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DEVELOPMENT AND APPLICATION OF FLOOD MANAGEMENT MODEL IN PRE AND POST DAM SCENARIO AT RIVER SUTLEJ

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Flooding is globally a major natural hazard. Pakistan faces flooding problems almost every year. In present study a model was developed to compute the flood discharge and to propose structural management intrusions accordingly. Gumbel's Extreme Value Type –I distribution (EV-I), Log Pearson Type-III distribution (LP-III), and HEC-RAS software approaches were used to develop the model based on Visual Basic for Applications (VBA). LP-III distribution gave more accurate results only for combined pre & post dam scenario having long discharge series (1926-2018) with R-square (R^2) value of 0.9973. While Gumbel (EV-1) distribution is good fit of line to data with R^2 value of 0.971 for post-dam scenario having shorter discharge series (1978-2018). Further, in post-dam scenario flood discharge computed by Gumbel (EV-1) distribution was 37% lower than LP-III. EV-I distribution results presented that flood peaks for the lower return periods were reduced significantly due to the construction of Indian dams. The design flood discharge of 384,765 cusec is peak design discharge computed by EV-1 distribution for 100 year return period. However, current discharge capacity of barrage to pass the flood computed by developed model was 295,492 cusec. To pass the peak flood discharge safely, evaluation of four structural flood management interventions revealed that barrage can pass the flood discharge of 384,765 cusec safely by raising the HFL by 1.94 ft. The proposed research is helpful in devising the guidelines for the rehabilitation of hydraulic structures to address 100 year return period flood without breaching the protection embankments.
Keywords: Flood Frequency Analysis, Gumbel (EV-I) distribution, Log Pearson Type-III (LP-III) distribution, Pre & Post dam analysis, hydraulic design, structural interventions, barrage.

INTRODUCTION

Hydrological studies are necessary for management of water resources and design of hydraulic structures such as dams, barrages, spillways, culverts and irrigation channels. So far 24 major floods occurred in Pakistan during the 1948 to 2016 period, causing financial loss of US$ 38 billion (FCC, 2016). Climate change has aggravated flood frequency and magnitude of floods (ADB, 2017). Riverine flows gradually variate owing to construction of storages & hydropower dams. Further, encroachments of flood plains of rivers due to rapid growth in population, enhanced industrialization; and increased agricultural developments have resulted in such areas which are always prone to flooding during flood seasons (Naseer, 2010-2012). Construction activities within flood plains like bridges, small dams, weirs & barrages have changed river's behaviour at structure locations and the river's morphology tends to change both on upstream and downstream of the structures. Therefore design & up-gradation of hydraulic structures are necessary required in line with advance hydrological studies & flood prediction models.

Many researches have conducted a research on selection of best fit flood frequency distribution methods. They found that it is difficult task to select the best fit flood frequency distribution due to estimation procedures available in the literature (Vogel and Wilson, 1996; Abida and Ellouze, 2007; Nirman, 2017; Kjeldsen et al. 2001). A World Meteorological Organization surveyed in 28 countries and revealed that Generalized extreme value (GEV) is a standard in ten countries, and LP3 is a standard in seven countries (Vogel, 1996). Researchers found that in USA, results of log Pearson Type III (LP3) & parameter lognormal (LN2) were the best. Whereas in Australia only LP3 was suitable distribution (Abida, 2007). However, study conducted in India at Lower Mahi Basin, endorsed that Gumbel (EV-I) distribution found best fit to data series (Nirman, 2017). Kjeldsen& Smithers studied at river KwaZulu-Natal in South Africa, and found that Pearson Type-3 was suitable at site (Kjeldsen, 2001). Mahdavi researched in Iran and compare seven probable distributing models including Pearson type III, log Pearson type III and Gumbel (EV-I) distribution and found that Gumbel distribution (EV-I) was best distribution by having more fitness with data series. Federal Flood Commission of Pakistan mentioned in National Flood Protection Plan -1978 that Gumbel's distribution method gives more accurate result for barrages in Pakistan compared to all other distribution methods. It illustrates that different frequency distribution methods are suitable at different research regions. An evaluation of peak flood discharge through flood frequency analysis and identification of structural interventions for hydraulic structures (dams, barrages, spillways) to pass the flood is vital for effective flood assessment and flood management.

Barrage is the back bone of irrigation, when floods damage the barrage it means entire irrigation system can be vulnerable to cause major damage (Akhtar, 2013). In 1988, in late moon soon season Indians dam reservoirs were full on River Satluj and Suleimanki headwork experienced the flood of 4,00,000 cusec, authorities breached the guide bank to safe the hydraulic structure and to avoided higher damage. The total loss evaluated by Punjab Govt was about US$ 858 Mn (FFP, 2013). The situation demands for an effective flood management to minimize flood damages. The impacts of the flood events are estimated based on the history of the flood events and study of the risk resulted as a consequence of the events (Wu *et al.,* 2011). There is a dire need to develop a model to predict peak flood discharge which would pass through structure, and evaluate various structural measures to pass severe flood through the barrage. Basically, the flood control standards are classified based on three steps: (i) determining the engineering classification of the reservoir type (ii) determining the grade of the hydraulic structure according to reservoir engineering guidelines (iii) determining the flood control strategies based on the location of the hydraulic structure (Ren *et al.,* 2017).

