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# JAN. 2016 – DEC. 2017

# PROCEEDINGS

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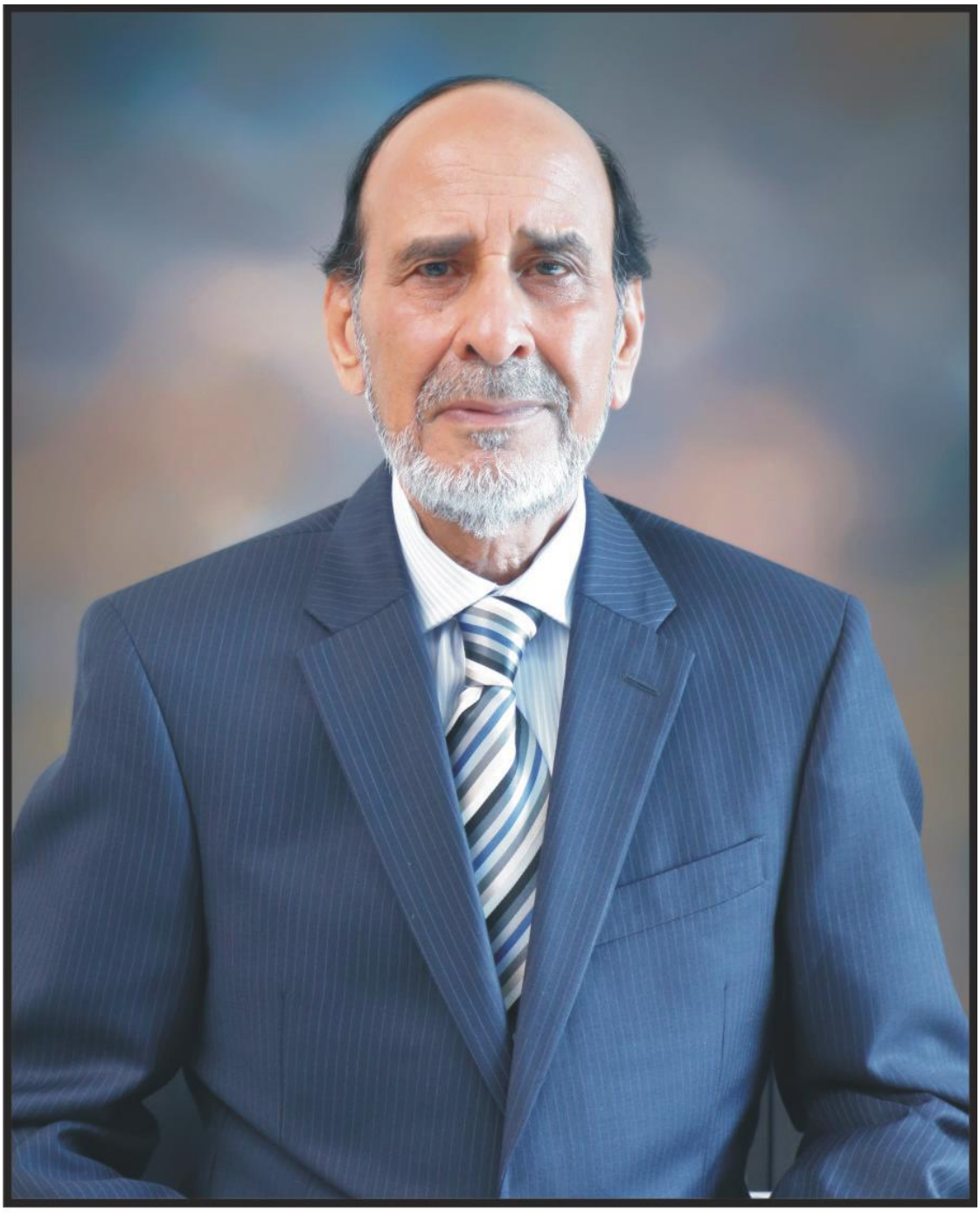
## President, Pakistan Engineering Congress

Engr. Ch. Chulam Hussain is the Chief Executive of M/s National Development Consultants (Pvt) Limited, one of the leading Engineering Organizations of the Country. He was born on October 10, 1042 at Amritsar India. He passed B.Sc (Civil Engineering) in 1965 from the University of Engineering and Technology, Lahore. He joined "Water and Power Development Authority" and was involved in the execution of Indus-Basin replacement works. He did M.Sc. in Civil Engineering with specialization in Soil and Foundation in 1978. Engr. Ch. Ghulam Hussain has about Fifty two (52) years vast and varied experience of National and International Development Projects in Libya and Saudi Arabia where he supervised the construction of Makkah Jeddah Madina Expressway. In Pakistan he remained involved in the execution of mega projects such as: National Drainage Programme, Punjab, Sindh, NWFP and Balochistan Components; Chashma Right Bank Canal (CRBC); Pehur High Level Canal; Right Bank Outfall Drain – I & III; Fordwah Estern Sadiqia Drainage Component; SWABI SCARP; Gomal Zam Dam Multipurpose Project; High Head Indus Tributaries (Alai Khwar-Duber Khwar-Khan Khwar) Hydropower Project; Diamer Basha Dam; Hingol Dam;

He is author of Nine (9) Technical Papers which were presented at National and International Forums and Published in renowned journals.

