with the advice of its Consultants and American Aid started the first SCARP.

In 1961, President Ayub sought help from President Kennedy of USA for solution of the problem. President Kennedy sent Dr Revelle with a team of experts. Dr Revelle's recommendations were received in 1964.

In 1967 World Bank entrusted Peter Leiftinik to prepare a comprehensive Report of Water and Power Resources of Pakistan. Leiftinik also acquired the help of Irrigation Agriculture Consulting Association (IACA) and Chas T Main.

In 1970, Indus Basin Review Mission of World Bank arrived to evaluate Leiftinik report. In 1973, Pakistan Planning Division started an accelerated programme to control the problem.

In 1975, WAPDA with guidance of Harza and other Consultants started a comprehensive study and their report named Revised Action Plan was available in 1979.

In 1990, Water Sector Investment Planning Report become available for completion of the works in progress in all the four Provinces and undertaking new projects during 1990 to 2000.

The advise received fundamentally centered round the use of Tubewells some action on which had already started in this country in 1915 and practical demonstration of their design and working was available in case of 1500 Tubewells of UP constructed by Sir William Stamp in 1932-30 and 1800 proposed Tubewells of Rasul Project Planned by Mr. Haigh in 1944.

The first SCARP Project started in 1960 was fundamentally based upon Tubewells. It was claimed that these Tubewells will last for 50 to 100 years. On operation, the mild steel strainer started to misbehave. All help of scientists could not improve the working of these wells. Rather than to revert to the experienced design of this country, mild steel was replaced by Fibre Glass and such large stocks were imported which could last for all future scarps This new material had its own problems Another serious problem with SCARP Tubewells was the pumping of

deep located saline water In case of SCARP I, about 40% tubewell yielded water with T.D.S. value of 1000 ppm.

By 1985-86 Punjab had 8070 tubewells pumping usable water and 1134 Tubewells pumping saline groundwater The cost of their construction was Rs 36680 million. The estimated expenditure for transition for all fresh water Tubewells of Punjab was Rs 11143.0 millions

In this paper alternative suggestions are putforth to save this national wealth and take measures to keep the Tubewells in operation, increase their yield, and improve the water quality and entrust their operating responsibility to water users.

Farmers Tubewells have saved the Agricultural economy of the Punjab by yielding (about 22 to 25 maf. Water of usable quality and without pumping the operating cost from Public funds The Public Tubewells are designed to yield 70 maf at a very heavy cost.

The serious disadvantage is the deterioration of the quality of water due to rising saline water intrusion into fresh groundwater zone.

With all this experience of Tubewells, Scavenger type Tubewells were installed at a very high cost to pump water as saline as sea water and spread over on to the land surface. Nobody has experimented with a design costing 1/5th the cost of well being installed and with much less chance of pumping highly saline groundwater.

In zones of certain canals, the groundwater is declining Punjab has 60 percent area in which the underground has water deeper them 10 ft Punjab also has about 10 to 15 maft of surface water which runs off during monsoon. It is time that we should start development a recharge technique to conserve surface water in underground formations for our needs.

It is being stressed to save loss of water as canal seepage. Lining of Distributaries and Minors is being practiced. The seepage loss through distributaries has been worked out which is hardly 0.2 cusec per mile. In the Water Sector Investment Planning Report, the loss order from

distributaries has been raised from 7 to 15 percent without giving any supporting data.

In case of water-course the loss has been raised 1025 percent and is stated to be of the same order as in Fields.

Volume of water saved from lining of distributaries is too small and cost of lining already practiced exorbitantly high.

The propagation of lining of distributaries in areas possessing saline groundwater is being propagated without the basic data about the position of existence of saline groundwater.

The first drainage measure for the irrigated area Located in Pakistan, was undertaken in 1904 along the Lower Chenáb Canal just a few years after its construction The problem of rising groundwater and land deterioration by salinity continued in spite of several counter-measures.

Actually with the creation of WAPDA, the working system of Irrigation and Power Department was completely changed. Several consulting Firms and experts found a vast field for their working. The funds for implementing the schemes were provided by World Bank, American Aid and other loan giving agencies.

Three years later in 1970, World Bank sent an Indus Basin Review Mission to evaluate the implementation of the recommendation of Leiftinik Report. Guided by these fresh reports, the Planning Division of Pakistan in 1973, prepared an Accelerated Programme for control of waterlogging and salinity.

About two years later in 1975, WAPDA in association with Harza with funds of United Nations started another comprehensive study in the name of Revised Action Programme for Irrigated Agriculture. The results of this study were made available in 1978 by the Master Planning and Review Division of WAPDA. In 1990, Federal Planning Cell of Pakistan issued an exhaustive report of four volumes on Water Sector Investment Planning.

Drainage Started by Tubewells

With the evolution of Central-fiugal pumps by 1880 and perfection of strainer by 1912, tubewells were started to be installed to pump groundwater and use it for Irrigation. The Punjab Irrigation Engineers were very cautious in the use of this technique and installed this system at a few sites. The first attractive scheme was prepared by Sir William Stamp who got installed 1500 tubewells in United Province (U P) of India in 1930 32 The Punjab Engineers constructed 16 tubewells in Amritsar area in 1920, and twenty two tubewells in the Karol area near Shalamar Gardens in 1938-40. A Rasul Tubewell scheme was got sanctioned in 1944 under which 1800 tubewells were to be installed along Major Canals of Rechna and Chaj Doabs.

The installation of these tubewells was in progress at the time Independence It was thus possible to complete 1526 tubewells by 1951. 25 tubewells were got installed in the Chuharkana area during 1952-54 In this period a soil Reclamation Board was created by the Punjab Government which helped to get another 41 tubewells installed in Jaranwala area, 12 tubewells in Chichoki Mallian and 21 tubewells in Pindi Bhattian. Some of these tubewells in Jaranwala and Chichokimallian area were provided with turbine pump to withdraw higher discharges of 3 to 4 cusecs

Design of Rasul Tubewells

The Rasul Tubewells had a slotted brass strainer made out of brass sheets. Its usual length was 100 to 120 feet At top a blind pipe, 30 to 40 feet in length and five feet long bail plug at bottom was provided.

Design of SCARP tubewells

In 1961, the construction of SCARP tubewells was started. Each tubewell vas to be located near an outlet, the pumped water was to be used mixed with canal supplies. The capacity of each tubewells was fixed to increase the available water required to raise the agricultural intensity to 150 percent. The capacity of majority of tubewell was between 2.5 to 3.5 cusecs. For an average yield of 3.0 cusecs the depth of each tubewell was invariably more than 200 feet. The Consultants

considered the use of brass strainer too costly. They decided to import seamless mild steel pipes of 10 inches bore. Slots to a maximum width of 1/16 inch and about 8 percent open area were drilled locally.

On the protests of the farmers, many tubewells yielding highly unfit water had to be closed. The groundwater quality of tubewells of this SCARP between 1962-63 and 1980-81 deteriorated significantly.

First report on the performance of these tubewells was issued for the period of October 1964 to September, 1965. It was stated that out of more than 2000 tubewells 1712 were progressively deteriorating, 53 had seriously deteriorated, and 47 were also heading towards deterioration. There were some tubewells which pumped sand and then suddenly got sunk into the formation. Two or three tubewells thus got damaged every month. Thirty nine tubewells had already been rebored. The discharge by that time had declined by 3637 cusecs out of designed capacity of 5877.3 cusecs.

Then very extensive investigations for the causes of so quick deterioration of these tubewells were started. Vast useful information about the presence of Chemicals in water and formation and the type of salts in existence were gathered.

A few rehabilitation measures were tried but without much success. The Consultants rather than to revert to the use of tried brass strainer and groundwater withdrawal through the design of Rasul Tubewells decided to us strainer and pipe made of fibre glass bonded with epoxy, resulting in increased failures.

The average discharge of each tubewells was about 3.5 cusecs or 7 A. ft per day. If these tubewells operate for 182 days in a year the total salts being pumped out by tubewells of varying salinity are as under:

1	SCARP Tubewells	13.6 m.tons
2	Salts out of Saline Water being pumped	42.5 m.tons
3	Salts being added from Water of farmer's Tubewells	25.95 m.tons
4	Salts from Irrigation Water	10.4 m.tons
	Total	92.45 m.tons

The area under irrigation in Punjab is 20.0 m.acres so that each acre is receiving 4.62 tons of salts per year. Some alternative measures need to be utilized to reduce the spread of these salts.

