Paper No. 315-B

Studies in some Hydraulic Features of the Design of Taunsa Barrage

PART II—ESTIMATION OF MAXIMUM AND MINIMUM DISCHARGES & LEVELS IN INDUS AT TAUNSA BARRAGE

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The maximum discharge is a basic figure in the design of any headworks, On the correct estimation of this figure depends the safety and the economy of the work. If we estimate too low we will have to encounter more than design ; floods vary often, every time endangering the work, and if we over-estimate the cost of the work goes up accordingly, The correct assessment of maximum and minimum levels and their limits of fluctuation due to retogression and accretion cycles are another important data on which the correct design to a large extent depends. Setting of the downstream floor level without keeping sufficient provision for retrogression ever likely to occur will create conditions of inadequate flow depth for the formation of Hydraulic Jump, resulting in shooting or undulatory flow, which may cause the failure of the work. The failure of Islam Headworks was to a great extent due to this cause, and ultimately a subsidiary weir had to be constructed to save the original one.

Estimation of maximum flood discharge

Unfortunately it is not possible to estimate the magnitude of the maximum possible flood on a river to any great degree of exactness. A river flood owes its magnitude to a great variety of random and systematic factors. Some authorities have given formulae for maximum flood discharge but as Greager[1] has pointed out most of them are empiricial to the extent that they take into consideration only a few factors affecting the run-off. Hayford[2] assigns as many as 23 factors affecting the flood

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run off. There are cycles of floods of varying magnitudes and if we have record for a short period then it is possible that it may correspond to a low or high cycle and therefore may not represent the true conditions.

In estimating the maximum flood discharge from the existing record it must be remembered that what has once occured can not only be equalled but can also be exceeded in future. More-over extreme floods of rare occurence can succeed each other at short intervals.

Cases have been noted in U. S. A. which reveal interesting information. In 1890 the greatest flood known in a certain area was 200 sec-ft per square mile and it was 1900 sec-ft in 1939 *i.e.* 8·5 times as great. Again in 1935 the Republican river in Nebraska had a flood ten times as large as had ever occured there during the previous 39 years.

The following methods were used to get at a probable estimate of maximum flood at Taunsa.

(1) Empirical Formulae.

(2) Probability Method.

(3) Estimation based on record of floods at Kalabagh and Ghazighat.

1. Maximum discharge from the catchment formulae.

A large number of empirical formulae have been given by different authors for the determination of the maximum floods *e.g.* Dickens, Ryves, Inglis, Fanning, Fuller, Murphy, etc. Some of these involve only catchment area and some involve a few catchment characteristics in addition to the area. These formulae involve some constants which are based on the observations of the respective authors for the particular areas. The use of M — the drainage area alone in the formula is due to the supposition that extreme floods are due to rainfall sufficient in duration, extent and intensity to saturate the entire catchment area. The slope of the catchment its shape, length and other characteristics have comparatively less effect. From some of the formulæ the maximum discharge for Taunsa has been calculated and recorded in Table I. It will be noted that for the same value of the drainage area we get values of the discharge from the various formulae which differ from each other to a very great extent. Floods are the result of rainfall and actually the rainfall data should get prominence in the estimation of the maximum flood. If the drainage areas of two streams are equal but they differ greatly in their rainfall and other characteristics, any formula if applied to two catchment areas will give almost the same value of the maximum discharge for the two streams. In fact the simultaneous existance of all the metrological and hydrological factors responsible for heavy rainfall and run off contributes to make a record flood. On the probability of as many more factors occuring simulatneously as possible will depend the magnitude of the flood. Therefore in the presence of 23 or more contributory factors it is too much to expect from a single constant in any formula to

cover the effect of all the physical characteristics. These formulae very roughly *give only indications and not a reliable result* on which a design can safely be based without complimentary studies.