- Currently he holds the portfolio of the President of Pakistan Engineering Congress, which is the most prestigious and the oldest Engineering Organization of the Sub-continent established in 1912.
- He remained President of Association of Consulting Engineers Pakistan (ACEP) from April 2001 to April 2004 and April 2014 to April 2016.
- He was the Vice-President of Federation of Consultants from Islamic Countries (FICIC) Region-6 for 2 terms, i.e.: 2012-14 and 2014-16.
- Member, American Society of Civil Engineers.
- He worked as Secretary General, Pakistan Engineering Congress from January 2001 to December, 2003 and January, 2007 to July, 2011.
- He also worked as Vice-President, Pakistan Engineering Congress from January, 2004 to December, 2015.
- He remained member of the Central Council of IEP for three consecutive terms from 1992-2000.
- He is member of the Governing Body of Pakistan Engineering Council.

Engr. Ch. Ghulam Hussain has been elected un-opposed as President of Pakistan Engineering Congress, for term 2016-2017. With his extensive International administrative experience, he managed the Congress affairs in a very successful and befitting manner during his tenure.



**Engr. Ch. Ghulam Hussain**
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**Paper No. 774**

# WATER ACCOUNTING AND AUDITING TO MANAGE WATER SCARCITY -- GLOBAL EXPERIENCE AND POSSIBLE GUIDELINES FOR PAKISTAN

**Dr. Mahmood Ahmad**

# WATER ACCOUNTING AND AUDITING TO MANAGE WATER SCARCITY -- GLOBAL EXPERIENCE AND POSSIBLE GUIDELINES FOR PAKISTAN

By

**Dr. Mahmood Ahmad[1]**

## Abstract

Pakistan boasts the largest contagious irrigation system in the world, which also poses large it difficulties for its management. There is a need for proper accounting and auditing of water resources, taking stock beyond the simple accounting of volumes and flows, and a need to focus on issues relating to accessibility, uncertainty and governance of water. This paper introduces the concept of water auditing pertaining to institutional and governance issues related to water management. This paper looks at innovative work ranging from simple crop and water budgets to the use of remote sensing and in-situ sensing technologies (water informatics), that can allow real-time water accounting for over as small an area as a farm or as large an area as a river basin. Based on the results of Remote Sensing technology driven accounting exercises in Pakistan, we identify loss of storage, low beneficial depletion, and low land and water productivity as the main water resources management issues.

The Center for Water Informatics and Technology (WIT) at LUMS is expected to initiate policy work on water accounting which is critical in addressing the challenges of water scarcity. The scope of research work can include: first to rationalize water use in agriculture taking the case of rice and sugarcane and secondly to cross-pollinate water economics with water informatics technologies (e.g. remote sensing) for on-farm water management. We expect private sector, international donors and the Government to support this initiative at all levels.

## 1. The Context

Pakistan being the largest contagious irrigation system also makes it difficult to manage. There is a need for proper accounting and auditing of water resources, taking stock beyond the simple accounting of volumes and flows, there is also a need to focus on issues relating to accessibility, uncertainty and governance of water. The report card often presented that compares ecological, social and economic information against predefined goals or objectives does not reflect actual health of the water basin. In our view there is a need to develop performance driven scores that reflect the status of a river basin on regular basis so that information can be communicated to the decision makers which is not so difficult with digital connectivity at its best.

## 2. The concept of Water Accounting and Auditing

Behind the concept of water accounting is understanding the hydrological cycle as indicated in illustration 1. It entails assessing sources and uses of water. It also points out some concepts that are not well understood in water accounting analysis such as: 1) necessity to distinguish storages and fluxes; 2) endless possibilities of double counts as the same water should not be

[1] Visiting Researcher, Center for Water Informatics & Technology (WIT), SBA School of Science & Engineering, LUMS, Lahore, Pakistan, Senior Policy Officer (Water), FAO Regional Office Cairo, Egypt (retired)

counted twice (upstream and downstream, or aboveground and underground). It is important to understand that keeping separate accounts for surface water and groundwater make no sense.