Some Causes for the Unsatisfactory Performance of SCARP Tubewells

SCARP-l tubewells started operation in 1962-63. Some problems about yield of water started appearing. The operating agencies did not realize that tubewells performance is like the working of a machine which needs off and on some minor repairs. An important component of a tubewells is the turbine pump which is contained in deep groundwater in a -housing. There are no performance rules to examine its components after a certain period of operation. Usually the pump is extracted only when some trouble has advanced too far.

The second important component of a tubewell is the- strainer which is made of a mixture of different ingredients some of which are reactive with water. Reactive materials commonly contained in the formation are carbonates of Calcium and Magnesium which change to soluble bicarbonates by water containing Carbon Dioxide. Bicorbonate release carbon dioxide at low pressure due to suction of the tubewell and changes into solidified Calcium and Magnesium Carbonates. It is also known that fungus is often found in standing water. It is due to some bacterial action. The presence of different types of bacteria such as sulphate reducing or iron reactive has been established - to exist in the Indus formation. Colloidal clays produced from clayee deposit have electrical charge. These are a few well know parameters which can change the character of the underground formation near the zone of an operating tubewell. Thus it is very necessary that there need to be some operating rules, to counter act these obvious causes of deterioration of yield of a tubewell. We know that for each barrage certain procedures are framed for its operation. Still every year after the flood season each barrage is examined and any defect found is rectified. Tubewell is such a device which is considered to be so perfect that after its construction for years one expects no deterioration and damage. It is really disappointing to note that in 32 years since the start of SCARP tubewells no manual for operation and maintenance and procedures for upkeep of a tubewells have so far been issued.

Recharge Technique Needs Perfection

For Pakistan recharge technique is as important as the withdrawal of water by tubewells. The lndus formation is most suitable for recharging welts. It has a thin top soil about 5 to 20 feet thick. It is then followed by sand deposits. The sand of the plains is finer than coarse sand of particles smaller than 0.5 mm, but it is an alluvia deposit transported by water so that each deposit has high uniformity between 1.25 to 2.5. The pores of sand constitute about 40 percent and a saturated sample can drain upto 25 percent. Thus surface water can saturate the formation. The Indus formation particularly in the Punjab is made up of about 70 percent sand. The remaining 30 percent is not all clay deposit. It also contains particles made up of low yielding silt.

A large volume of water supplies in rivers during winter is out of recharge from the valley storage. This water had seeped into the formation during high flow summer periods and drains out during low supplies of rivers. Thus if it is arranged to recharge the sandy formation during Monsoon when large volume of surface water is available, we can utilize groundwater during the dry months. So far, we have not utilized any of the recharge method.

The source of water for drinking in Cholistan is out of rain water which is conserved in depressions. This water is used both by residents and animals. The water collected just after rain-fall is of low salts contents which are washed from surface rainfall run off. Cholistan has high evaporation of the order of about 80 inches per year. Water thus continues to evaporate resulting in increase of salt accumulation. The groundwater in the area is generally saline and 70 to 100 feet deep. At a few sites, there exist pockets of fresh water probably, as remants of seepage of Hakara River which has dried out. There is a great possibility of conserving rain water in the underground formation. The rain water after removing the sediment may be recharged into the formation.

Water Losses Through Distributaries and Minors

WAPDA, HARZA and IACA endeavoured to develop a method to determine seepage loss of each canal command on the basis of soil

texture under the bed of a canal. They used soil category, water delivery category and loss category and on these basis estimated the loss through various parameters of a canal such as main line, branches, distributaries and minors. The loss of water from water-courses and Farms was also determined. They also worked out the recharge from these two parameters separately. This information was put forth in Groundwater Development and Potential of Canal Command, a Supporting publication of Water Investment Plan issued in July, 1981. This very information is included in Groundwater Development Potential of Canal Command Area, issued by Water Resources Planning Division of WAPDA in November, 1988.

Volume of Water Saved by Lining of Distributaries and Minors

The canals in Punjab are either perennial or non-perennial. The length of perennial distributaries of the Punjab is 9621.9 canal miles and those of minors are 6436.8 canal miles. The seepage used at the rate of 0.2 cusec per mile gave 1.822 ma. ft. No lining can be hundred percent impervious. The amount of seepage will reduce according to the type and condition of lined canal.

Paper No. 554
Year 1994

ON THE FLOOD FREQUENCY ANALYSIS AT IMPORTANT DISCHARGE MEASURING SITES OF PAKISTANI RIVERS

By

SYED ALI RIZWAN
&
MUHAMMAD AZAM CHAUDHARY

Paper No. 554
Year 1994

ON THE FLOOD FREQUENCY ANALYSIS AT IMPORTANT DISCHARGE MEASURING SITES OF PAKISTANI RIVERS

By

SYED ALI RIZWAN
&
MUHAMMAD AZAM CHAUDHARY

A study is reported on the topic which is a continuation and significant extension of the previous work published and presented in the proceedings of Pakistan Engineering Congress (1 & 2). The data is based on actual maximum yearly flows at almost all important discharge measuring sites of Pakistani rivers. At the end normalized flood frequency curves based on Pearson's type III distribution have been plotted on the log-probability paper in terms of normalized discharges, percentage exceedance of previous yearly maxima and corresponding return periods. The results have also been compared by graphical method and Gumbles distribution approach. Picking up an arbitrary constant flood level of 200,000 cusecs, the corresponding probability of future occurrence of this magnitude of flood is reported alongwith the necessary supporting hydraulic data plotted again on probability paper. The results of analysis are very interesting and may

[1] Professor of Civil Engineering, UET, Lahore Pakistan, M. ASCE
[2] Executive Engineer, Taunsa Barrage Division, I & P Deptt, Government of Punjab

be used to determine the probability of occurrence of any given magnitude of flood at the desired discharge measuring sites and also to judge the adequacy of maximum design discharges used for the existing hydraulic structures considering return periods alongwith their useful service life from structural and hydraulic considerations.

The phenomena of floods is common in Pakistan. In order to get a better probabilistic idea of the problem based on theory of probability, present investigation has been undertaken. Gaussian or normal distribution is one of the most commonly used probability distribution which is closely approximated by most of the natural phenomena. This distribution is completely specified by the two parameters, viz population mean and population variance in general. Usually the size of sample in hydrological records is not infinite as assumed therefore usually an assumption is made that the sample mean is equal to population mean and the sample standard deviation is equal to that of population standard deviation. Generally stream flow data does not seem to be adequately described by the normal distribution because the lower bound on stream flow is zero rather than minus infinity. However if logarithms of stream flows are used in the analysis, the resulting data conforms to known characteristics of normal distribution as when stream flow approaches zero its logarithm approaches minus infinity(3). The method of frequency curve computations has already been explained in detail in an earlier investigation (1), however, a brief mathematical formulation is given below. Analytical method of frequency cuve computation is almost exclusively limited to maximum annual stream flows as it given more reliable results than those predicted by graphical methods (3, 4). Logarithmic Pearsons type III distribution requires three parameters in general for the complete specification of the hydrologic problem mathematically. These parameters in general for the complete specification of the hydrologic problem mathematically. These parameters include sample mean X, sample variance S^2 and skew coefficient g. If g is taken as zero, for less than 100 events, this distribution becomes a two parameter normal distribution.

The following steps are taken to plot the probability curves.

1. For selected values of P∞, tabulate k values obtained from table 1 corresponding to adopted skew coefficient (Zero for less than 100 events).

2. Multiply each of these by standard deviation and add each product in turn to the mean logarithm according to equation 4.

3. Tabulate Pn values from table 3 corresponding to the selected P∞ values and the value of N-1 wherein N is the number years of record.
4. Plot antilogarithms of each of the sums obtained in step 2 above against corresponding Pn values obtained in step 3.
5. The normalized discharge is defined as the ratio of maximum yearly discharge to the average discharge corresponding to the years of record. On the probability paper, originally designed by the Codex Company of USA, the normalized discharge has been plotted as ordinate and the other plotting procedures remain the same as was done in an earlier investigation (1). On one graph all the curves corresponding to important discharge measuring sites of a particular river have been plotted.

A side investigation at some sites have also been carried out by using Gumbles Distribution utilizing the probability weighted moment method for parametric evaluation of ά and μ and the results are presented in table 6. However it must be kept in mind that every distribution is based on certain assumptions and therefore if a comparison is desired between results of any two distribution, their assumptions must be kept in mind before the interpretation of results.

The results of the probability analysis have been presented in form of probability cuves and tables. The normalized curves correspond to rivers Chenab, Indus, Jhelum and Ravi respectively. A good agreement between the observed and the fitted curves pertaining to Pearsons type III distribution and Gumbles Distribution exists due mainly to the availability of a lot of data points in that range. However, a scatter exists at higher flood levels due to availability of lesser number of points.