Karpov and Kanwar Sain prepared an envelope curve between drainage area in square miles and maximum discharge in cusecs for Northern and Central India rivers shown in Fig. I. From this curve for the drainage area of Indus at Taunsa equal to 136934 square miles, the maximum discharge will be about 22,00,000 cusecs.

2 The probability method.

There are two probability methods viz. (a) Basic stage and (b) Yearly flood method. The former involves the consideration of all flood peaks greater than given basic stage flood, generally the lowest maximum flood in any year of the period used in the study and the latter involves the analysis of only the maximum yearly floods. The results derived by both the methods do not differ materially.

It has to be borne in mind that the results of analysis by probability methods depend on the data put in. If the data were for a considerably more period or say a little less than what has been used, the result might have been different. This is again due to the fact that the analysis is based on data alone which may or may not include all the physical factors which contribute to make a maximum flood. Fully realizing the inadequacy of this method it may still be preferable to the method of application of a factor of safety to the maximum recorded flood.

The probability studies of the river Indus were made for Kalabagh and Ghazighat sites by the yearly flood method. The annaul maximum floods were arranged in the ascending order of magnitude and the number of times n, that each flood was equalled or exceeded was noted. According to the law of probabilities the percentage of years in which a flood equal to or greater than a given discharge Q will occur is obtained from the equation.

$$p = \frac{100\ n}{N}$$

Where p = percentage of future years in which a flood equal or greater than a discharge, Q, will occur.

N = the total number of years of record. The peak discharges are plotted against the values of p on probability paper and a curve through these points is drawn.

$I = \frac{100}{p}$ gives the interval in years, between floods equalling or exceeding a given flood. The coefficient of variation CV is defined as

$$CV = \sqrt{\frac{\Sigma V^2}{N-1}}$$

where V denotes variation of the discharges from unity when the discharges are expressed in terms of their arithmetic mean.

For Kalabagh site the data was available for the years 1922 to 1946 and for Ghazighat the data was analysed for periods 1923 to 1940 and 1950 and 1952. The data and the results are given in Tables II and III. The probability curves for the two sites are drawn in figure 2. From these curves we find that a flood equal to or greater than 20, 13 and 11 lakhs can occur once in a thousand, hundred and 50 years respectively at Ghazighat. It is admitted that the periods are too short to make the analysis as trust worthy as one would have liked to be, but this was all we could have. The paucity of data at such times reminds us of the need of collecting authentic data for all our rivers. Design and Research workers who feel the lack of required data at such occasions cannot but emphasize, this point too strongly.

3. Estimation based on records of flood at Kalabagh and Ghazighat.

The maximum flood recorded at Kalabagh according to the records of Discharge Division is 8,19,000 causecs on 29-8-29, while Croock Shank of North Western Railway[3] placed it in the neighbour-hood of 12,00,000 causecs. Data given by Kanwar-Sain in his project report records a maximum discharge of 10,00,000 ocusecs on 29-8-29 and 9,17,015 cusecs on 15-7-43. These are divergent estimates and in this analysis we have given more weight to the authentic record of Discharge Division. Assuming a maximum flood discharge of 10,00,000 cusecs at Kalabagh, Kanwar Sain assumed 20% absorption loss between Kalabagh and Sanghar giving a maximum discharge of 8,00,000 cusecs. Computing the maximum discharge of Sanghar as 1,00,000 cusecs and allowing for synchronising of bursting of Shayak dam with flood in Indus and Sanghar he estimated 10,00,000 cusecs as the maximum discharge at the site.

The figure of 20 percent absorption is only an assumption. It was considered desirable to try to work out some relationship between the maximum annual Kalabagh discharge and the corresponding Ghazighat discharge. A Logarithmic plot of Kalabagh and Ghazighat peak discharges exhibited a linear trend and a straight line was fitted by the method of least squares. It comes out as :—

$\log y = .267 + .869 \log x$, where x and y denote Kalabagh and Ghazighat discharges in thousand cusecs.