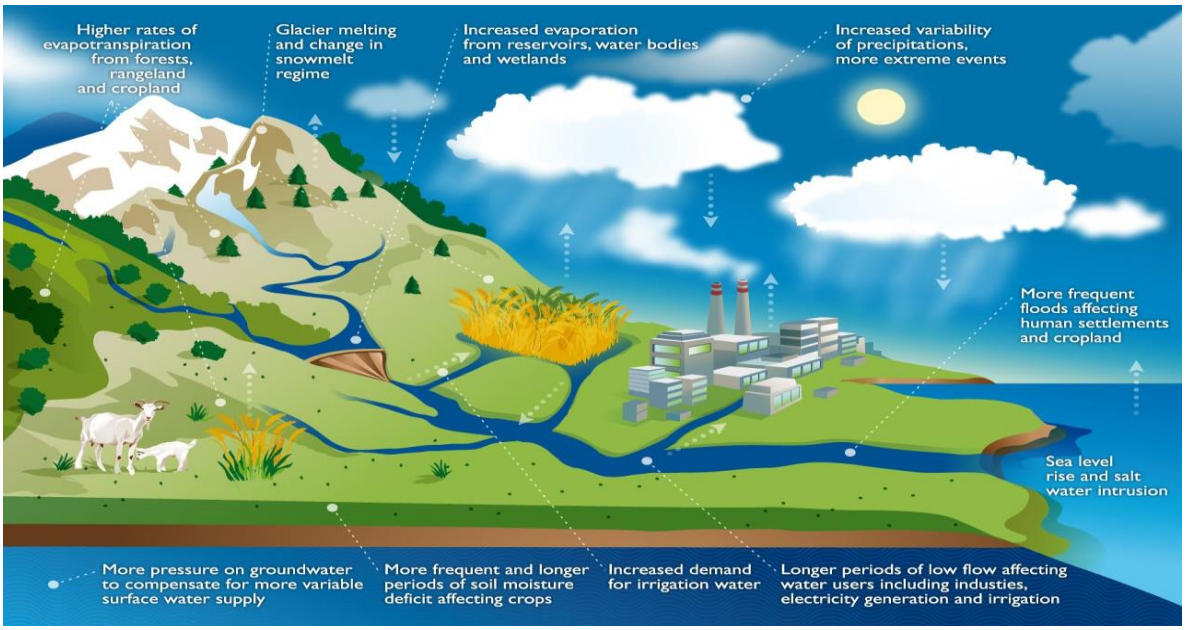



***Illustration 1:*** A typical Hydrological Cycle showing water sources and users (Source: FAO)

## 2.1 Water Accounting

Water accounting includes the systematic identification, assessment and analysis of the following in space and time[1]:

- Status and future trends in water supply, demand accessibility and use;
- Underlying causes of imbalances in supply and demand;
- Levels of environmental sustainability;
- Levels of water efficiency and productivity from source to sink and along the value chain;
- Capacity, functionality and O&M of water supply, storage, treatments and drainage related infrastructure;
- Water services, levels of different uses and users;
- Condition of environmental flows;
- Levels of equity and levels of competition for water;
- Functionality and effectiveness of M&E systems;
- Identification and assessment of potential externalities linked to, for example, land use change and agricultural intensification.

[1] FAO (2017), World Water Council members-driven initiative on “Water Accounting for agriculture”. Online discussion on “Water Accounting & Water for Food”.

## 2.2 Water Auditing

On the other hand water auditing looks at issues related to institutional and governance's issues facing water sector, includes the systematic identification, assessment and analysis of the following at different institutional levels:

- Systems of water governance and particularly the way in which power and authority are exercised and mediated;
- Mandates, interactions, functionality and accountability of formal and informal water-related institutions;
- Utility and effectiveness of different water services delivery models;
- Effectiveness and utility of policies relating to water and food security, poverty alleviation etc;
- Levels of public and private capital and recurrent expenditure;
- Profitability and cost-benefits of different water uses and users;
- Laws and regulations (formal and informal and their enforcement);
- Effectives and transparency of systems water allocation and regulation;
- Wider political economy issues that often underpin sanctioned discourses and resistance to change.

## 3. Water Accounting and Auditing Analysis – Framework

It is proposed to initiate policy work on water accounting which is critical in addressing the challenges of water scarcity. The scope of the research work can include: Environmental-Economic Accounting for Water (EEAW), Water availability cost curve, Participatory groundwater monitoring, Trading water rights, Water footprints, Water accounting for firms, Remote sensing based water accounting and Auditing – Problem focused approaches. The flow chart below provides framework to analyze water accounting and auditing issues and options to be addressed in Pakistan under the frame work provided in figure 1.

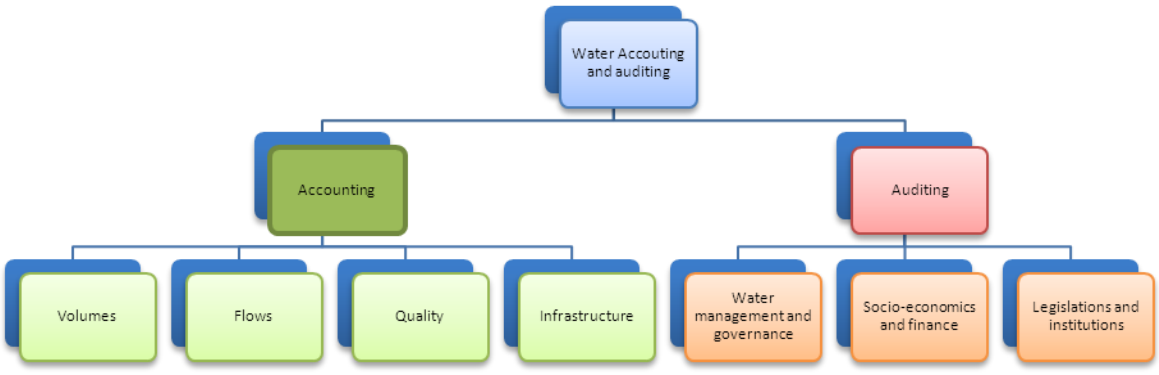



***Figure 1:*** Flowchart of Water Accounting and Auditing framework

## 4. Issues and Concerns in Pakistan

### 4.1 The Volumes

Though the water volumes increased post Tarbela/Mangla era, Pakistan's water shortage cycle is getting worst over time as the Kharif season demands three times more water than Rabi, causing water shortage of 13.12 MAF. Having very little storage capacity to store the summer excessive flow, limits the ability for the canal diversions within a range from 100 to 106 MAF[1]. This shortage is putting added pressure on already depleting groundwater. Two solutions may be implemented, in the short-medium term to rationalize water use in agriculture, especially crops like rice and sugarcane and in the long term creating more storage at all levels.