PREDICTION OF SELECTED HIGH FLOODS BY GUMBLES DISTRIBUTION

River / Site	Flood Level x 1000 cfs	Return Period Years
Chenab / Khanki	1100	200
Jhelum / Rasool	800	81
Indus / Sukkur	1200	58
Sutlej / Islam Head	400	130
Ravi / Balloki	200	40

CONCLUSIONS

1. The probability curves and the parameters calculated in the tables are very useful in understanding the behavior of rivers in terms of their flood producing characterstics on the basis of laws of probability. Many things including the flood producing characteristics of a river at any of the analysed discharge measuring site may be known in terms of average flood, calculated skew coefficient, standard deviation, return periods and probability of occurrence/exceedance of previous maxima of any magnitude of flood at any site which of course is a very useful and interesting.

2. The flood peaks in the fitted lines may deviate from the observed ones at higher flood levels due to breaching effects.

Paper No. 565
Year 1994

THE INCIDENCE OF RUTTING ON BITUMINOUS ROADS

By

ENGR. SHAUKAT ALI

Paper No. 565
Year 1994

THE INCIDENCE OF RUTTING ON BITUMINOUS ROADS

By

ENGR. SHAUKAT ALI [1]

As a result of satisfactory performance as road paving material in US $ 27 Million AASHO Road Test conducted at a location near Ottawa, Illinois about 80 miles South West of Chicago, bituminous concrete began to be accepted as the standard paving material in most of the American States. There were so many methods of bituminous mix design prevalent at that time but it was the U.S. Corps of Engineers Marshall Method which was universally accepted. The Marshall Method of Mix Design, was primarily meant for temperate climates where the road surface temperature was not to exceed 60°C with standard axle loading. Without keeping in view these limitations, the German, British, Italian and American Consultants paved all the Roads and Highways in Saudi Arabia, and other parts of Middle East and Asia using Marshall Method of Mix Design. Not long after the construction of these roads wherever the road surface temperature substantially exceeded 60°C and the axle load was also in excess of the standard axle longitudinal depression or rutting was noticed.

U.S. National Research Council realizing the necessity of constructing more durable roads set up a US $ ISO million Strategic Highway Research Programme in 1987. The research under this programme was

[1] General Manager (Civil), Nazir and Company (Pvt.) Limited, Lahore-54600

initiated in 1988 and was finalized in March 1993. The results of the Strategic Highway Research Programme are neither fully known nor can be made use of before the end of the second millennium since long-term pavement performance test results being carried out on 1000 pavements in 50 State/countries would not be available earlier than next 20 years. So far as Pakistan is concerned the research carried out by the Americans would not be of much help to us for still many more years because the newly devised field and laboratory testing equipment required for this purpose would not be commercially manufactured so soon.

In this milieu an endeavour has been made, in this technical paper, to highlight the mechanics of rutting not hitherto force attempted in as much details and depth and to suggest measures to minimize it indigenously and cost-effectively till "Superpave" mix is evolved as a result of the new research. Follow-up research on various issues has also been suggested to be carried out in Pakistan by various laboratories involved in highway testing.

The Incidence of Rutting on Bituminous Roads

In order to comprehend the mechanics of rutting it is necessary to understand various parameters which affect the behaviour of asphalt concretes from manufacture in the asphalt-plant to performance on the road. All the parameters which increase or decrease rutting have been discussed. At present there is no mix design which takes into account the variations occurring in the mix over the full range of temperatures to which a mix gels subjected in hot weather nor does it measure directly its rutting resistance at different temperatures.

Mix Constituents and Composition

In a bituminous mix bitumen is the binding or cementing material and its tendency to rut primarily depends upon the quality and quantity of bitumen used in the mix. By manipulating mix composition and constituents the tendency to rut can greatly be modified and controlled. If for the sake of durability the quantity of bitumen is increased there will be greater tendency in the mix to rut because, its cementing

strength gets weakened with decrease in viscosity at higher than normal temperatures. Similarly if highly viscous bitumen is used its rutting potential will get reduced but it will have greater tendency to crack in cold weather. Therefore, more viscous the bitumen the lesser the rutting tendency.

Rutting can be controlled to some extent by increasing the resistance of the mix to deform. One way of increasing strength or stability is pragmatic manipulation of coarse and fine aggregates.

Traffic Polishing and Smoothening

On completion when a road is opened to traffic, its surface gets smoothened and polished with passage of traffic; specially lighter traffic like cars, vans, wagons and buses. This smoothening and polishing effect influences the behaviour of the laid asphalt concrete. It not only further increases its impermeability and seals it off against the possible ingress of water during rains and other modes of precipitation but it also decreases evaporation of volatiles from the mix. With the absorption of fines from the air as well as the dusty surroundings the top of the road surface becomes more resistant to rutting than its interior.

Oxidation and Weathering

Both the parameters of oxidation and weathering influence the pavement rutting potential as well as development of alligator or block cracking and make the mix stiffer and stiffer with the passage of time. A bituminous mix becomes harder and harder when exposed to air. This hardening process primarily takes place in the bitumen or asphalt and makes the mix less liable to rut with the passage of time as compared to a fresh mix. A newly laid mix therefore, is more likely to rut.

Road Pavement Temperature

Rutting has much to do with the road pavement temperature. In both Hveem and Marshall methods of mix design there is an implicit limitation that the road pavement temperature would not rise above 60°C or I40°F. This is why the resistance to deformation or stability is

measured by application of the load at 60°C. Since both Hveem and Marshall methods measure mix strength at 60°C, they can only be relied upon if the pavement temperature does not exceed 60°C.

It was observed during the construction of Section of National Highway N-5 that the road pavement temperature goes as high as 75 °C during day time. Road pavement temperature could be still higher in such hotter areas as Sibbi and Jacobabad. Hveem or Marshall mix designs which arc based on the upper higher pavement temperature limit of 60°C arc apt to rut or deform in such areas.

Axle Loading

Axle loading is closely related to rutting. There will be no rutting if-the axle load is lighter. Car traffic whatever its density and intensity can never produce any rutting. It is the heavy axled traffic alone which produces rutting when the road pavement temperature is high. There are, therefore, very remote chances of rutting taking place in very cold climates and weathers if the mix is properly designed and bitumen contents are not very high.

Rutting in the Global Context

Realizing the mal-effects of the malaise of rutting various endeavours has been made at international level either to eliminate rutting totally or to minimize it. Strategic Highway Research Programme under the National Research Council of America, French Method of Mix Design for hot countries under the LCPC (Laboratory Central dcs Ponts et Chaussccs) and the Franco - Israel technique of Mix Design by Moshe Livneh are some of the global attempts to resolve the problem of rutting.

The Franco - Israel Method

The method proposed by Dr. Livneh Director Technician of the Israel Institute of Technology Haifa is an improvement of the Marshall Method for the interim period till sophisticated and state-of-the-art equipment as deployed by the French becomes available. Dr. Livneh

suggests changed Marshall mix design criteria for hot countries and specifically recommends that as per Israel experience water absorption of aggregates should be less than 2.0 per cent and harder grade bitumen than 85/100 penetration should be used in hot climatic regions like Southern Israel. As far as Pakistan is concerned all of our approved sources of aggregates have water absorption much lower than 2% and harder grade bitumen is also being already used. As per Pakistan experience all aggregates having water-absorption higher than 2% fail in other quality tests.

Review of the Global Efforts

At this juncture when various aspects of the phenomenon of rutting have been adequately highlighted and global endeavors to resolve the problem have been explained, it would not be inappropriate to see if the work done by the Americans, the French and the Israelis would be applicable to us and if yes to what extent. In the first place there should not be any doubt or ambiguity that whatever research work is being done at international level, is primarily meant for the nations who are carrying the research. The American, French or Israeli climatic, traffic and loading conditions are not the same as in Pakistan and their solutions to the problem of rutting cannot be applied without understanding the basic limitations under which the solutions were primarily derived. It is evident that it is the Pakistanis alone who have themselves to find solutions to their problems and it would be too much to expect solution from people foreign to our land who do not know the solution themselves. This is also understandable because they neither know the extent of our axle loading, loading habits of our transporters nor the peculiar hotness of our weather. Foreign Highway Consultants are therefore least bothered about our problems and tend to give solutions which could be appropriate for them but would be inapplicable in our conditions.