In the present research, a modelling study is done by developing the visual basic model for the Suleimanki barrage to evaluate; (i) the peak flood discharge for 100-year retune period under the pre- and post-Indian dams scenarios using Gumbel Extreme Value Type-I (EV-I) Method and Log Pearson Type-III Distribution Method (LP-III) through a model developed using programming codes in visual basic for application (VBA), (ii) the current discharge capacity of Suleimanki barrage by using the formulae given in HEC-RAS manual through a VBA worksheet model, and (iii) different structural flood management interventions using the VBA worksheet model. The study also recommended appropriate structural flood management interventions for Suleimanki barrage.

***Study area*:** Suleimanki barrage of Punjab, Pakistan (Figure 1) is located at latitude 30^{o} North and longitude 73^{o}East**.**Barrage is located about 12.42 Mile east of Haveli Lakha Town of District Okara. It was constructed across the Sutlej River during 1924-1926 under the Sutlej Valley Project (SVP) to irrigate 2.5 Ma area of Punjab, Pakistan. The river supply was cut off by India in 1960 according to Indus Water Treaty (IWT) between Pakistan & India (Sajid, 2011). The normal river supply was disconnected by the Indian Government after construction of storage dams on the upper reach of Sutlej River as list shown in table 1 (AAR, 1999). A link canal system was constructed from Mangla Reservoir to Suleimanki headworks to feed the off taking canals of Suleimanki headworks, total withdrawal capacity of these canals is 15,942cusec. However, due to construction of Indian dam the climate, atmosphere and historical peak flood pattern of study area has been changed. Historical annual peak discharges of Suleimanki barrage are shown in Figure 2 which depicts significant change in discharge peaks from 1977 to 2018.

Suleimanki barrage performed in a normal way till the implementation of Indus Water Treaty (IWT) of 1960. Flows towards Sutlej River gradually diminished by 1977, after construction of storages (Bhakra & Pong) and hydropower (Nangal & Pandoh) dams in India. River discharge downstream of Ferozpur barrage is almost zero for about 10 months in a normal year and river channel below the outfall of Balloki-Suleimanki (BS) link canal causes only the discharge brought by BS link, which being very low about 15,000cusecas compared to the river/pond capacity (Nespak, 2012).

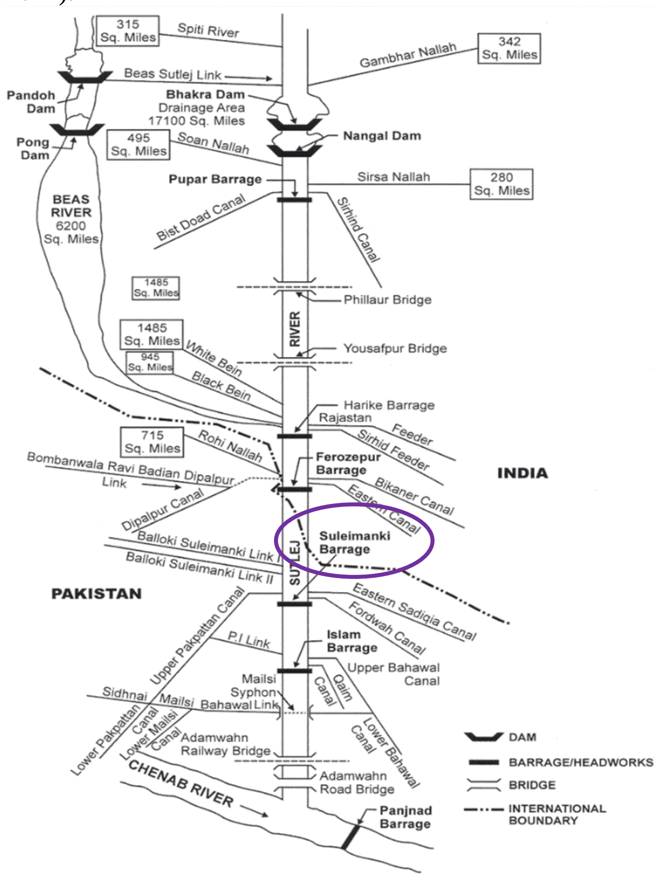

Figure 1. Location View of Suleimanki Barrage (Source: Sajid Iftikhar., 2011)

***Historical Floods*:** Three high floods have been experienced in River Sutlej at Suleimanki barrage after partition of Indo-Pak in 1947. During 1955, an unprecedented flood of 5,97,000 cusec experienced when surplus discharge passed

Table 1. Dams on River Satluj in India (Source: AAR., 1999).

Item	Dams in India				
	Bhakra	**Nangal**	**Pong**	**Pandoh**	**Nathpa dam**
River	Sutlej	Sutlej	Beas	Beas	Sutlej
Location	160 miles (257.4km) u/s from Ferozepur	8 miles (12.8km) d/s from Bhakra Dam	120 miles (193.1km) u/s from Ferozepur	95 miles (152.8km) u/s from Pong Dam	187 miles (300.9km) u/s from Bakhra dam
Year of completion	1964	1963	1972	1977	2004
Year of completion	1964	1963	1972	1977	2004
Gross storage capacity, Km³	9.62	0.0197	8.57	0.041	0.00343

through the breached of Right Marginal Bund (RMB) and Left Marginal Bund (LMB). It caused to affect the water supplies over 2.5 Ma for irrigation purpose. Second in 1988, when it was general thought that after the enforcement of IWT and construction of Indian dams at River Sutlej and Beas (list of dams is shown in Table 1) there would be no chance of flood in Pakistan at river Satluj, however these general thoughts proved wrong in late September 1988, and experience the flood of 4,99,000 cusec. Consequently, it caused huge damage to several villages, huge loss of crops, properties human beings. Third in 1995 i.e 3,00,000 cusec safely passed through the barrage and no allied structure was damaged (FFP, 2013; Nespak, 2012).