The above equation can be written as

$$\log y = \log (1{\cdot}8493) + \log x^{\cdot 869}$$
$$= \log 1{\cdot}8493\, x^{\cdot 869}$$
$$\text{or } y = 1.8493\, x^{\cdot 868}$$

This is an approximate relationship and holds for the range of data analysed.

The average net decrease in discharge per mile was determined from the above formula and the discharge at Taunsa Barrage site was computed. The values so obtained are compared with the values for the above assumption of 20% in columns 4 and 5 of table IV.

Discussion of results and estimation of maximum discharge.

For a maximum discharge of 9,17,000 cusecs at Kalabagh, the discharge at barrage site is 7,61,000 according to the above analysis. Even assuming the Railway figure of 12 lacs at Kalabagh the discharge at barrage site from this formula is 8,76,605 cusecs. If the maximum yield from the Sulaiman range is taken as 1 lac cusecs and it is assumed to synchronise with the Indus flood then we expect a discharge of 8,60,000 cusecs at the barrage site. Allowing a further margin of 1,40,000 cusecs for the bursting of Shayak dam and other contingencies an estimate of maximum discharge of 10 lacs at the barrage site is the probable figure from this analysis. From catchment formula the discharge estimation varies from 500,000 to 40,00,000 cusecs. It will be noted that value obtained from the catchment formula of Kh. Azim-ud-Din, which is applicable to tropical countries and is more rational in that x in that formulae $Q = CM^x$ is taken as a function of M—the catchment area, is not a very unreasonable figure. The envelope curve between drainage area and maximum discharge fixes the value near 22,00,000 cusecs. From the probability studies floods equal to or greater than 20, 13 & 11 lacs can occur once in a thousand, hundred and 50 years respectively.

It is emphasized that it does not mean that discharges above 11 to 12 lacs will not occur. All that it means is that they will be very rare. It is to be noted also that even if provision is made at the headworks for such abnormal floods, due to the very fact that water-way adjusts itself only to normal conditions a sudden on-rush of an abnormal flood will still find the water-way inadequate. Even a super-flood of 1955 in Ravi has not been able to wash away the island on the right side within the guide banks of Shahdara Railway Bridge. It is therefore reasonable to keep some flexibility and margin in the design to pass discharges higher than the maximum design discharge. In view for the uncertainty of estimation a better procedure for the design will be to estimate values for not only the maximum designed discharge, Q_{pmax}, but also of the maximum permissible discharge, $Q_{pmax.}$ and unsafe discharges. All discharges greater than $Q_{pmax.}$ will infact be unsafe for the weir and additional measures, such as breaching the marginal bunds etc. should be planned right in the begining and should be reviewed off and on depending on the river course upstream. It is better to keep this point in view in the original design, in the lay out of the marginal bunds and location of diversion regulators and training works. The design of the guide banks, marginal bunds and training works as far as the free board and hydraulic gradient are concerned should be made safe for levels of $Q_{pmax.}$ The level of the floor and its lenght may be designed for Q_{dmax} but should be made safe for Q_{pmax} by adopting such level for retrogression that $Q_{pmax.}$ can be passed without standing wave being repelled down the toe of the glacis. The extra provision in the length of the floor may be made in the form of blocks and extra rip rap. The design should always be tested for its performance at low flood, medium flood, maximum flood and maximum probable flood. The performance of Taunsa weir for these discharges from the theoretical and experimental studies will be discussed more fully in the next part of this series of papers.

From this analysis, the reasonable figure for Q_{dmax} is 10,00,000 cusecs, and Q_{pmax} for the present may be kept equal to 1.25 Q_{dmax} till more rational basis of fixation depending on some hydrological or metrological characteristics of the catchment can be found. It is then equal to about 12,50,000 cusecs. Any value above Q_{pmax} should be considered unsafe for the works.

Estimation of Minimum and Maximum Water Levels at Taunsa.