What was missed by the Americans in the post-test evaluation and appraisal of AASHO Road Test in sixties is again being missed in the SHRP and other global efforts to resolve the issue of rutting. Any research which does not cover the total temperature spectrum of road pavements especially for hot climatic region is apt not to succeed and to

be fruitful to us as well as all those countries which are very hot. It can, therefore, safely be concluded that work done by the Americans, the French and the Israelis would not be useful for us to the extent it could have been, had these temperature limitation been observed. The people living in temperate and cold climates are likely to forget that there are countries in the world where temperature during the day remains close to 50°C and that too for weeks and months together without any cooling wind-flow at nights and shaded trees along the roads and highways during the day.

One thing, however, is certain from the work carried out by the French and the Israelis that the present Marshall Method of mix design cannot be applied and used in a hot country. This was pointed out in Engg. Congress Technical paper No. 41 much earlier than the Israelis though little attention was paid to what was said in the Paper. Prior to the publication copies of the paper were sent to all the Engineers who were responsible for deciding the fate of highway construction in Pakistan.

Indigenous Solutions to the Problem of Rutting

Rutting can be eliminated by evolving a bituminous mix having high resistance to deformation at road temperatures higher than 60°C under repeated applications of heavy axled traffic. To some extent resistance to deformation is indicated by the mix stability. It is, however, not a dependable indicator of resistance to deformation as it is highly variable at different temperatures. Stability measured at a fixed temperature of 60°C is, therefore, not a dependable indicator of rutting resistance though it does indicate the ability of a mix to resist deformation at temperatures upto 60°C. It is now an established fact that road surface temperature in hot climatic regions exceeds 60°C and goes beyond 75°C.

In the present milieu and in the present stage of development of bituminous mixes bituminous concrete cannot be used in climatically very hot regions of Pakistan without fear of rutting till we carry out our own basic research, develop our own modifiers and additives and build our roads accordingly or we make intelligent use of the research carried out at international level. The present altitude of those who allow

ruttable mixes to be placed on highways and motorways is ultimately going to cost the Nation tremendously.

Indigenous and most cost-effective solution to the problem of rutting is to lay 2" to 3" or even bigger size hand broken stone aggregate over a well compacted sub-base in loose thickness of 6" to 8" and compact it with Vibratory Rollers of adequate weight and vibration till it gets almost compacted and evinces no sign of movement. The aggregate must have all sides broken and should produce maximum interlocking. Los Angeles abrasion value of the aggregate should be below 30 and sulphate soundness below 10. The interstices between the 2" to 3" stone aggregate arc to be filled with properly designed bituminous slurry using fine aggregate passing sieve No. 8. The slurry can have sulphur, synthetic rubber or other additives to improve its interstices-filling qualities. After filling the interstices with slurry, the layer can be further rolled till it gives 100 % modified compaction. Similarly another layer of hand broken stone having maximum interlocking can be laid as per requirements of the design structural number. The interstices between the aggregates can also be filled with any other void filling material as cheaply available or found suitable with experience. In other words for the time being till we find more suitable interstices filling material we can lay bitumen bound macadam instead of the water bound. If properly laid and filled with bituminous slurry such a bituminous concrete would neither rut nor evince flow at the edges nor would require such an expensive fleet of machinery as is required for normal laying of asphalt concretes. Moreover, there would be no chances of reduction of layer coefficient in summers. In our case the role of the slurry will be not only that of filler but also of aggregate binder in such a manner that it's softening or reduction of stiffening would not affect the load bearing strength or stability of the bitumen bound macadam. The whole purpose of BBM is to let the stone aggregate take the total brunt of stress and strain of heavy traffic and let the bitumen play the secondary role of void-filler and aggregate binder. This role of binder would only come into play if per chance the aggregate interlocking fails and the stress get transferred to bitumen. Voids can also be filled with finely ground slurry of lime and slag or other pozolanic materials capable of forming cementitious compounds when wet.

For providing extra smoothness the top of the bitumen bound macadam can be overlaid with bituminous concrete using 6mm aggregate. If, however, skid-resistant surface is required it can be given one or two coats of surface treatment.

Paper No. 578
Year 1996

EXPERIENCE GAINED FROM INTERCEPTOR DRAINS INSTALLED IN LBOD STAGE-1 PROJECT

By

YAWAR HAMID

Paper No. 578
Year 1996

EXPERIENCE GAINED FROM INTERCEPTOR DRAINS INSTALLED IN LBOD STAGE-1 PROJECT

By

YAWAR HAMID[5]

About 500 000 ft long interceptor drains along with 54 pump stations have been installed in Nawabshah Sub-project of LBOD Stage-1 Project and are in operation since 1992. This paper describes the experiences gained from sub-surface investigations, design and operation of interceptor drains in LBOD Stage-1 Project. If proper sub-surface investigations are conducted and drains are installed only along canals reaches underlain by coarse textured soils, then the interceptor drains become an economically viable means for the interception of canal seepage.

Interceptor drains were installed in Nawabshah Sub-project of the LBOD Stage-1 Project during the period February 1990 to October 1992. These are sub-surface pipe drains installed parallel and close to Nasrat, Amerji and Gajrah branch canals of Rohri canal system. Rohri canal offtakes from Sukkur Barrage located at the river Indus. Corrugated and perforated PVC pipes surrounded by an envelope material were laid as interceptor drains at depths ranging from 7.5 to 11 ft below ground surface. Design distance of the drain from the outer toe of canal embankment was 30 ft for a single drain line; and 30 and 90 ft for a twin drain line. Actual drain distances varied according to the site conditions. Diameters of the drain pipes were 6, 8, 10 and 12 inches. Total length of the installed drains, along 255 000 ft length of canals, is 499 730 ft. Fifty four pump stations having total installed capacity of

[5] General Manager, Water and Agriculture Division NESPAK.

126 cusec have been constructed to pump out the drain effluent into the adjacent canals. The drains are designed to intercept 65 cusec of canal seepage. Total construction cost of the drains including pump stations and cross drainage structures was Rs 256 million [1].

In addition to the installed interceptor drains in Nawabshah Sub-project, 654 000 ft long interceptor drains along with 111 pump stations, having total installed capacity of 173 cusec, are proposed for Sanghar and Mirpurkhas Sub-projects of LBOD Stage-1 Project. The proposed drains are designed to intercept about 160 cusec of canal seepage from Mithrao canal, Jamrao canal, Dim branch, Shahu branch and West branch of Nara canal system off taking from Sukkur Barrage.

The objectives of installation of interceptor drains are:

1. To intercept and reuse canal seepage before it mixes with highly saline ground water and is rendered useless for irrigation.
2. To reduce the sub-surface drainage requirements of the area adjacent to canals.
3. To reduce the size of saline effluent disposal system.

Sub-surface investigations were carried out along the canals of LBOD Stage-1 Project, where interceptor drains were proposed to be installed. Ten ft deep auger holes were drilled at selected locations to determine soil texture at different depths, hydraulic conductivity by pump out test, depth to water table and water table profiles by drilling 5 to 6 auger holes along, sections starting from the canal water edge and extending upto 300 ft away from the canal. Some of the auger holes were drilled upto 16 ft depth.

Water table Profiles

Ground water profiles were prepared at 188 locations along the project canals. Five to six auger holes were drilled along a section starting from the canal water edge and extending upto 300 ft away from the canal. Three typical types of the water table profiles for Amerji, Gajrah and Nasrat branch canals have been identified as shown in Figures 1 to 3.

Type-I profile shows an almost flat water table starting from the edge of

the canal and extending upto the observation distance. However, seepage from the canal creates a small water table mound near the canal. Type-II profile is intermediate between Type-I and Type-Ill profiles. Type-II profile shows a relatively high water table mound below the canal and an almost flat water table away from the mound. Type-Ill profile shows a clear seepage line starting from the edge of the canal and extending upto 300 ft away.

Design values of drain interception rate for Nawabshah Sub-project were 0.07 to 0.25 cusec/1000 ft of drain.

For Sanghar and Mirpurkhas sub-projects the lower limit of design interception rate has been fixed as 0.15 cusec/1000 ft and no drains will be installed at places where estimated interception rate is less than this limit. Design interception rate ranges from 0.15 to 0.35 cusec/1000 ft of drain.

Canal Seepage Interception by Two Drains

For LBOD Stage-1 Project; where average drain depth is 10.0 ft, depth to water table is 7.0 ft and average value of an-isotropy ratio is 8; drain interception by a single interceptor drain varies from 25 to 30 percent of the canal seepage. If a second drain is also installed at a distance of 100 ft from the first drain, then the total interception is about 1.5 times the interception by one drain. If the depth to water table, remote from the canal is shallower than 7.0 ft, then seepage interception by two drains increases and is between 60 to 80 percent of the canal seepage when depth to water table is 5.0 ft [3].