MATERIALSAND METHODS

***Evaluation of Flood peak (Design) Discharge*:** After the implementation of Indus Water Treaty (IWT), Indian dams of high live storages on Sutlej and Beas rivers, up to 1977, have resulted in the drastic change in pattern of river flows at Suleimanki barrage. Consequently, the pre-dam and post-dam data series are mutually non-homogenous. Due to this fact, the frequency analysis of data series has been carried out by splitting the full discharge series into three parts; first part was combined pre-dam and post-dam series (1926 to 2018), second part was the pre-dam series (1926 to 1977) and the third part was the post-dam series (1978 to 2018). The frequency analysis was carried out using Gumbel's Extreme Value Type-I (EV-I) distribution and Log Pearson Type III distribution (LP-III). The plotting positions have been computed by using Weilbull's formula (Harter, 2007; Sun *et al.*, 2018).

For the present study, required data included:

i. Flood history at Suleimanki barrage;
ii. Drawings of barrage in its present condition; and
iii. Historical data of discharge at Suleimanki barrage.

***Visual Basic for Application (VBA) Based Flood Management Model Development*:** A computer flood management model was developed by programming Gumbel Extreme Value Distribution Type–I (EV-I), Log Pearson Type-III Distribution (LP-III), and HEC-RAS approaches using Visual Basic for Applications (VBA) along with Microsoft Excel. Using the Microsoft Excel VBA codes were developed to make the functions, sub-routines and macros. Subroutines were used to break down large pieces code into small manageable parts however; functions were used as large pieces of codes (Green *et al.*, 2007). Moreover, in programming, newton-Raphson method was used for doing iteration of different numbers and repetition of the same work (Kaw *et al.*, 2011).

***Evaluation of Flood Peak Discharge for Suleimanki Barrage*:** In the present study, Gumbel Extreme Value distribution Type-I (EV-I) and log Pearson Type III distribution (LP-III) methods were used to evaluate flood frequency and flood magnitude for Suleimanki barrage (Kot & Nadarajah, 2000).The primary objective of these methods was to determine the return period of recorded event of known magnitudes (discharge values) and then, to estimate the magnitude (flood) for design return period within or beyond the recorded range.

The Gumbel Extreme Value distribution Type-I (EV-I) and log Pearson Type III distribution (LP-III) methods are briefly described below.

***Gumbel Extreme Value Type-I (EV-I) Distribution Method*:**. Gumbel's method is commonly used as statistical and probability distribution function to evaluate magnitude of severe flood for design return period. Gumbel distinct the flood as the largest value of the 12 months (1 year) flow therefore, the annual largest value of flow is considered as final value for that year and other all values of flow is considered as final value for that year and other all values of that year are ignored in this method (Subramanya, 2009; US Army Corps of Engineers, 1993).

The basic equation is given below.

$$X_T = \overline{X} + K\,\sigma_x \qquad (1)$$

Where, X_T = Magnitude of the event reached or exceeded on an average once in T years, X = Mean value, σx = Standard deviation of the variable

$$K = \text{Frequency factor} = K = \frac{y_T - \overline{y_n}}{S_n} \qquad (2)$$

Where, $\bar{y}$= Reduced mean, S_n = Reduced standard deviation, a function of sample size "n",

y_T = The reduced variate is related to return period =

$$-[\ln \ln \frac{T}{T-1}] \qquad (3)$$

T = Return period

Curve-expert software converted the tabular values into 6-degree polynomial equations, and used these equations in VBA modelling. Reduce mean "yn" and reduce standard deviation "Sn" were calculated by 6-degree polynomial equation as given under:

a = 0.432177862907054
b = 9.21946026433499E-03
c = -3.54972279509188E-04
d = 7.86464650409748E-06
e = -9.83602711100285E-08
f = 6.43758306207239E-10
g = -1.71085432185036E-12

$Reduce\ mean\ "Yn" = a + b \times n + c \times n^2 + d \times n^3 + e \times n^4 + f \times n^5 + g \times n^6$(3.1)

a = 0.696345885806217
b = 3.72609722142698E-02
c = -1.4503062094025E-03
d = 3.23565871906443E-05
e = -4.06501591578017E-07
f = 2.66878497093886E-09
g = -7.10824963497108E-12

Reduced standard deviation $Sn = a + b \times n + c \times n^2 + d \times n^3 + e \times n^4 + f \times n^5 + g \times n^6$(3.2)

Log Pearson Type III Distribution Method: Cronshey *et al.* (1981) described that in this method yearly highest magnitude of flow is first converted into logarithmic form (base 10) and the converted data is analysed. If 'X' is the discharge value from a hydrologic series, then the value of 'X' is converted into logarithmic (base 10) value as shown in equation given below:

$$Z = \log X \quad (4)$$

Where, Z =Logarithm of maximum annual flow.