### 4.1.1 Flows

Other than large seasonal variations in water availability (80% in summer, 20% in winters). Climate change and global warming is making water flow more erratic – too much (floods) or too little (drought) have become a regular phenomena. At farm levels the reduced flow creates an issue of equity (head/tail) seriously questioning the viability of existing Warabandi system. There are two kinds of flows which are important from the integrated water management in Pakistan. First is the concept of return flow - often not well understood, it carries huge implications for investments in adopting water saving technology. Secondly, the issue of returns flows needed below Kotri to preserve the ecology of coastal areas (see section on climate change).

Water flows are increasingly of low quality as river carry heavy sediment load in the ranges from 200 to 400 million tons, causing depletion of storages at the rate of 1% per year which in turn reduces the life of the dams on one hand but quantity/quality of available water for productive use on the other. There is a need to reduce sedimentation load in water flows in streams through proper water shed management and using other techniques. The existing storage is running out of economic life, for example Tarbela's life is expiring in 2025, if a new reservoir of about 10 MAF capacity is not constructed by then, about 18 million hectare agricultural land in Indus Basin Irrigation System (IBIS) will start drying up.[2] Pakistan must start construction of Kalabagh or other feasible dams immediately otherwise after 2025 provinces will be fighting over water, the country's food security will be at extreme high risk, more than 60% of population will lose its livelihood, major exports that are agriculture based will diminish, economy could also collapse, as our economy is now essentially a water economy.

### 4.1.2 Quality

Water quality has been degrading over time due to sewage and industrial effluent discharges, urban and agriculture run off as well as saline water intrusion making its quality so low that it might not be safe for public use. Further, UNDP report[3]highlights that water pollution caused by organic matter, pathogenic agents and hazardous and toxic wastes is serious. Pollution loads discharged inland water bodies have estimated to double by 2025. The impact on health has been that good share of stomach diseases are attributed to deteriorating water quality and affordability.

The water quality is also affected by lack of proper management of watersheds; there has been a denudation of rangelands and watersheds depletion of forest cover and vegetation resulting in soil erosions. On the other hand 38 percent of Pakistan's irrigated land is waterlogged and 14 percent is saline; there is now saline water intrusion into mined aquifers. The quality of

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1 BIPP's 9th Annual Report (2016), The State of the Economy Agriculture and Water , Lahore, Pakistan

2 Asian Water Development Strengthening Water Security in Asia and the Pacific (2016), ADB.

3 Water Security in Pakistan: Issues and Challenges (2016), Development Advocate Pakistan, UNDP.

groundwater ranges from fresh (salinity less than 1000 milligrams per liter) near the major rivers to highly saline further away, with salinity more than 3000 mg/l TDS. Close to the edges of the irrigated lands, fresh groundwater can be found at a depth of 20-50 meters but start to fall significantly, further increasing the cost of pumping.

The solution lies in better regulations and their implementation for point and non-point pollution - proper treatment of sewage and industrial toxic waste necessary to improve the water quality. There are cost effective ways to treat water which can be used for economic activities. Secondly, a proper ground water management and drainage system are key to solving the issue. However, education and public awareness especially young people are needed to prevent pollution at source.

### 4.1.3 Asset Management

Snowfall, Glaciers and Rainfall are the main water resources of Pakistan. Overtime, the country has invested heavily in creating large water assets that include dams, canals , distributaries , moga and an extensive water distribution system to make beneficial use of available supply. There are two water assets where Pakistan has under invested but benefits could have been much higher – creating a proper drainage and water harvesting. Though investment in drainage has taken place but very little policy actions on water harvesting. Rainwater can be harvested in areas receiving rainfall as little as 50 – 80 mm of average rainfall. Presently it is estimated that no more than 20-25 percent of rain water is harvested with current practices. Modern harvesting techniques followed in different parts of the world can enable Pakistan to capture almost up to 90% of the total rainfall. This potential needs to be exploited on a more scientific basis to meet the future food and fodder requirements. [1]

Groundwater is an asset that is the most valuable, as it now provides a larger share in water supply and acts as a bank to provide water on demand. There are presently more than 1.5 million tube wells in the Indus Basin area, but this growth has been unchecked in the absence of strong regulation resulting in ground water depletion. For example, traditionally water boring in Punjab was undertaken at 120- 150 feet but at present boring depths of 300-400 feet are common with replacement bores required every 5-8 years. In the wake of canal irrigation, water shortages and reduced water flows to middle and tail enders, farmers largely depend on this reliable water supply without knowing the economic consequences to the economy and our future generations in terms of reduced supply. In other words the most valuable water asset is being squandered with irreversible damage and high future cost to the economy.