PERFORMANCE

Drain Discharge

To check the performance of the interceptor drains discharge of every drain length was measured by the contractor after the completion of the installation works in Nawabshah Sub-project. During subsequent measurements, only the combined discharge of all the drains out falling into a pump station was estimated by measuring the recovery of water level inside the pump sump. Discharges of 54 pump stations, measured

at different timings, are given in Table 1.

Drain interception rates, at the time of acceptance tests, ranged from 0.039 to 0.94 cusec/1000 ft of drain. Actual drain interception rates differ from the design rates because of the following reasons:

1. Actual water table depth is shallower than the design depth of 7.0 ft.
2. Actual hydraulic conductivity of the soil is different from the value adopted for the design. For the design purposes one or two point values of hydraulic conductivities are used for a single drain length. Whereas, actual drain discharge is based on the actual hydraulic conductivities of the soil strata falling within the entire length.
3. Actual an-isotropy of the soil is different from that adopted for the design.

Fig. 5 shows the variation of discharge with time for some of the selected pump stations. Table 1 and Figure 5 show that discharge of almost all the pump stations have reduced. This is because of the fact that, due to the operation of interceptor drains and drainage wells, water table has gone down and as discussed in drain design that drain discharge is inversely proportional to the depth to water table. Hence, the drain discharge is supposed to decrease till the water table is stabilized and attains design depth of 7 ft [4,5]. However, from Table 1 and Fig. 2, it appears that the discharge of interceptor drains have become almost constant to a value of about 76 cusec against a design discharge of 65 cusec.

Minor changes in the discharges of individual pump stations are due to fluctuations in the depth to water table.

Drain Water Quality

A single line of interceptor drain installed close to the canal, when depth to water table remote from the canal is 7 ft, intercepts water equal to about 30 percent of the canal seepage from one side while the rest of seepage water mixes with the ground water. Water quality of the drain water indicates that it is not pure canal water but some saline ground

water has also mixed with it and thus has increased its salt content. Electrical conductivities of canal water, drain water and ground water are 300 to 500, 410 to 1850 and 7000 to 27000 micro mohs per centimetre, respectively. Electrical conductivity of drain water from most of the pump stations is less 1000 micro mohs per centimetre. Average value of the electrical conductivity of drain water, at the time of acceptance tests, was 850 micro mohs per centimetre [4].

Induced Seepage from the Canals

The shape of Type-I profile indicates that there is no hydraulic connection between the water table and water in the canal. Water seeps from the canal at a constant rate depending upon the soil characteristics and canal geometry. Canal seepage is not affected by the changes in depth to water table. Hence, with the installation of interceptor drains there will be no effect on the canal seepage and no induced recharge will take place. Such profiles canal be related to sites where the canal passes through fine textured soils and hydraulic conductivity of the sub-strata is relatively low.

The shape of Type-II profile indicates a very poor hydraulic connection between the water table and water in the canal. Water seeps from the canal at a constant rate depending upon the soil characteristics and canal geometry but at a relatively higher rate. Canal seepage is not affected by the changes in depth to water table. Hence, with the installation of interceptor drains there will be no effect on the canal seepage and no induced recharge will take place. Such profiles canal be related to sites where the canal passes through fine textured soils and hydraulic conductivity of the sub-strata is relatively more than that for Type-I-profile.

The shape of Type-III profile indicates a hydraulic connection between the water table and water in the canal. Water seeps from the canal at a variable rate depending upon the depth of water table, water level in the canal, soil characteristics and canal geometry. With the installation of interceptor drains there will be an increase in the canal seepage due to lowering of water table or induced recharge will take place. Such profiles can be related to sites where the canal passes through medium and coarse textured soils and hydraulic conductivity of the sub-strata is

relatively high.

Recommendations

Following recommendations are made for the design of interceptor drains from the experiences of LBOD Stage-1 Project:

1. Extensive sub-surface investigations should be carried out before the design of interceptor drains. Additional cost incurred on investigations is compensated by the savings due deletion of interceptor drains in low seepage reaches.
2. Interceptor drains should be installed along canal reaches which are underlain by highly saline ground water and if tube wells were installed to intercept the canal seepage would yield saline water.
3. Interceptor drains shall be installed along canal reaches which are underlain by coarse textured formations so that more water flows to the drains. Drains shall not be installed along canal reaches where soil layers of low hydraulic conductivity are found within drain depth.
4. Canal reaches where depth to water table, remote from the canal, is more than 8 ft, interceptor drains should not be installed.
5. Canal reaches where estimated drain interception rate is less than 0.15 cusec/1000 ft of canal length, interceptor drains should not be installed.
6. Drains should be installed as close to the canal as possible. In this way maximum canal seepage will be intercepted and length of the pump discharge pipe will be minimum.
7. Synthetic geo-textile fabric envelope performs well in sandy soils, therefore, can be used if natural gravel is very costly or not available.
8. Synthetic geo-textile fabric is a choice envelope material for the sites where gravel cannot be transported due to difficult soil conditions.
9. All the drain pipes should be tested, after installation, by pulling a steel bar ball through the pipes.
10. Drain water should be disposed off into main and branch canals only.
11. Pumps should be installed at least 2 to 3 ft above the ground surface and a strainer shall be provided at the suction end of the

pump.

12. Cost of the pump station can be reduced by reducing the sump diameter, elimination of standby equipment and by using simple motor control panel.

Conclusions

Following conclusions can be drawn from the experience of interceptor drains installed in LBOD Stage-1 Project:

1. Interceptor drains are economically feasible if installed at proper location and according to proper design.
2. If, depth to water table is shallow i.e. close to 5.0 ft, remote from the canal, two or more lines of interceptor drains can be considered for installation.

Quality of drain water is acceptable and is not hazardous to the canalwater.

Paper No. 606
Year 1998

EFFECTS OF UPSTREAM STORAGES ON THE PRESENT ECO-SYSTEM IN AREAS DOWNSTREAM OF KOTRI BARRAGE

By

ENGR. BARKAT ALI LUNA [1]
&
ENGR. MUHAMMAD JABBAR [2]

Paper No. 606
Year 1998

EFFECTS OF UPSTREAM STORAGES ON THE PRESENT ECO-SYSTEM IN AREAS DOWNSTREAM OF KOTRI BARRAGE

By

ENGR. BARKAT ALI LUNA [1]
&
ENGR. MUHAMMAD JABBAR [2]

The National dialogue on the proposed Kalabagh Dam raised the issue of possible effects of new upstream storages on the present ECO-System downstream of Kotri Barrage. This has been the subject of heated discussions for quite some time in the past. Different views have been presented by different people and the issue has been politicized quite unnecessarily. The authors believe that it is imperative to place the facts and non-facts about the issue before the profession and the public. Through this paper, the authors have attempted to distinguish facts from non-facts about the issue. Relevant conclusions from this paper are abstracted below in the following summary:-

There will be no adverse effect due to Sea water intrusion because the groundwater in the entire reach from Kotri to Sea (174 miles) is already saline and hazardous for irrigation and drinking purposes. There is no possibility of its improvement in future even if the river flows downstream of Kotri Barrage are increased.

[1] EX-DIRE IRRIGATION RESEARCH INSTITUTE PUNJAB
[2] EX-GENERAL MANAGER, (WAPDA)

The Indus Active Delta area is already (almost) devoid of mangrove forests. Only 2.3% of the mangroves exist in this area. The remaining 97.7% exist on the west and east of the Delta area where the mangroves are thriving. They consist of the high salt tolerant species' "Avicennia Marina" which can also be grown in the Delta area. As such the construction of any dam of the Upper Indus will not produce any adverse effect on mangroves in the Indus Delta area. Fish movement in the Indus River is presently suffering due to the defective design of fish ladders at Kotri and Sukkur Barrages. A small reduction in the downstream Kotri releases during summer as a result of construction of any dam on the upper Indus will have no effect on fisheries in the Delta area.

The productivity of the riverine forests is already on the decline because of the reduced extent and duration of annual flooding. Irrigated forestry is essential for the maintenance of forest ecosystem which is possible from the canals running outside the flood embankments.

Reverine irrigation is already in poor shape with low and erratic yields. Pumped irrigation is already being practiced in some areas which can be extended to the rest by installing pumps on the existing canals outside the flood embankments.