By using following basic equation for any return period (T), the logarithm value of discharge is found.

$$Z_T = \overline{Z} + K_z \sigma_z \quad (5)$$

$$\sigma_z = \sqrt{\frac{\sum(Z-\overline{Z})^2}{(N-1)}} \quad (6)$$

$$Cs = \frac{N\sum(Z-\overline{Z})^3}{(N-1)(N-2)(\sigma_Z)^3} \quad (7)$$

Where, $\overline{Z}$ = Mean of Z values, σ_z = Standard deviation of Z series, Kz = Frequency factor function of coefficient of skew, Cs = Coefficient of skew for Z series, N = Size of sample

After evaluating 'Z_T' using equation 3.8, then its antilog is taken to find the discharge values as given below:

$$X_T = \text{antilogarithm}\ (Z_T) \quad (8)$$

There are many methods for frequency distribution to calculate the probability of flood for various return period. These methods are developed on the basis of historical data, by using statistical and probability functions. This is not necessary that all methods will give the same answer, because each method has its own statistical approach (Hamadi at el., 2013).

In these methods, frequency factor does not depend on geotechnical conditions. Rather it depends on climate and atmosphere change. Due to climate change, a historical peak flood pattern is changed. These distributions are dependent on statistical functions and formulae (Mahdavi, 2010). Historical annual peak discharges of Suleimanki barrage are shown in chart (Figure 2), which were used in the frequency analysis. Frequency analysis of data series has been carried out by splitting the full discharge series into two parts, pre dam (1926-1977) and post dam data series (1978 to 2018).

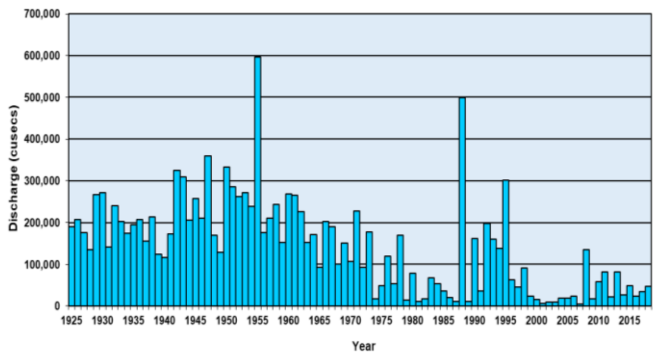

Figure 2. Annual Peak Discharges of River Sutlej at Suleimanki Barrage during 1926 to 2018 Period

***Evaluation of Current Discharge Capacity of Suleimanki Barrage*:** In order to calculate the discharge capacity of barrage, it was essential to find out the alternate depths just upstream of the barrage crest. Cubic equation was used to find the alternate depths. Cubic equation gave the three roots of the equation one negative root and two positive roots. In two positive roots bigger value was sub-critical depth of water whereas smaller value showed the super critical flow depth. To find the alternate depths cubic equation is given below (Bulu, 2020);

$$y^3 - Ey^2 + \frac{q^2}{2g} = 0 \quad (9)$$

Where, E = Energy head over crest =y + $v^2/2g$ (ft), y =Water depth (sub critical depth, super critical depth & critical depth), q =Discharge Intensity per unit width (cusec/ft), g =Gravitational acceleration (ft/s^2)

The VBA based function developed, to calculation sub-critical depth u/s of barrage and used computed value in calculation of discharge capacity of the barrage. Following equation is used to evaluate non-modular discharge (Brunner & Fleming, 2010).

$$Q = CL_eE^{1.5} \quad (10)$$

Q = Total discharge (cusec), C = Coefficient of discharge (-), Le = Actual length of waterway (ft)

Gibson Curve was used for applying necessary corrections in coefficient of discharge (C) due to submergence effects. Manning's equation and flow rate formula were used to estimate discharge capacity through silt excluder tunnels.

Tail water rating curves can be expressed in equation form as:

Existing average water level

$$H = 516.06\ Q^{0.0079} \quad (11)$$

Equation no. 11 used to calculate the tail water level (Nespak, 2012). Discharging capacities of the various tunnels of silt excluder have been calculated after making due allowance for the head losses inside the tunnels as mentioned by USBR (1987) and Chow (1988), following head losses alongwith their loss coefficients were considers and used to provide allowance for head losses inside the tunnels:

i. Entrance Loss ………With a loss coefficient of 0.7
ii. Contraction Loss …...With a loss coefficient of 0.3
iii. Exit Loss ……………With a loss coefficient of 1.0
iv. Friction Loss ……….With Manning's equation with 'n' value of 0.018

After determining the 100-year flood discharge at barrage, checked the barrage capacity by fixing Total Energy Level (TEL) with respect to guide banks levels and then calculated the hydraulic jump elevation. According to Garg (2005) hydraulic jump should be at the toe of the d/s glacis of the hydraulic structure. Hydraulic jump should not sweep from the toe of the d/s glacis. In the present research the VB function was developed using the conjugate depth methods for calculate the hydraulic jump, with 10 % submergence in Tail water level (Garg, 2005). Also use criteria given by Peterka (1984), "Hydraulic Design of Stilling Basins and Energy Dissipaters" (Peterka, 1984).

***Evaluation of Flood Management Structural Interventions*:** The existing barrage capacity was inadequate to pass design flood for 100-year return period. The following four interventions were evaluated to manage estimated flood design discharge:

(1) Enhancing barrage capacity by raising high flood level (HFL):
    i. Raising emergency HFL by 1.5 ft
    ii. Raising emergency HFL by 1.94 ft
    iii. Raising emergency HFL by 2 ft
(2) Enhancing barrage capacity by lowering barrage crest and raising HFL
(3) Enhancing barrage capacity by addition of bays and raising HFL
(4) Construction of bypass weir and raising HFL of barrage

RESULTS AND DISCUSSION

Flood Peak Discharge Analysing Combined Pre- and Post-dam Data Series of 1926 to 2018 Period: In this series, data of 93 years was used to estimate flood for different return periods. In Pakistan, 100-year return period is generally used in the design of barrages and small dams or small hydraulic structures. In this series, barrage faced only two time an exceptionally high flood first in year 1955 and second in 1988. The frequency analysis of combined pre-and post-dam series for period of 1926 to 2018 carried out by EV-I and LP-III distributions. The results of this analysis are presented in Table 2 and Figure 3.