Then there are water assets that have higher returns on investments but largely neglected, such as micro investments in village storage, micro hydel projects, small dams, and embankment protections/dikes.

## 4.2 Water Auditing

Three aspects of water auditing are important in Pakistan (1) too many institutions serving the water sector (2) the water reforms lacks the needed commitments from stakeholders and, (3) water allocation policies are faltering as water scarcity grows.

### 4.2.1 Institutions

Institutional arrangements to deliver water at different levels of distribution are inefficient, inadequate, inequitable, and full of corruption and with low public participation. There are too many organizations serving the water sector and often lack coordination among agencies responsible for water supplies to the end users. There is a need to bridge the disconnect

[1] Ibid

between Ministry of Water/Irrigation (water provider) and Ministry of Agriculture (water user). A large part of the budget is spent on administration (salaries) with very little left for O&M. On the other hand, only about 50% of the operation and maintenance costs of the government run irrigation facilities are recovered through irrigation service fees and many existing water distribution systems for irrigation are deteriorating fast.

Addressing water management issues needs an out of the box thinking and major transformations - from supply to demand management, from engineering dominated solutions to integrated water resources management (IWRM). At a global stage, water policy/management "paradigm shift" is taking place but progress in Pakistan is slow. The traditional paradigm, which continues overwhelmingly "technocentric" and emphasizes more on engineering solutions, water storage and huge bureaucratic set up with too many institutions, continues to do business as usual. For planners and policy makers, water is treated as "a mere raw material," and "technical and scientific knowledge" is deployed "to harness it to its fullest capacity.[1] For this reason a change in mind set is needed --- water resources all along the supply chain carry huge value, which has not been fully tapped.

## 5. Water reforms lack commitment

There is substantial scope for the rationalization of water related public institutions in Pakistan including processes of decentralization and devolution of authority. These reforms need deep commitment at all levels. Past reforms have proceeded slowly and to different extents in different provinces. While the Government agrees on the direction of the reforms, the pace at which the decentralization and process can be implemented remains unclear. The likely trajectory of institutional reforms in each province would depend on a number of factors, a complex interaction where the outcome is difficult to predict.

## 6. Water allocation policies are faltering as water scarcity grows

The main problem for policy makers/institutions is to allocate water amongst competing uses as scarcity increases. Broadly, water can be allocated through pricing mechanism or through administrative mechanism, in Pakistan water is allocated legally through a well established mechanism – at international levels, trans boundary agreement with India, at provincial levels, water accord provides a working mechanism and at farm levels warabandi system provides water to farmers on time sharing basis, and according to their crops requirements. There are a number of researchers[2], who argue that despite the accurate criticisms made of mismatch between supply and demand, warabandi is an exceptionally stable and productive system for sharing scarce water supplies with minimum administrative input over vast areas, warabandi clearly defines the water rights', it is a complete and complicated system to equitably share uncertain and very scarce supplies among very large numbers of farmers of extremely large (millions of hectare) systems. If we combine the area in Pakistan and NW India, it alone would constitute to one of the largest irrigation systems with warabandi system up and running. Research further points that, one of the reason why WUA interventions have failed in Pakistan because water rights have not been defined to the group collectively or, in consequence to the individuals.[3]

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1 Simi Kamal (2009), Pakistan's Water Challenges: Entitlement, Access, Efficiency, and Equity In RUNNING ON EMPTY ,Pakistan's Water Crisis , Woodrow Wilson International Center for Scholars, Washington, D.C. www.wilsoncenter.org World Bank, Middle-East and North Africa, Outlook, Future Water, April 2011

2 FAO (2011), Coping with water scarcity- An action framework for agriculture and food security, Report 38, Rome, Italy.

3. FAO (2017), World Water Council members-driven initiative on "Water Accounting for agriculture". Online discussion on "Water Accounting & Water for Food".

However, as the time goes and water scarcity increase, a second generation of problems are arising, the irrigation supply system at farm level has lived its time and future direction need be researched, piloted and more important to be scaled, the new water policies be reflecting that, otherwise like energy (gas) we would forced to enact unpopular policies. Creating water rights is often suggested, water rights and market though alleviate the problems of water allocation, availability, and accessibility, but are not conducive to water saving. This is true especially where water rights are not well defined or enacted. In Pakistan, given its social setup, paramount concern is that trading in water would add to the existing inequity between up and down stream user and therefore putting a value on the water rights is not desirable. One has to weigh these negative externalities against the expected benefits from policy shift, keeping in view the facts on the ground. Though water markets are illegal, more than 70 percent farmers trade water on the watercourses to meet scarcity and improve supply.[1] In response to the water scarcity, informal water markets exist. The debate should focus more on whether water rights should be tied to the land or can be traded separately.