There are about 200 villages in the riverine area out of which about 135 villages get their domestic water supply directly from the canals of the Kotri Barrage running parallel to the bunds. The remaining villages can be served from the canals proposed for irrigation. This arrangement will improve the present situation.

Under the provisions of the Water Apportionment Accord, Sindh will be the maximum gainer from the stored supplies if a dam is built on the Upper Indus and Sindh will be worst hit, in case no dam is built.

Recommendations for Minimum Flows Downstream Kotri

The study recommended minimum flows downstream Kotri, on 10-days period basis, to meet all requirements discussed above. These minimum are given in Table-3 and same are summarized in the Box below.

BOX Recommended Minimum Flows Downstream Kotri

Sr. No	Item	Recommended Flows (MAF)	Remarks
1.	To check sea water intrusion	Nil	-
2.	To maintain mangrove forests	Nil	-
3.	To maintain riverine forests	0.64	As per recommend ations in sub-para 6.4
4.	To dilute saline ground water seepage entering into Indus	0.08	To met the present need as Explained under sub-para 8.2
5.	To maintain riverine irrigation	0.31	As per recommend ations in sub-para 7.4
6.	To maintain Palla fish migration	1.79	As per recommend ations in sub-para 5.4
7.	Domestic water supply	Nil	
	TOTAL	2.82	

Paper No. 650
Year 2000

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

By

ENGR. MUHAMMAD IQBAL QURESHI

Paper No. 650
Year 2000

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

By

ENGR. MUHAMMAD IQBAL QURESHI

The Romantic River Chenab of Punjab passes through two Hilly gorges near Chiniot City. One is called east Channel (Chiniot Side) and the other is named as West Channel (Sargodha Side). Maximum discharge for a hundred years flood cycle is estimated to be about one million cusecs to pass through the two gorges in a super flood.

The C&W Department, Govt of Punjab constructed a highway bridge on the downstream of and in close proximity to the Chiniot composite railway bridges on the two channels of the river to provide an independent transportation road crossing diverting the road traffic from the railway bridges to the new highway bridges.

The bridge during construction developed serious cracks in the piles and transoms of the piers and some piles had to be replaced while others were strengthened.

The author in this paper has presented in a concise way the process of design, construction and the problems in a candid way. The author has

also made recommendations which give a brief picture of the whole episode and suggestions which have been summarized in the following:

Conclusions and Recommendations by the Author

Selection of Site

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

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

a) Extensive land acquisition.
b) Long Approach Roads,
c) Long Training Works

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

a) the direction and the acceptance that the Bridges on West and East Channels be designed for discharges of 450,000 and 3,50,000 cusecs respectively was not logical as it did not take into consideration the changes in the bed of the river, water flow pattern and the construction of a spur, a mosque and Dargah on the left side and their effects.
b) There was absence of Expert’s advice or opinion about the effect of deterioration of the channels, formation of curved

embayment, rising flood levels, and oblique flows/currents which will continue to influence and tax the safe flood capacity of these bridges and scours around piers.

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

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

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

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

Design Discharge

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

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

Construction of foundation of Bridges in Rivers of Punjab is a seasonal work. The working season starts approximately from middle or 3rd week of September every year. Some times in winter in March, April due to wet cycle sub structure construction comes to a standstill. It is always not possible to provide super structure to stabilize the completed piles. So a free standing submerged pile has a tendency to flutter in the flowing water. The Designer/Consultant states that "when the natural frequency of a submerged pile coincides with the frequency of Vortex shedding behind the pile then large oscillations are induced which result in loosening of the pile in the bed of the River. This is a condition peculiar to a free standing pile. The completed Bridge does not allow this to happen because of restraints provided by the super structure. Piles at pier 3 were observed to move to and fro at right angle to the flow in the River in floods of 1996 and also during the subsequent floods".

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

There is a lesson to be learnt from above. The construction of piles in river beds be considered a stage construction. There should be a built in Risk Factor in the design for fluttering of free standing submerged piles during floods.

The length of replacement piles of West Channel Bridge has been increased by about 10% with enhanced steel reinforcement. This may be one of the solutions of the above mentioned problems as it appears that this change in design has been done by taking into account the

increased concentration of flow in effected spans. To emphasis this point the comments/observations of Pakistan Engineering Council in its Enquiry Report in this connection are reproduced below:

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

Cracks on the Faces of Transom No. 6

The Consultants/Designers have given a detailed explanation for cracks on the faces of the transom No. 6. However, the findings of Pakistan Engineering Council are quite different and are reproduced below:

"The large width of the cracks on the faces of the transom in the web is attributed to inadequate steel on the faces of the web as against code requirements. This is a design deficiency".
It is therefore for the Designers to take into consideration both explanations/reasons while designing the transoms over Piles.

Breaching Section (B.S):

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

Breaching Sections were useful and easy to operate in the past but now their operations are fraught with danger due to reasons mentioned

above. This is a problem, which needs serious consideration. B.S was provided to save cities and expensive structures on Rivers worth Billions of rupees. By operation of B.S Private Properties worth millions of rupees are endangered now. Whether public or private, they are all Pakistani Properties. They are our valuable assets. We should therefore consider ways and means to safeguard interest of all the parties involved in the matter.

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

Paper No. 651
Year 2000

PERFORMANCE OF SUBSURFACE DRAINS IN MIRPUR KHAS AREA OF LBOD STAGE-1 PROJECT

By

YAWAR HAMID AND
IRSHAD AHMED BOHIO

Paper No. 651
Year 2000

PERFORMANCE OF SUBSURFACE DRAINS IN MIRPUR KHAS AREA OF LBOD STAGE-1 PROJECT

By

YAWAR HAMID AND
IRSHAD AHMED BOHIO

Two tile drainage contracts i.e. T40.IB1 and T40.PC1 were implemented under LBOD Stage-1 Project in Mirpurkhas area during the period 1994-96. These contracts are in operation for the last 6 years. An attempt has been made to summarize the experiences gained from the design, construction; post construction monitoring results and views of the farmers for one of these contracts i.e. T40.IB1. Under this contract about 943,000 and 359,000 feet of lateral and collector drains, respectively, along with 16 pump stations were constructed at a cost of Rs. 243.29 million to drain an area of about 12,000 acres. The results of the study are:

(a) Better agricultural benefits were observed only at pump stations where the farmers take interest in the maintenance of the pump station. In most of the cases the farmers take interest in the maintenance.

(b) Lack of interest observed at some locations is due to conflicts between the land owners of a common pump station. If separate drainage systems and pump stations, for each land owner, are constructed then these conflicts will not appear in the maintenance of a pump station. This supports the idea of small drainage systems under private investment.

(c) A substantial percentage of abandoned land has been brought under cultivation. Abandoned lands not brought under cultivation may be due to shortage of irrigation water or lack of interest on the part of land owners.

(d) Due to the operation of the drainage system the farmers get

benefits in the form of increased crop yield per acre, increased cropped area due to reclamation of abandoned land and increase in the value of land.

(e) Average yield per acre for sugarcane has increased from a range of 10-28 to 32-40 tons, of cotton from a range of 200-600 to 800-1,600 kg and of wheat from a range of 400-1,200 to 1,200 to 1,600 kg.

(f) Average cost per acre of land within the area has increased from a range of Rs. 4,000-20,000 to Rs. 50,000-100,000.

(g) The major complaints of the farmers are a lack of proper O&M of project works, unreliable electric supply and shortage of irrigation water to reclaim abandoned lands.

(h) For sustainable irrigated agriculture, a reliable drainage system is required to maintain the lands in good condition. If the O&M of the drainage system is not carried out properly then sustainable irrigated agriculture is not possible. Before the installation of the drainage system about 50 % of area was out of production. With drainage, a substantial part of the abandoned land has been brought under cultivation and the reclamation process is in progress. However, the operation and maintenance of the project works is not of proper standard. If the O&M conditions remain as they are at present then the pumping equipment will wear out in a very short time resulting in a rise in water table forcing lands out of production.

(i) There was over drainage in two pump station catchment areas during the canal water shortage period. This phenomenon was observed in high elevation lands. Farmers of one of the catchment areas have blocked some of the sub-surface drains to check over drainage.

(j) Uncertainty prevails, in that after the expiry of the current O&M Performance Contract funded by the World Bank, the availability of funding for continued O&M is in question.