It is a well known fact that in alluvial channels the same discharge can pass at different levels at various times. This wide variation in river stage for different discharges is sometimes due to silt bars and dunes travelling down river, avulsions upstream and downstream of the gauge site, extra super-floods in particular years, etc., etc. There are cycles of accretion and retrogression but these are not regular in frequency and form. It is very essential, therefore, to correctly asses the minimum retrogressed and accreted levels likely to occur at the barrage site. Apart from normal accretion and retrogression in a river away from the control point, there is a special upsetting of river regime after construction of a barrage, which in the initial stages after the barrage construction causes an extra retrogression downstream of barrage and recovery at a later stage through cycles of accretion and retrogression as shown by the author while studying the causes of retrogression below Islam Barrage. The retrogression after constructing the barrage is due to selective withdrawls of silt by off-taking channels, decrease in discharge downstream required steeper slope. The maximum retrogression, within 8 years after the construction of some of the weirs is tabulated below :

Station	Q in cusecs	Maximum retrogression in feet
Sukkur Barrage ...	1,00,000	5·0
	4,00,000	2·3
Islam Barrage ...	5,000	6·4
	25,000	6·6
Sulemanki Barrage ...	5,000	8·2
	1,00,000	2·9
Ferozepur Barrage ...	5,000	6·4
	2,00,000	1·6
Trimmu Barrage ...	5,000	4·7
	1,00,000	2·6

It is therefore very essential to provide for maximum retrogression likely to occur below the weir.

A method generally adopted in the past is to draw the discharge gauge curve from the existing or observed data, and assuming retrogression arbitrarily at low and high stages, to draw the discharge gauge curve for the retrogressed conditions. Kanwar Sen in project report prepared a discharge rating curve for the observed data of 1946 and assumed a retrogression of 4 feet at low discharges and one foot at the maximum discharge of 10 lacs.

A rational procedure for this would be to work out the maximum and minimum accreted and retrogressed levels from the gauge discharge observations at the proposed barrage site or any other site nearest to it. The method adopted for estimation of these levels for Taunsa are discussed below in some detail. The method was initiated by Mr. Foy[5] in the design of Kotri Barrage. The analysis is made with certain modification for Taunsa on similar lines. No discharge gauge site was available at the proposed Taunsa Barrage site. The nearest discharge gauge site was at Ghazighat about 37 miles below the proposed barrage site. Authentic discharge gauge observations were available for a period of 9 years at this site. It is admitted that the period is small but to make the best use of this data the following procedure was adopted for the analysis.

(1) Log of discharge was plotted against gauge R. L. for each year from 1930 to 1939 separately on graph paper on semi log scale. Envelopes demarcating the maximum and minimum limits of scatter of the gauges were drawn for each year. A typical diagram exhibiting these envelopes for 1939 is shown in fig. 3. It will be noted that envelopes are diverging for low discharges and converging for high discharges.

(2) From the above semi log scale there are two possible ways of framing the maximum, minimum and retrogressed discharge rating curves for the barrage site.

(a) For selected specific discharges read the values of minimum and maximum on the envelopes and work out the mean of maximum and minimum separately for each year. Draw the mean maximum and minimum gauge discharge curves for the site; some extra-polation on the lower and upper side of semi log scale curves has to be done for covering the entire range of discharges of the rating curve. To draw the rating curve for retrogressed conditions it was decided to assume the same order of retrogression at each discharge stage as had already been noticed between the mean maximum and minimum rating curves.

(b) Yet an other method proposed by Ch. Muhammad Nazir[6] was to select the maximum of maximum and minimum of minimum on the envelope curves of different years and to prepare maximum and minimum discharge rating curves. The retrogressed rating curve was plotted by him by assuming 5 feet retrogression at the low stage and 2·5′ retro-

gression at the stage for Q=9,00,000 cusecs with Nil retrogression at 10 lacs and higher stages.