Though it is time to get back to the drawing board to craft supply and demand strategies on basis of water zones/river basins. Gilgit Baltistan is a good place to initiate work on a pilot basis, to measure water supply and provide water on demand to competing sector (agriculture domestic and energy generation). With Satpara Dam up and running there are serious water governance problems that need to be sorted out with an opportunity to learn from lessons in Indus Basin Water allocation regime.

## 7. Possible Future Work

How we are using some techniques outlined earlier, a quick review is undertaken on selected tools. Environmental-Economic Accounting for Water (SEEAW) is a standardized approach consistent with other types of Environmental-Economic Accounting techniques. It provides a valuable information base for integrated development of planning and policy. The approach allows for segregation and elaboration of all environment-related flows and stocks of traditional accounts, linkage of physical accounts with monetary environmental accounts and balance sheets, assessment of environmental costs and benefits, accounting for the maintenance of tangible wealth, and elaboration and measurement of the indicators of environmentally-adjusted production and income.

Unfortunately, it is not being used in Pakistan. Though economy wide water linkages are being recognized, how much value addition of water is being made and what is its potential is anyone's guess. Pakistan which was once a cotton economy is now cited as a water economy[2]. We can add value all along the water supply chain, according to Hisaar Foundation report, the water economy of Pakistan should obtain one billion dollar of output per million acre feet (MAF) of water and agriculture alone can add 1.5 billion of value annually instead of present 1 billion (see figure for other possible potential benefits). There is no exact accounting on how much we are losing, as many as 2.2 million hectares of land forming 13 percent of the cultivated area in Pakistan suffers from an acute problem of water logging and salinity, i.e., water table is less than 5 feet from the normal surface level.[3] Despite the government's effort to resolve the problem through an expansive network of public tube wells under the salinity control and reclamation project (SCARP), the problem seems to have worsened over time. The accounting

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1 Pakistan Irrigation and Drainage: Issues and Options, (1994). World Bank (1993), Agriculture Operations Division Afghanistan, Maldives, Pakistan and Sri Lanka Department South Asia Region, Report No.11884-PAK.

2 Pakistan's National Water Policy Framework by Hisaar Foundation (2016). Think Tank on Rational Use of Water.

3 Pakistan Irrigation and Drainage: Issues and Options, (1994). World Bank (1993), Agriculture Operations Division Afghanistan, Maldives, Pakistan and Sri Lanka Department South Asia Region, Report No.11884-PAK.

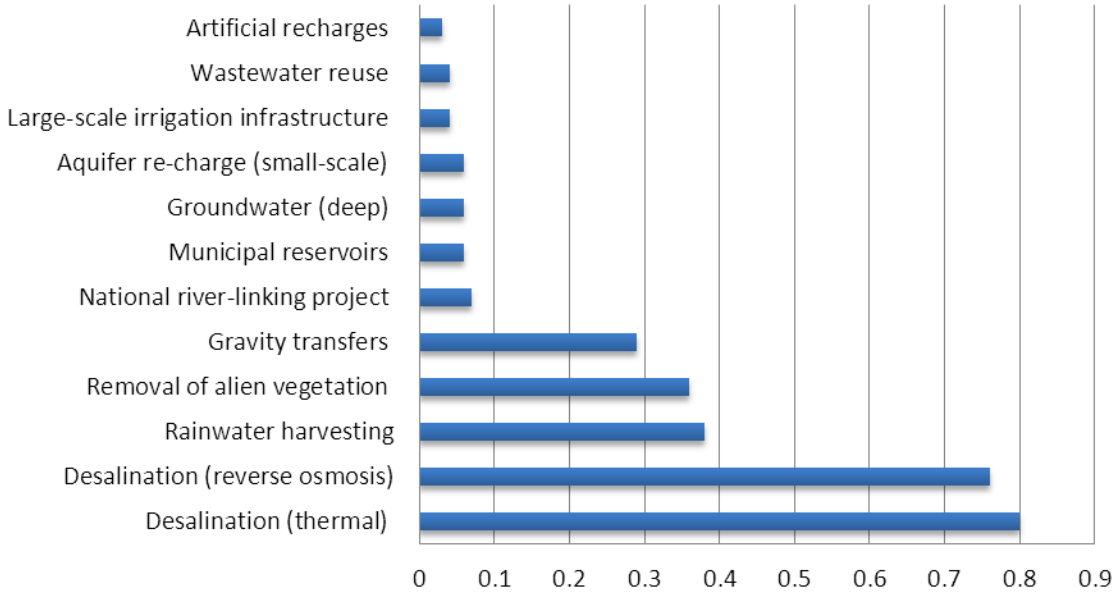
process requires a huge amount of data. According to FAO[1] , one limitation of SEEAW methodology is that it focuses on stocks rather than flows.

## 7.1 Water availability cost curve

The water availability cost curve is often used as a planning and negotiation tool. The approach provides information on what the potential cost would be of adopting a set of policy options that entails the strategy to close the excess demand gap. The unit cost of each measure in the future is uncertain, and they are likely to differ by country. The options to cope with water scarcity can be divided between supply enhancement and demand management. Supply enhancement includes increased access to conventional water resources, re-use of drainage water and wastewater, inter-basin transfers, desalination, and pollution control. The demand management options include improving agriculture practices (including crop variety), reducing irrigation area and increased reuse of irrigated agricultural land. Demand management options can save sizable amount of water at a least cost, a win-win scenario given budgetary constraint faced by many countries of the region. Pakistan happens to fall in this category.