To lower water table below the root zone of the crops, drainage tube wells as well as tile drains were installed in Mirpurkhas Component of LBOD Stage-1 Project. Tile drains were laid only in those areas where tube wells could not be installed due to the non-availability of aquifer required for the tube wells. The area selected for tile drains is located in the South of railway line connecting Mirpurkhas and Umerkot towns

and between Mithrao and Jamrao canals. Total area selected for this purpose was about 60,000 acres

Tile drains installed in Mirpurkhas area fall under the command of Jamrao as well as Mithrao canals. A part of the area under Jamrao canal receives increased canal supplies due to canal remodelling while the entire area under Mithrao canal receives unchanged canal supply as the canal system is not remodelled. Areas drained by pump sumps SD-20, SD-23 and SD-25 receive supplies from Mithrao canal. However, the whole of the Jamrao canal command area, under Contract T40IB1, receives increased canal supplies. Due to different rates of canal water supply, the volume of excess sub-surface water to be removed through the drainage system i.e. drainable surplus for the two areas are also different and is computed on the basis of steady state flow conditions.

Un–plasticized, corrugated and perforated polyvinyl chloride (PVC) pipes have been used for the lateral as well as for the collector drains.

Natural gravel was used as an envelope material for the lateral drains. The designed grain size distribution of the gravel is:

- The maximum size shall be 12.5 mm;
- No more than 5 percent of the weight of a sample shall pass through 0.33 mm sieve;
- The D_{85} size shall be between 2.5 and 8.0 mm;
- The D_{60} size shall be between 1.0 and 3.0 mm;
- The D_{15} size shall be between 0.35 and 0.80 mm;
- The D_{10} size shall be between 0.30 and 0.70 mm;
- The coefficient of uniformity (CU) shall be greater than 4; and
- The coefficient of curvature (CC) shall be between 1 and 3.
  Where:
  $CC = D_{60} / D_{10}$;
  $CU = (D_{30} \times D_{30}) / (D_{10} \times D_{60})$

Synthetic geo-textile fabric, suitable for soils with D_{50} <= 0.075 mm, was used as an envelope material for the collector drains. All the

collector drains were perforated. The D_{95} of the synthetic envelope was less than 0.149 mm.

The laying of collector drains was started from the sump by installing corrugated blind PVC pipe sleeves in the sump as per the required number of collector lines coming towards the sump. The installation of the manhole was also carried out during the installation of collector drains. Installation of drains and backfilling and compaction were carried out simultaneously. The progress of collector drain laying was 3,000 to 5,000 feet per day. Drain markers were provided along the alignment of the collectors so that exact position of the drain is known during the post construction period.

Problems during the Construction Work:

a) Excessive dewatering was required, during sump construction, at places where the water table was shallow.
b) Drain laying was delayed due to the watering of fields by the farmers.
c) Obstructions were made by the farmers to avoid crop damage although crop compensation was paid by the Implementing Agency.
d) Work was delayed due to short supply of PVC pipes from the factory.
e) Break downs of machinery and delays in the supply of spare parts.

6. Comments and Recommendations

To increase the life of the project the following actions are of prime importance:

a) If the designed agricultural benefits are the objectives of the installed drainage system then electric supply should be reliable and continuous. Unreliable electric supply is one of the major complaints of the farmers.
b) Social mobilization is required so that the farmers take maximum interest in the operation and maintenance of the drainage system. Without farmer's interest, the project

objectives will not be achieved.

c) Additional canal supplies are required to reclaim the abandoned soils.

d) Monitoring scope needs to be extended. Monitoring should also include:
   - study of the behaviour of tile drains;
   - determination of actual drainable surplus;
   - determination of canal seepage interception rates along Mithrao canal;
   - in depth study at sumps where drainage inflow is lower than the design rate; and
   - economics of the drain installation.

e) Areas to be delineated where over drainage are being experienced and provide site specific solutions. One of the solutions may be the construction of manholes with sliding gates so that the water flow can be stopped to control the depth to water table. This problem, in future designs, can be avoided by splitting the catchment area in to two parts and providing independent pump station for high and low lying areas.

f) At some locations the pumping equipment is not suited to the incoming drainage effluent. At the time of pump replacement this factor should be taken into consideration.

6. Conclusions

The following conclusions can be drawn from the study:

a) Better agricultural benefits are observed only at pump stations where the farmers take interest in the maintenance of the pump station. In most of the cases the farmers take interest in such maintenance.

b) Lack of interest observed at some locations is due to the conflicts between the land owners of a common pump station. If separate drainage systems and pump stations, for each the land owner, are constructed then these conflicts will not appear in the maintenance of a pump station. This supports the idea of small drainage systems under private investment.

c) A substantial percentage of abandoned land has been brought under cultivation. Abandoned lands not brought under cultivation may be due to shortage of irrigation water or lack of interest on the part of land owners.

d) Due to the operation of the drainage system the farmers get benefits in the form of increased crop yield per acre, increased cropped area due to reclamation of abandoned land and increase in the cost of land.

e) Average yield per acre of sugarcane crop has increased from a range of 10-28 to 32-40 tons, of cotton crop from a range of 200-600 to 800-1,600 kg and of wheat crop from a range of 400-1,200 to 1,200-1,600 kg.

f) Average cost of land per acre has increased from a range of Rs. 4,000-20,000 to 50,000-100,000.

g) Major complaints of the farmers are lack of proper O&M of project works, unreliable electric supply and shortage of irrigation water to reclaim abandoned lands.

h) For sustainable irrigated agriculture, a reliable drainage system is required to maintain the lands in good condition. If the O&M of the drainage system is not carried out properly then sustainable irrigated agriculture is not possible. Before the installation of the drainage system about 50 % of area was out of production. With drainage, a substantial part of abandoned land has been brought under cultivation and the reclamation process is in progress. However, the operation and maintenance of the project works is not to a proper standard. If the O&M conditions remain as they are at present then the pumping equipment will wear out in a very short time resulting in a rise of water table and forcing lands out of production.

i) There was over drainage in two pump station catchment areas during the canal water shortage period. This phenomenon was observed in high elevation lands. Farmers of one of the catchment area have blocked some of the sub-surface drains to check over drainage.

j) Uncertainty prevails, in that after the expiry of the current O&M Performance Contract funded by the World Bank, the availability of funding for continued O&M is in question. One solution to the problem is that the farmers may be motivated to take over the O&M of the project.

Paper No. 656
Year 2000

USING ENVIRONMENT FRIENDLY FINELY DIVIDED MATERIALS IN BRITTLE MATRIX COMPOSITES

By

SYED ALI RIZWAN & HUSNAIN AHMAD

Paper No. 656
Year 2000

USING ENVIRONMENT FRIENDLY FINELY DIVIDED MATERIALS IN BRITTLE MATRIX COMPOSITES

By

SYED ALI RIZWAN & HUSNAIN AHMAD

Material engineers the world over are increasingly recommending the use of environment friendly efficient construction materials, which otherwise would have been classed as waste materials for improving the durability of concrete.

It is well known that modification of construction materials always aims at some predetermined and carefully selected objective/(s). The choice of admixtures to be used also depends upon the objective/(s) of modification which may either be for strength, architectural finishes, concrete conveyance, placing, permeability, durability or a combination of these in addition to their availability.

As reported by ACI (10) titled "Guide for use of Admixtures in/ Concrete", finely divided mineral admixtures include hydrated powdered lime, fly-ash ground quartz, ground limestone, bentonite and talc. If concrete aggregates are deficient in fine particle sizes particularly those passing BS sieve 100 and 200, the use of finely divided mineral admixtures can reduce bleeding and segregation and increase in strength while reverse may also be true if these are used for concrete aggregate not deficient in fine aggregates in said range suggesting that its use should be made under supervision of a good material engineer.

Lime and fly-ash as independent admixtures and their combination has

been used for modifying the concrete properties and a long term research has been conducted by the researchers to exploit the properties of locally available lime and fly-ash.

Powdered hydrated lime of type S is basically used in mortars and concrete to reduce the permeability of concrete by filling the pores in concrete. It improves cohesion and achieves economy through cement replacements. It can also be used in hot weather concreting. In addition to mineral admixtures, synthetic polymers can also be used to improve durability and other properties of concrete. The authors have also carried out research on this type of modified concrete.

Fly ash is a byproduct of burning finely divided coal in electrically generating power plants.

When used in concrete, fly ash acts like cement, and actually replaces a percentage of the Portland cement used. Fly ash generally replaces around 15% of cement in much of the concrete used today, but we can do much better by using it to replace up to 50% or more to be known as High Volume Fly Ash Concrete (HVFAC).