To convert the discharge gauge rating curve at Ghazighat to Taunsa weir site, which is 37 miles upstream of Ghazighat, the slope of 1·14' per mile was assumed in the river in this particular reach as worked out by Kanwar Sain in high medium floods. It is obvious that this assumption about slope is not absolutely correct as according to Lacey's formulae for regime channels the slope is inversely proportional to $Q^{1/6}$. But as alluvial river never approaches stable equilibrium conditions the percentage error with this assumption of slope will not be much. Ch. Muhammad Nazir assumed a slope of 1·14′ for discharges greater than 1 lac cusecs and 1 foot per mile for discharges below 1 lac cusecs. The discharge curves prepared by author, Ch. Muhammad Nazir and Kanwar Sen are plotted along with retrogressed rating curve in fig. 4. Comparison of values for specific discharges in three cases is made in table V. It is admitted that both the curves of the author as well as of Ch. Muhammad Nazir are based on certain assumptions and that of *Kanwar Sen is based only on one year's data*, not taking into consideration the variations in the field data. From the rating curves of Ch. Muhammad Nazir it will be seen that for discharges between 6,00,000 to 9,00,000 cusecs there is as much as 6 feet difference in levels between maximum and retrogressed rating curves as compared to 9 feet difference at 10,000 cusecs stage, whereas by the author's method the difference of levels between accreted and retrogressed levels is equivalent to about 2·4 feet at a discharge of 10 lacs as compared to about 9 feet at the low stage, which appears to be more in conformity with the general trend of envelope curves in different years where the difference in accreted and retrogressed stages is very much less at high flood stage than it is at the low stage. The arbitrary assumptions for arriving at the discharge gauge curves are still open to further improvements but for the present the method adopted for analysis is a rational way of getting at the discharge rating curve for testing the performance of weirs. The actual performance of the weir, theoretically as well as experimentally, for these rating curves will be discussed in the next part of this series of papers.

Acknowledgment.

This part of the paper is a result of investigations made on the initiation from Mr. A. D. Ashraf (Additional Chief Engineer Construction). The help and guidance of Chaudhri Muhammad Nazir, the then Director is gratefully acknowledged. At a later stage he fixed a new criteria for getting maximum and minimum levels. This is also particularly acknowledged and discussed in the paper itself. Frequent exchange of ideas with Mr. Mohy-ud-Din Khan Director Central Design (Taunsa) and his close cooperation during the studies are also gratefully acknowledged.

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TABLE I

S. No.	Authority	Formula	Maximum discharge in cusecs
1	Fanning ...	$Q=200\ M^{5/6}$	38,14,600
2	Dickens ...	$Q=500\ M^{3/4}$	35,59,000
		$Q=600\ M^{3/4}$	42,70,800
3	Ganguillet ...	$Q=\frac{1421\ M}{3{\cdot}11+\sqrt{M}}$	5,21,517
4	Italian ...	$Q=\frac{1819\ M}{0{\cdot}311+\sqrt{M}}$	6,72,632
5	Kuichling ...	$Q=\left(\frac{44000}{M+170}+20\right)\times M$	27,82,636
6	Kuichling ...	$Q=\left(\frac{127000}{M+370}+7{\cdot}4\right)\times M$	11,39,976
7	Murphy & others	$Q=\left(\frac{46790}{M+320}+15\right)M$	21,00,704
8	Kh. Azim-ud-Din	$Q=2100\ M^{0{\cdot}92-1/14\ \log M}$	14,56,400
9	Inglis ...	$Q=\frac{7000\ M}{\sqrt{M+4}}$	25,90,293
10	Bremner ...	$Q=\frac{3000\ M}{3+2\times\sqrt{M}}$	5,52,896
11	Harman ...	$\frac{30000\ M}{M+200}+15M$	20,83,966
12	Metcalf and Edy	$Q=440\ M^{0{\cdot}73}$	24,72,210

TABLE No. II

Probability studies of the river Indus at Kalabagh.