Table 2. Results of Frequency Analysis Using Combined Pre- &Post-dam Data of 1926 to 2018 Period.

Return Period	EV-I (Cusec)	LP-III (Cusec)
2	127,081	108,408.7
10	300,235	341,990.5
25	387,385	463,569.6
50	452,038	548,221.5
100	**516,214**	**625,898.0**
200	580,156	696,663.0
1000	728,721	848,971.8

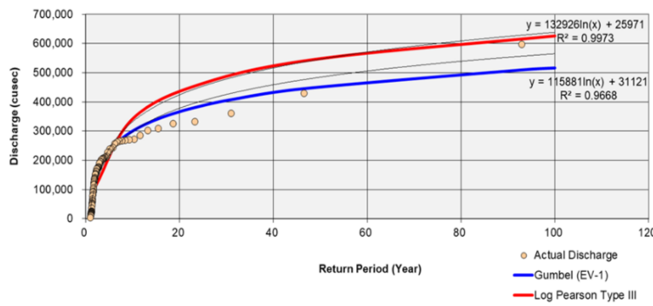

Figure 3. Flood Frequency Analysis for Combined Pre- and Post-dam Data of 1926 to 2018 Period.

The results showed that flood peaks even for the lower return periods are quite high. The 100-year return period flood discharges determined by EV-I and LP-III were 516,214 and 625,898 cusec, respectively. It reflected that LP-III estimated 21 % higher flood discharge compared to that determined by EV-I method. Based on regression analysis, trend line value of R-square (R^2) was 0.9668 for Gumbel (EV-I) distribution while R^2 value for LP-III distribution was 0.9973. It shows that LP-III is good fit of line to data, because its R^2 value is close to 1 as compared to EV-1 value.

***Flood Peak Discharge Analysing Pre-dam Data Series of 1926 to 1977 Period*:** The results of frequency analysis of pre-dam data series of 1926 to 1977 period performed using EV-I and LP-III distribution methods are presented in Table 3 and Figure 4. For this discharge data series, flood peaks calculated by EV-I distribution for different return periods were quite high compared to those calculated by LP-III distribution. For example, for 100-year return period, estimated flood discharge value by EV-I distribution was 519,029 cusec whereas same value determined by LP-III method was 355,061 cusec reflecting Gumbel's estimate 46% higher than that of LP-III.

These results revealed that flood peaks estimated by LP-III distributions do not vary after 10 year to 1000 year return periods which is not reliable and acceptable due to an error in prediction of flood for 1926-1977 series. Therefore, this method is not appropriate for this discharge series due to unrealistic results. This reflects that Gumbel method is more appropriate for Pakistani conditions. Whereas, based on regression analysis, trend line value of R-square (R^2) for

Gumbel (EV-I) distribution was 0.9692 while R^2value for LP-III distribution was 0.8652. It presents that Gumbel (EV-1) is good fit of line to data because its R^2 value is close to 1.

Table 3. Results of Frequency Analysis Using Pre-dam Data of 1926 to 1977Period

Return Period	EV-I (Cusec)	LP-III (Cusec)
2	185,211	205,088
10	333,751	303,059
25	408,513	321,987
50	463,975	330,126
100	**519,029**	**335,061**
200	573,881	338,185
1000	700,942	348,993

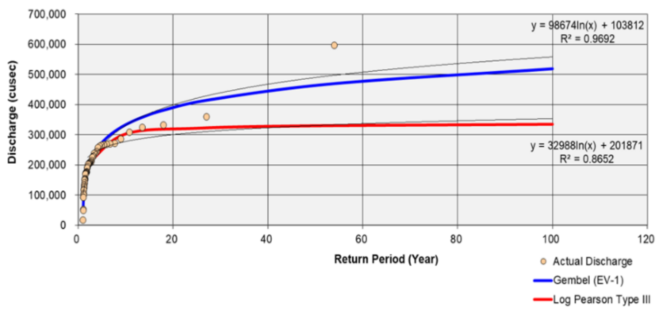

Figure 4. Flood Frequency Analysis for Pre-dam Data of 1926 to 1977 Period

***Flood Peak Discharge Analysing Post-dam Data Series of 1978 to 2018 Period*:** The observed peak flood discharge in 1988 (post-dam scenario) at Suleimanki barrage was 499,000 cusec. The results of frequency analysis of post-dam discharge data series for period of 1978 to 2018performed using EV-I and LP-III methods are presented in Table 4 and Figure 5. For 100-year return period, EV-I flood discharge estimation was 384,765 cusec compared to 626,572 cusec computed by LP-III reflecting 63% higher flood discharge than EV-I value. Table 4 presents a comparison of flood peak discharges estimated by EV-I and LP-III methods for discharge data series. Trend line value of R-square (R^2) for Gumbel (EV-I) distribution was 0.9692 while R^2 value for LP-III distribution was 0.8652. It presents that Gumbel (EV-1) is good fit of line to data.