The case of India shows that closing the water deficit by 2039 through traditional supply measures would be quite expensive and would face a steep marginal cost curve, due to the ceiling price set by expensive technologies such as desalination. As figure 2 shows, many of the supply measures required to close gap are demand management options that come at a cost between $0.05-0.10/ m3 —with the most costly measures largely supply side options could reach $0.50/m3 or more.[2]

***Figure 2:*** India Supply measures face a steep marginal cost curve, $/ m3




***Sources:*** A. Ahmad, Paper on Water Policy Issues and Options: Some ideas to use Economic Tools to close the Growing Supply and Demand Gap, 1st Technical Session of Engineering Congress Pakistan Engineering Congress, 73 Annual Session, 2005.

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1 FAO (2017), World Water Council members-driven initiative on “Water Accounting for agriculture”. Online discussion on “Water Accounting & Water for Food”.

2 2030 Water Resources Group (2009). Charting our water future. Economic framework to inform decision-making. Mc Kinsey & Company. 185 p.

## 7.2 Participatory Groundwater Monitoring

The participatory groundwater management system is a user driven tool, it involves local stakeholders to regulate and manage water resources in a sustainable way. The case of Pakistan is well explained by Arif Anwar [1], *successful community based groundwater monitoring program requires collection of representative samples in relevant locations in a timely manner. It hinges on the involvement of key stakeholders—farmers and farmer organizations—in the monitoring process. The scientific kits are complex and expensive to install, needing technical supported by the government. Data is collected monthly by farmer organizations, a cheaper method than the government collecting data. Government buys the data from these farmer organizations. Permanent sensors give data on water temperature, water-level elevation and depth, and water quality, which can be turned into digital photographs presenting data in an informative format.*

India provides a good example of a success story that can be of interest to the regions/countries facing groundwater depletion. Andhra Pradesh Farmer Managed Groundwater Systems (APFMGS) offers useful lessons on participatory groundwater management. Gram Panchayat is held responsible for local rural water supply, environment, agriculture, and irrigation.[2]

According IWMI report, at local level the community monitors hydrological information sharing, capacity building in groundwater management, hydrological data, prepare crop water budgets, and identifies over exploited aquifer zones. Strict regulatory actions included sanctions on excess water users and declared borehole drilling holidays to allow aquifers to recover. Water was used based on comparative advantage of a given crop and blended well with adoption of good agriculture practices. The typical institutional structure at a local level in is presented below. The core organizational component of the project is the groundwater management committee, and a village- level community-based institution comprising of all groundwater users. The committees are in turn grouped into hydrological units. The key is that its builds a strong link between hydrological and agronomic data, done through monitoring of rainfall and groundwater levels on one side and crop water budget on other.[3] The World Bank reports that the state of Andhra Pradesh has produced the first global example of large scale success in self-regulation of groundwater use. It incurs a cost of US$2,200 per village per year with benefits as farmers have doubled their income, while bringing their groundwater use close to sustainable levels. In many cases, farmers have voluntarily reduced their water use, while continuing to safeguard their crops[4]. The results being sustainable use of ground water, higher water productivity, higher profitability and crop diversification - creating a win- win outcome.

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1 Dr. Arif Anwar, Farmer Participatory Groundwater Monitoring: A Blueprint for Pakistan, Principal Researcher, International Water Management Institute (IWMI).

2 Participatory Ground Water Management in AP Draft Outline Of the legal framework emerging following discussions with the Principal Secretary, RD, Govt of AP on 10th August.

3 Sanval Nasim and Steven Helfand (2015). Optimal Groundwater Management in Pakistan's Indus Water Basin, working paper 034.

4 World Bank (2012), Feature Story, India Groundwater: a Valuable but Diminishing Resource March 6, 2012, www.worldbank.org/en/news/feature/2012/.../india-groundwater-critical-diminishing