Year	Q Max. discharge in thousands of cusecs	Col. 2 in order of magnitude	No. of times n Peaks was equalled or exceeded	P = precentage of years	$I = \frac{100}{P}$ intervals between floods	Col (3) in terms of mean flood	Variation from mean, V	V^2
1	2	3	4	5	6	7	8	9
1922	505	339	25	100	1.00	.633	−0.367	.135
1923	550	357	24	96	1.04	.666	−0.334	.112
1924	520	390	23	92	1.09	.728	−0.272	.074
1925	459	432	22	88	1.14	.806	−0.194	.038
1926	390	459	21	84	1.19	.857	−0.143	.020
1927	432	460	20	80	1.25	.859	−0.141	.020
1928	357	468	19	76	1.32	.873	−0.127	.016
1929	819	473	18	72	1.39	.883	−0.117	.014
1930	484	484	17	68	1.47	.903	−0.097	.009
1931	339	491	16	64	1.56	.916	−0.087	.007

1932	570	502	15	60	1.67	.937	−0.063	.004
1933	520	505	14	56	1.79	.943	−0.057	.003
1934	591	511	13	52	1.92	.954	−0.046	.002
1935	535	520	12	48	2.08	.971	−0.029	.001
1936	460	520	12	48	2.08	.971	−0.029	.001
1937	473	535	10	40	2.50	.999	−0.001	.000
1938	502	540	9	36	2.78	1.008	0.008	.000
1939	540	550	8	32	3.13	1.027	0.027	.001
1940	491	562	7	28	3.57	1.049	0.049	.002
1941	468	570	6	24	4.17	1.064	0.064	.004
1942	917	591	5	20	5.00	1.103	0.103	.011
1943	562	691	4	16	6.25	1.290	0.290	.084
1944	691	708	3	12	8.33	1.321	0.321	.103
1945	708	819	2	8	12.50	1.529	0.529	.280
1946	511	917	1	4	25.00	1.711	0.711	.506
Total =		13394						1.447
Mean =		535.8						

$$CV = \sqrt{\frac{\Sigma \overline{V^2}}{24}} = \sqrt{.060291} = .245$$

TABLE No. III

Probability studies of the river Indus at Ghazighat.

Year	Q Max. discharge in thousands of cusecs	Col. 2 in order of magnitude	No. of times Peaks was equalled or exceeded	P = percentage of years	$I=\frac{100}{P}$	Col (3) in terms of mean flood	Variation from mean, V	V^2
1	2	3	4	5	6	7	8	9
1923—24	512	194	19	100	1.00	0.424	−0.576	0.332
1924—25	525	349	18	94.7	1.06	0.763	−0.237	0.056
1925—26	194	350	17	89.5	1.12	0.765	−0.235	0.055
1926—27	425	358	16	84.2	1.19	0.783	−0.217	0.047
1927—28	350	365	15	78.9	1.27	0.798	−0.202	0.041
1928—29	358	366	14	73.7	1.36	0.800	−0.200	0.040
1929—30	700	385	13	68.4	1.46	0.842	−0.158	0.025
1930—31	460	425	12	63.2	1.58	0.929	−0.071	0.005
1931—32	349	427	11	57.9	1.73	0.934	−0.066	0.004
1932—33	447	442	10	52.6	1.90	0.966	−0.034	0.001

1933—34	385	446	9	47.4	2.11	0.975	−0.025	0.001
1934—35	614	447	8	42.1	2.38	0.977	−0.023	0.001
1935—36	442	460	7	36.8	2.72	1.006	0.006	0.000
1936—37	365	498	6	31.6	3.16	1.089	0.089	0.008
1937—38	366	512	5	26.3	3.80	1.119	0.119	0.014
1938—39	498	525	4	21.1	4.74	1.148	0.148	0.022
1939—40	427	614	3	15.8	6.33	1.342	0.342	0.177
1950—51	828	700	2	10.5	9.52	1.530	0.530	0.281
1951—52	446	828	1	5.3	18.87	1.810	0.810	0.656
	Total =	8690						1.706
	Mean =	457.4						