Table 4. Results of Frequency Analysis Using Post-dam Data of 1978 to 2018Period.

Sr #	Return Period	EV-I (Cusec)	LP-III (cusec)
1	2	56,521	36,430
2	10	202,580	164,870
3	25	276,094	296,896
4	50	330,631	438,221
5	**100**	**384,765**	**626,572**
6	200	438,701	874,136
7	1000	563,640	1,773,968

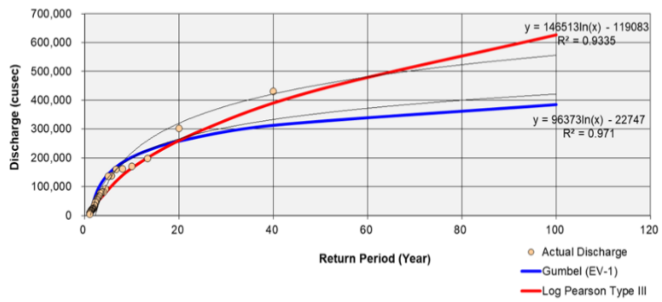

Figure 5. Flood Frequency Analysis for Post-dam Data of 1978 to 2018 Period.

The results reveal that the upstream dams in India have significant effect on flood peaks at Suleimanki barrage therefore, water flow in River Sutlej have been changed and for flood evaluation in present condition at Suleimanki barrage, post-dam series was most suitable condition.

In post-dam series, it showed that peak floods for low return period have been reduced at Suleimanki barrage but for high return period there is still chance to face a high flood at barrage. Design return period of 100 year is considered satisfactory for fixing design flood values for barrages in Pakistan. For 100-year return period Gumbel distribution estimated the flood peak discharge of 384,765 cusec. The Gumbel (EV-I) R^2 value for post-dam series is 0.971 while LP-III R^2 value is 0.9335 showing that Gumbel (EV-1) is good fit of line to data and Gumbel (EV-1) distribution is reliable for predicting expected flow in the river for 100 year return period.

Table 5. Comparison of 100 Year Return Period Flood Peak Discharges Determined by EV-I and LP-III Methods for Three discharge Data Series

Sr.#	Discharge Data Series	EV-I (Cusec)	LP-III (Cusec)
1	Combined pre-and post-dam series:1926 to 2018	516,214	625,898
2	Pre-dam series: 1926 to 1977	519,029	335,061
3	Post-dam series: 1978 to 2018	384,765	626,572

Table 6. Comparison of trend line R-square values by EV-I and LP-III Methods

Sr.	Discharge Data Series	Gumbel (EV-1) R^2 Value	Log Pearson R^2 value
1	Combined pre-and post-dam series:1926 to 2018	0.9668	0.9973
2	Pre-dam series: 1926 to 1977	0.9692	0.8652
3	Post-dam series: 1978 to 2018	0.9710	0.9335

Trend line R^2 (R-square) values for Gumbel (EV-1) and LP-III distributions have been depicted in Table 5. A trend line is most reliable when its R-squared value is at or close to 1.

Therefore, R-square values indicates that LP-III distribution is suitable for combined pre-& post-dam series (1926-2018) whereas Gumbel (EV-1) distribution is relatively more good fit of the line to the data for pre-dam (1926-1977) & post dam series (1977-2018).

***Comparison of results with Previous Study*:** Feasibility study of Suleimanki Barrage has been carried out by Nespak in November, 2012, the frequency analysis has been carried by only Gumbel (EV-1) for discharge series of 84 years (1925-2008). Estimated flood peak discharge for 100-year return period, was 449,000 cusec for the pre-dam and 416,000 cusec for the post-dam conditions [14]. It is necessary to mention that the present condition of barrage has been changed, it has silt excluder in left pocket and its stilling basin level have been raised by 0.75 ft. Therefore, it was necessary to re-evaluate its flood discharge for 100-year return period by taking into account the latest riverine discharge data up to 2018 (93-year discharge series).

In present research updated discharge series of 93 years (1926-2018) is used. Developed a VBA based computer model to compare the two distributions methods Gumbel (EV-1) and LP-III along with R^2 value and found that LP-III distribution is appropriate only for combined pre & post dam scenario having long discharge series (1926-2018). However, Gumbel (EV-1) distribution is more accurate for post dam scenario having shorter discharge series. Estimated flood peak discharge for 100-year return period, by Gumbel (EV-1) is 384,765 cusec, 8% lower than the feasibility study estimated discharge. The continues reduction in discharge peaks have significant impact on estimated flood peaks. Similarly, flood management interventions accordingly need to decide.

Evaluation of Current Discharge Capacity of Suleimanki Barrage: In 1926, barrage was designed for 325,000 cusec with upstream HFL of 572.00 ft and downstream HFL of 569.00 ft. The main weir share was taken as 210,000 cusec and each under sluice was taken as 57,500 cusec. The coefficient of discharge for weir portion was considered as 3.10 and for under sluices 2.5 based on the submergence conditions of 75 % in main weir portion and 85 % in under sluice portion, respectively.

But, during evaluation of discharge capacity of barrage in its present condition (2017), it was observed that now it has flood passing capacity of 295,492cusec, water level downstream of the barrage had been raised due to phenomena of accretion at downstream of barrage. It directly affected the submergence conditions of barrage. The submergence was 80% in main weir portion and was 88% in under sluice portion. Consequently, hydraulics of barrage was changed. The submergence directly affects the coefficient of discharge. The VBA worksheet model revealed that the coefficient of discharge for weir portion was 2.95 and 2.5 for undersluices (Sharma, 2017). The VBA model found that to restore the original design capacity of barrage (325,000 cusec), HFL needs to be raised by 0.66 ft as shown in Table 7.