Modification with Fly ash usually results in cement savings, increased workability, increased strength after 56 days, increased cohesion of mix, pore refinement, increased frost and chemical resistance and reduced permeability of concrete etc, which makes a high performance concrete. The above desirable properties are achieved because of its spherical shape and fineness. About 40% of fly-ash particles are under 10 microns while size of cement particle is 20 microns.

This eliminates the micro cracking and creates concrete that is much less permeable, and therefore more durable. Also, a reinforced cement concrete modified with fly-ash will be less prone to corrosion as it decreases the ingress of water to a greater extent which actually causes the corrosion of rebars in reinforced concrete.

Lime imparts a high water retentivity and reduces size of voids by accommodating itself in them. Some researchers have stated that bond strength also increases by using hydrated lime in concrete. Hydrated

lime slightly decreases plasticity and workability of concrete. It imparts ease of re-tempering, high water retentivity, resistance against efflorescence, high sand carrying capacity and more flexibility under stress, more bond strength and autogenous healing to the mortars. Also lighter and colored mortars can be made by using hydrated lime along with a suitable pigment.

Uses of lime include soil modification and stabilization, especially in pavements, environmentally friendly construction, papermaking, production of chemicals (sodium alkalis, calcium carbide, calcium hypochlorite, citric acid, petro-chemicals, refractory, sugar refining, glass making, softening of drinking water, sewage treatment, agricultural fertilizers, fungicidal and insecticidal action, steel fluxing, bleaches, separation of cream from whole milk and in handling chicken litter etc.

Fly ash can compensate for fines not found in some sands and, thereby, enhance pumpability and concrete finishing. The workability of fly ash concrete generally ensures that the speed of construction is faster which translates into a quicker return on investment.

In order to study the parameters mentioned above for both normal (control), lime modified, fly-ash modified and combination of both lime and fly-ash concretes specimens were cast in both rich and lean concrete mixes i.e 1:1.5:3, 1:2:4and 1:2.5:5 concrete mix proportions by weight having net water-cement ratio (W/C) of 0.6 at a room temperature of 34 ^{0}C and relative humidity of 55 %.

Concretes were modified with fly-ash and specimens were cast using fly-ash equal to 10% of weight of cement. Mixes with a combination of fly-ash and lime (cocktail) were also cast (5 % lime and 5 % fly-ash) with a total of 10% as addition-not replacement) of weight of cement.
Acid resistance test was performed on 2”x2”x2”(50x50x50mm) specimens cut from original 4”x4”x4”(100x100x100mm) cubes against N/2 solutions of HCl, HNO_3 and H_2SO_4. The procedure for making N/2 acid solutions is as follows.

1. Find the density of acid by using specific gravity flask/pycnometer.
2. Determine the percentage purity of acid from standard tables against density values either directly or by interpolation.
3. Find equivalent weight of acid i.e. by using formula,

The specimens were immersed in acid solution, an immediate reaction took place and after one week, the solution became weaker and was changed. This was repeated four times during 28 days cycle and pH changes were monitored during this cycle by means of pH sticks or by using pH meter. After 28 days weight loss indicated degree of vulnerability or indirect durability as per recommendations of international literature

From the behavior of fly-ash it appeared to be of class F accor4ding to ASTM classification because for class C fly-ash Bulk density is usually low. Increase of workability due to addition of lime may be attributed to its fineness and no water demand compared with fly-ash modified concrete in which workability reduces due to water demand of fly-ash. Addition of lime and fly-ash increases the strength of lime and fly-ash modified concretes. This may be understood to be the fact due to "packing effect" of the voids in the products of hydration. Lime and fly-ash modified concretes present variable results for the both mixes against three acids and no well defined conclusion may be made. Against HCL and H_2SO_4 control concrete gives more weight loss while for HNO_3 control concrete gives least weight loss.

1. Use of fly-ash and lime in concrete results in economical-high performance concrete.
2. The results are only for the materials and mix proportions used. For other mix proportions, same results may not be applicable.
3. Compressive strength of concrete increase with an increase in %age of this type of fly-ash and lime as an addition of some percentage of cement in concrete at early ages
4. Permeability, durability and cohesion of concrete in general are improved with the addition of mineral admixtures studied.

5. Workability of concrete is increased when fly ash is added to it but situation is reverse with lime.
6. For better results, it is recommended that lime and fly-ash should be batched on volume basis for mixing in concrete. If taken as a weight percent of cement, lesser percentages may give still better results.
7. From acid resistance tests on cut specimens, it became clear that acids attack severely the coarse aggregates (lime stone). The attack on the paste was that serious for N/2 normality.

Paper No. 662
Year 2004

DAMAGES TO THE RIGHT POCKET OF SUKKUR BARRAGE AND EMERGENCY RESTORATION WORKS (2004-2005)

By

BARKAT ALI LUNA[1], MALIK AHMAD KHAN[2],
CH. MUZAFFAR HUSSAIN[3],
&
DR. MUHAMMAD SALIK JAVED[4],

Paper No. 662
Year 2004

DAMAGES TO THE RIGHT POCKET OF SUKKUR BARRAGE AND EMERGENCY RESTORATION WORKS (2004-2005)

By

BARKAT ALI LUNA[1], MALIK AHMAD KHAN[2],
CH. MUZAFFAR HUSSAIN[3],
&
DR. MUHAMMAD SALIK JAVED[4],

1. Sukkur Barrage and Canals Project were sanctioned on 9th June 1923. The construction work was started in July 1923 and on completion, the canals were opened on 13th January 1932. The Project was completed at a total cost of about Rs. 200 million constituting one of the World's largest single unified irrigation systems.

2. Seventy years after its commissioning, a scour pit was discovered in the u/s stone apron in the right pocket of the Barrage in 2002 during the annual closure. No remedial measures were taken and the damages were allowed to grow un-checked till the closure of 2004. The soundings/probing observed in the area on January 12, 2004 revealed that the

[1] Chairman National Development Consultants (Regd)
[2] Project Manager, Sukkur Barrage Rehabilitation Project
[3] Dy. Project Manager, Sukkur Barrage Rehabilitation Project
[4] Engineer-in-Chief Branch, GHQ Rawalpindi

scour pit had grown to an alarming size measuring 60'x40'x9'(Av) causing collapse of u/s stone apron, first line of sheet piles and a part of the concrete floor u/s Piers 1 & 2. Moreover, cavities had developed under the floor below the crest of the under sluices. Like-wise cavity had been formed under the floor in front of Dadu Canal Head Regulator. The safety of the Barrage on the whole was at stake. The Government of Sindh took immediate notice of the catastrophic situation and constituted a Technical Advisory Committee of Senior Engineers who investigated the situation and advised long term and short term remedial measures for the safety of the Barrage. Accordingly, Irrigation and Power Department framed a Project to carry out the emergency repairs of the damages discovered so as to restore the normal functioning of the right under sluices and save the system from total failure. The execution of the project was assigned to the Frontier Works Organization (FWO) under a special order of the President of Pakistan on the request of Government of Sindh. The Joint Venture of M/s. National Development Consultants (Regd.) – NDC and M/s. Engineering Associates (Pvt) Ltd, Karachi (EA) in association with M/s Atkins of United Kingdom provided the Consulting Services.

3. Following works were undertaken to repair the damages and restore normal functioning of the Barrage:-

i. Mobilization of man force, equipment and material
ii. Construction of coffer dam upstream and downstream of the Right Undersluices
iii. Construction of Link Canal between Rice Canal and Dadu Canal
iv. Dewatering of the working area
v. Removal of sediment deposits/slush from the working area enclosed by the coffer dams
vi. Detailed physical checking of the actual scour caused in the pit and extent of cavities formed under the main weir floor and the upstream floor of Dadu Canal Head Regulator
vii. Geophysical investigation in the Right Pocket
viii. Sheet piling at the extended upstream and downstream ends of Main Weir concrete floor
ix. Replacement of old concrete blocks and underneath stone pitching downstream of the concrete floor by properly designed inverted filter, overlaid by concrete blocks measuring

4'x4'x4'

x. Provision of settling of cavities under the floor by cement sand mix of proper consistency

xi. Pressure grouting of cavities under the floor by cement sand mix of proper consistency

xii. Installation of Piezometers in the Main Weir and in the floor of Dadu Canal Head Regulator

xiii. Concreting for extension of upstream floor

xiv. Replenishing deficient stone aprons on upstream and downstream of the Right Pocket

xv. Removal of upstream and downstream coffer dams.

During execution of the works serious challenges and critical situations were confronted which were tackled with technical skill of the Consultants and the contractor. The experience gained can be usefully utilized under similar situations for rehabilitation of other barrages in the country.