$$CV = \sqrt{\frac{\Sigma V^2}{18}} = \sqrt{.094777} \doteq .307$$

TABLE IV

Year	Date of maximum Kalabagh flood	Maximum flood discharge cusecs	Maximum Calculated Discharge at Sanghar (Barrage site)	
			20 per cent absorption	By formula
1	2	3	4	5
1922	25-6-22	505324	400000	429000
1923	23-8-23	550000	440000	466000
1924	29-7-24	520000	415000	442000
1925	1-8-25	458959	367000	392000
1926	15-8-26	390000	312000	337000
1927	23-7-27	432350	346000	370000
1928	23-7-28	357077	286000	311000
1929	29-8-29	819000	655000	682000
1930	16-7-30	484145	387000	412000
1931	21-7-31	339070	271000	296000
1932	1-8-32	569573	456000	481000
1933	20-8-33	520212	416000	442000
1934	8-7-34	590863	472000	498000

1935	10-7-35	535475	428000	453000
1936	22-6-36	460272	368000	393000
1937	2-7-37	473190	379000	404000
1938	27-7-38	502152	402000	427000
1939	17-7-39	540000	432000	457000
1940	2-7-40	491000	393000	418000
1941	4-7-41	467784	374000	400000
1942	12-7-42	917015	737000	761000
1943	15-7-43	562000	450000	475080
1944	6-8-44	691181	553000	579000
1945	25-7-45	708000	566000	592000
1946	3-7-46	510619	408500	434000
1947	10-8-47	427999	342000	367000
1948	7-8-48	772000	618000	644000
1949	11-7-49	747500	598000	625000
1950	2-8-50	644701	516000	542000
1951	11-8-51	555144	444000	470000
1952	5-8-52	400000	320000	345000

TABLE

Kanwar Sain Method			Ch. Nazir's Method	
Discharge	Observed R.L.	Retrogressed R.L.	Maximum R.L.	Minimum R.L.
10,000	428·3	424·5	428·84	423·27
20,000	431·1	427·5	430 87	425·74
40,000	432·7	428·8	433·47	428·71
60,000	433·2	429·6	435·45	430 97
80,000	433·75	430·15	437·09	432·77
1,00,000	434·3	430·8	438·64	434·16
2,00,000	436·3	433·2	440·44	436 80
3,00,000	438 0	435·1	441·5	438·18
4,00,000	439·6	436·7	442·36	439·08
5,00,000	440·8	438·2	443·28	439·72
6,00,000	441·75	439·5	443·82	440·24
7,00,000	442·45	440·5	444·18	440 70
8,00,000	443·0	441·4	444·68	441·03
9,00,000	443·65	442·4	445·08	441·36
10,00,000	444·0	443·0	445·48	441·68