Table 7. Current Discharge Capacity of Suleimanki Barrage

Sr.	Raise in high flood level (ft)	High flood level (HFL) (ft)	Total discharge (Cusec)
1	0 (Present condition)	572.00	295,492
2	0.66	572.66	3,25,000

Evaluation of Structural Flood Management Interventions

***Intervention 1: Enhancing Discharge Capacity by Raising HFL*:** Safe discharge capacity for the barrage is the peak flood discharge which can be passed through the barrage with adequate (10%) hydraulic jump submergence. The evaluation of the intervention; *raising HFL* through VBA model revealed that entire high flood could be passed through the existing barrage resulting in higher flood level as compared to the design value. This situation gives rise to a potential danger of breaches through marginal bunds due to increased hydraulic pressures. Also, the quantity of flood discharge, passing per unit width of the barrage, gets enhanced. This may result in damages to different components of the barrage, like marginal bunds, guide bunds, spurs. Therefore, these components need to be raised and strengthened under modern design criteria. Other components like, stilling basin length and flexible stone apron were calculated by VBA model and it was found that just increasing the length of stone apron needs to be increased. The evaluation results of options of 1.5, 1.94 and 2 ft raise in original HFL of 572 ft are presented in Table 8 and Figure 6.

Table 8. Evaluation Results of Enhancing Barrage Capacity by Raising HFL Intervention.

Sr.	Raise in HFL (ft)	HFL (ft)	Total discharge (cusec)
1	0 (Present condition)	572.00	295,492
2	1.50	573.50	3,64,510
3	1.94	573.94	3,85,000
4	2.00	574.00	3,88,351

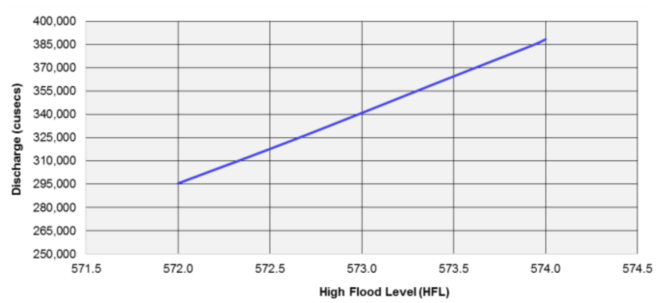

Figure 6. Evaluation Results of Enhancing Barrage Capacity by Raising HFL Intervention

Table 9. VBA model

HYDRAULIC PARAMETERS AT HFL= 573.94 ft

Totel discharge	cusec	385000	
Main wier	cusec	248920	
Undersluicies 3 bays (With silt excluder)	cusec	14151	Flood Passing Capacity
Undersluicies 5 bays (Without silt excluder)	cusec	43720	
Tunnels Discharge	cusec	8257	
Undersluicies Right	cusec	69952	
U/s Energy Level	El ft	574.71	
Tail Water Level; TWL= 516.06 x $Q^{0.0079}$	El ft	571.25	**385000**
Gravitational force (g)	ft/sec^2	32.20	

Description	unit	Left Pocket (without silt excluder)	Left Pocket (with silt excluder)	Main Weir	Right Pocket
Bay width	ft	30.00	30.00	60.00	30.00
Crest level	El ft	552	561	560	552
u/s Bed level	El ft	552	552	552	552
Pier width	ft	5.00	5.00	7.00	5.00
no of bay	#	5	3	24	8
Weir hight	ft	0	9	8	0
Assume Q	cusec	**8744**	**4717**	**10372**	**8744**
q (u/s Floor)	cusec/ft	250	135	155	250
Water depth u/s of crest	ft	20.37	22.13	21.94	20.37
Water Level (u/s)	El ft	572.37	574.13	573.94	572.37
Weigted Avg WL u/s	El ft	573.08			
V	ft/sec	12.26	6.09	7.06	12.26
Ec	ft	22.71	13.71	14.71	22.71
tail Water Level over crest	ft	19.25	10.25	11.25	19.25
Submargance	% = value x 100	0.848	0.75	0.76	0.85
Constant		2.73	3.13	3.08	2.73
Q	cusec	8744.03	4716.99	10371.66	8744.03
DIFF		0.00	0.00	0.00	0.00
Stilling Basin Level (SBL)	El ft	549.00		549.75	549.00
Jump Elevation (JE)	El ft	550.16		554.66	550.16
conjugate depths (d1)	ft	9.30		5.68	9.30
conjugate depths (d2)	ft	19.61		15.45	19.61
Discharge intensity b/t pier (q1)	cusec/ft	291.47		172.86	291.47
Discharge intensity after pier (q2)	cusec/ft	249.83		154.80	249.83
Velocity before jump (v1)	ft/sec	31.33		30.42	31.33
Froud no		1.81		2.25	1.81
JE above SBL	ft	1.16		4.91	1.16
Submergence of jump	%	1.1		1.4	1.1
Length of SB	ft	82		70	82
Provided	ft	80		65	80

Intervention 2: Enhancing Barrage Capacity by Lowering Barrage Crest and Raising HFL: The evaluation of the intervention; *lowering crest of weir*, by breaking the main weir crest the severe flood discharge could be passed through the existing barrage, but it was found that there is also a need to raise the HFL (Table 10). In under sluice portion crest can't be lowered because it is already at the level of upstream floor, therefore, this intervention will be applied only to main weir. But, this intervention has a major demerit for the modification process. Piers and the weir crest floors are two independent structural elements of two different materials. Breaking and lowering of crest, can cause cracks in the concrete masonry which cannot be ignored, this shall eventually create problems for the barrage structure. Consequently, this intervention was not considered feasible for adoption for flood management.

Table 10.

Sr.	Crest Level Lowered by 1.0 ft (ft)	HFL (ft) Table 10. Evaluation Results of Enhancing Barrage Capacity by Lowering Barrage Crest by 1.0 ft and Raising HFL by 1.5 ft Intervention	Total Discharge (Cusecs)
1	559	572 without increase	3,09,698
2	559	573.5 with 1.5 ft increase	3,85,000

***Intervention 3: Enhancing Barrage Capacity by Addition of Bays and Raising HFL*:** Evaluation of intervention; *addition of bays*, revealed that by increasing the number of bays on the right undersluice of barrage, danger of greater pressure on different embankments of barrage could be eliminated. For this purpose, the head regulator of Pakpattan canal will have to be dismantled and reconstructed at new location in the relocated right abutment wall. Upstream and downstream guide banks along the right side of the barrage would have to be dismantled and reconstructed at the extended end of the barrage. During low discharges, there is a tendency of bela and island formation upstream of the barrage. Widening of waterway is likely to accentuate this problem. Vigilant regulation and proper river training will be required to tackle such problems, which is common at almost all barrages to varying degrees. This alternative is not viable; therefore, this intervention was also not considered for enhancing barrage capacity. The evaluation results of this intervention are depicted in Table 11.

Table 11. Evaluation Results of Enhancing Barrage Capacity by Addition of Bays and raising HFL Intervention

Sr.	Parameters	Results
1	Discharge	89,273 cusec
2	Right undersluice one bay discharge	6926 cusec (At HFL 572.00 ft)
3	Number of additional bays	13
4	Width of each bay	30 ft
5	Clear waterway	390 ft
6	Number of piers	12
7	Width of each pier	5 ft
8	Total width of extended portion	450 ft

***Intervention 4: Construction of Bypass Weir and Raising HFL of Barrage*:** The intervention; *construction of bypass weir*, can be applied for management of flood for more than 100-year return period. Barrage has faced more than 1:100 year return period discharge. Under this scenario, provision of bypass channel with auxiliary weir is best option. Barrage can pass 388,000 cusec at HFL 574.00. Therefore, the only option to pass more than 1:100-year return period discharge, is to provide bypass canal with auxiliary head regulator at its right side. Proper arrangements will also be needed for level crossing/aqueduct or syphon at Pakpattan canal upper for crossing of bypass channel having capacity more than 89,273 cusec. For 100-year return period, this intervention is not recommended, but for more than 100-year return period or for 1988 flood value this intervention is best one. The evaluation results of this intervention are provided in Table 12.

Finally, the intervention; *raising HFL by 1.94 ft* can be considered as the most suitable intervention to pass the adopted design flood of 384,765 cusec.

Table 12. Evaluation Results of Construction of Bypass Weir and raising HFL of Barrage Intervention

Sr.	Parameters	Results
1	Discharge	89,273 cusec
2	Right undersluice one bay discharge	6926 cusec (At HFL 572.00 ft)
3	Number of additional bays	18
4	Width of each bay	40 ft
5	Clear waterway	720 ft
6	Number of piers	17
7	Width of each pier	5 ft
8	Total width of extended portion	805 ft

Conclusions and Recommendations: It is concluded that dams on River Sutluj have significant effect on flood peaks at Suleimanki barrage, water flow behaviour in River Sutlej has been changed for flood evaluation in present condition at Suleimanki barrage. Based on regression analysis, LP-III distribution is suitable for only combined pre-& post-dam series (1926-2018). Whereas, Gumbel's distribution (EV-I) trend line R^2 value is good fit of line to data in pre-dam scenario (1947 to 1977) & post dam scenario (1978-2018). Post-dam scenario was considered as the design (peak) flood discharge for the barrage for 100 year return period and find that it R^2 value is good fit of line to data. Therefore, Gumbel (EV-1) distribution method is more suitable for predicting expected flood flow in the river Sutlej for 100 year return period.Moreover, Gumbel's distribution (EV-I) results illustrates that flood peaks for the lower return periods are reduced significantly due to the construction of Indian dams on River Sutlej while for the higher return periods the reduction in flood peak is much smaller. The design flood discharge 384,765 cusec computed by EV-I method for 1:100-year return period. Whereas, current discharge capacity of barrage to pass the flood came out about 295,492 cusec. Evaluation of different flood management structural interventions revealed that Suleimanki barrage can pass the 1:100-year flood discharge of 384,765 cusec byraising HFL (1.94 ft) without any modification in the existing structure. Accordingly, upstream embankments may be upgraded with enough free board. The implementation of management techniques emerged from the proposed research could prevent flood damages in future. Developed model is also applicable to other barrages / hydraulic structures for estimation and management of flood.

Recommendations**:** Though the HFL computed by using computer model is quite accurate however, it is recommended that before applying recommended proposal practically, physical model study should be conducted. The prediction curves in EV-I and LP-III methods should be modified to obtain more accurate results especially when large series of historical floods are available.

There were limitations in developed model regarding economic analysis therefore, for structural flood management interventions economic analysis need to develop.

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