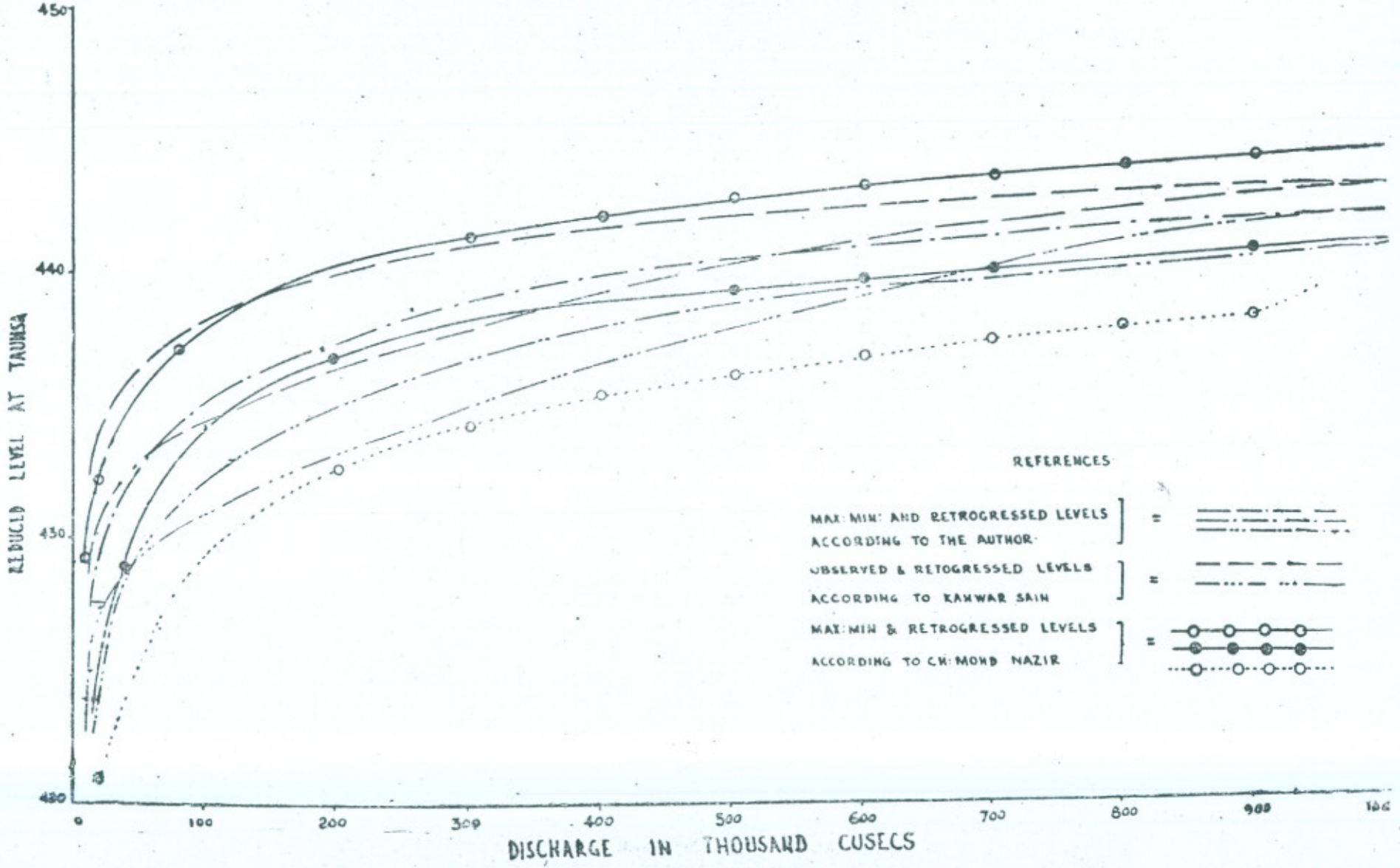
V

Method		Author's Method		
Mean	Minimum Retrogressed R.L.	Maximum R.L.	Minimum R.L.	Retrogressed R.L.
426·46	418·27	432·2	427·30	422·6
428·31	420·74	434·10	429·70	425·50
431·09	423·80	435·65	432·0	428·35
433·21	426·18	436·80	433·40	430·0
434·933	428·37	437·50	433·40	431·33
436·40	429·85	438·15	435·15	432·15
438·62	432 60	440·0	437·5	435·0
439·44	434·20	441·10	438·90	436·7
440·73	435·42	441·80	438·9	438·0
441·50	436·52	442·45	440·70	438·95
442·03	437·25	442 90	441·30	439·70
442·44	438·0	443·3	441·80	440·30
442·86	438·40	443·70	442·30	440·90
443·22	438·85	444·00	442·70	441·40
443·58	441·68	444·2	443·0	441·80

PAPER NO.315-B

FIG:-

DISCHARGE GAUGE CURVES FOR TAUNSA



MODEL OF RIVER SUTLEJ AT ADAMWAH