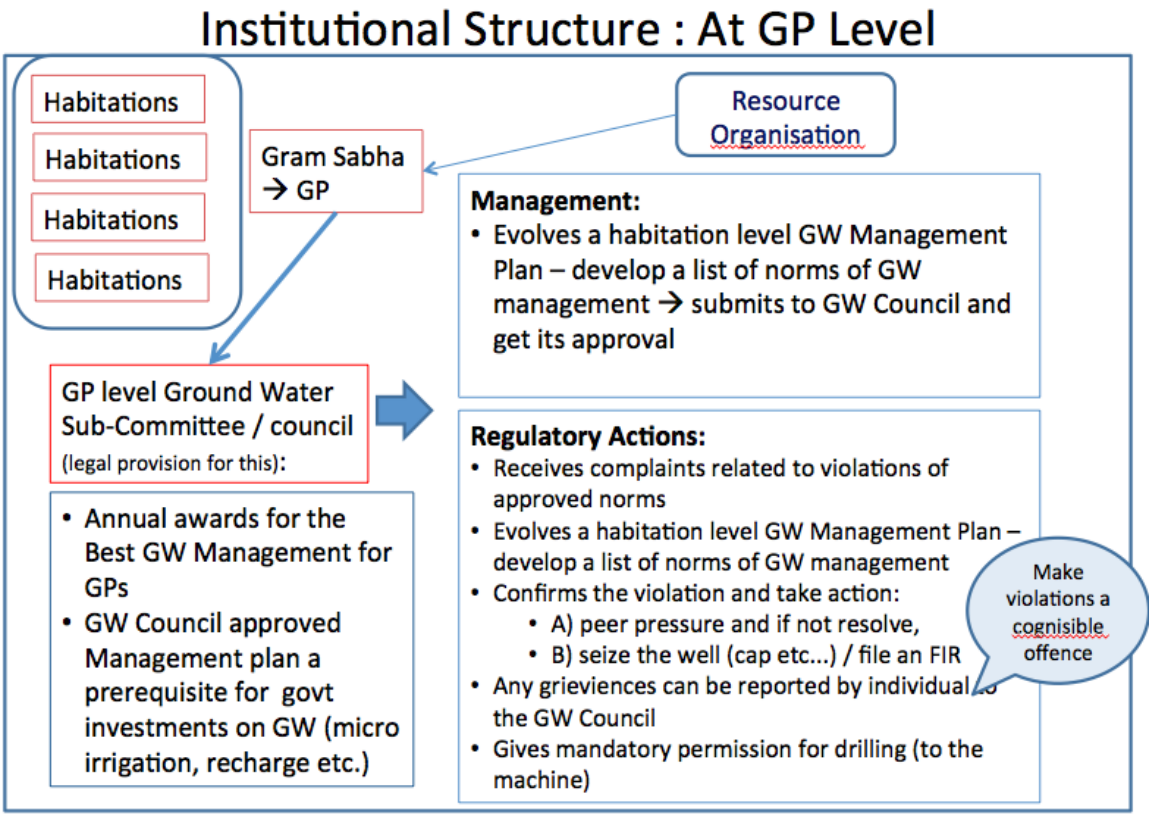



***Figure 3:*** Institutional Structure: At GP Level

## 7.3 Crop and Water Budgets

Water accounting at micro or farm levels can be done by preparing simple crop and water budgets. Water budgeting is key to the Participatory Hydrological Monitoring highlighted in above case and can be initiated on a pilot basis in Pakistan. The water budget tools include not only availability of data in terms of water stored in form of soil moisture, on calculating water requirements of different crops throughout their overall crop cycle but also preparation of a common crop plan in line with expected groundwater availability in Rabi (dry season) and resource inventory of groundwater availability at the end of Kharif (wet season). Crop–water budgeting at the beginning of the Rabi season is crucial for groundwater farmers. The crop budgets are also very useful to estimate water productivity in physical and value terms. It will assist to improve:

1. Improve productive efficiency (technology, water price, environmental awareness of water user).
2. Improve allocative efficiency (value of water in its alternative uses).
3. Trade in Virtual water (virtual water trade between water-scarce and water-abundant regions

A typical crop budget for Tomato grown in Oman is presented in the table below.

**TABLE: INPUT-OUTPUT DATA, GROSS MARGIN AND PROFIT PER FEDDAN**

**TABLE : INPUT-OUTPUT DATA, GROSS MARGIN AND PROFIT PER FEDDAN**

**جدول (A.1): بيانات المدخلات-المخرجات وحسابات هامش الإجمالى والربح للفدان**

| Activity | **Tomato (drip)** | | 18.53 | **0.11** | |
|---|---|---|---|---|---|
| Farming System | **drip** | | | | **نافوره** |
| Agricultural Zone | **El batanah** | | | | **الباطنة** |
| | Unit | الوحدة | Quantity | Unit Price RO. | Value RO. |
| Main product (compacted) | kg | كجم | 18530 | 0.065 | 1204.5 |
| Main product (scattered) | kg | كجم | | | 0.000 |
| **TOTAL GROSS OUTPUT** | | | **18530** | | **1204.5** |
| Seed/Seedling | gram | | 28.57 | 3.500 | 100.0 |
| Fertilizers: | | | | | 120.8 |
| Manure | kg | كجم | 8000.00 | 0.010 | 80.0 |
| Urea | kg | كجم | 100.00 | 0.108 | 10.8 |
| Super phosphate | kg | كجم | 100.00 | 0.14 | 14.0 |
| Potasiumm nitrate | kg | كجم | 0.00 | 0.00 | 0.0 |
| Compound | kg | كجم | 100.00 | 0.160 | 16.0 |
| Compound | kg | كجم | | | |
| Compound | kg | كجم | | | |
| Pesticides: | | | | | 179.8 |
| Herbicides | kg/lit | كجم | 28.000 | 6.42 | 179.76 |
| Fungicides | kg/lit | كجم | 0.00 | 0.00 | 0.0 |
| Nematocides | kg/lit | كجم | 0.00 | 0.00 | 0.0 |
| Leaf fertilizer | kg/lit | كجم | 0.00 | 0.00 | 0.0 |
| Hired Machinery | bales | بالة | 0.00 | 0.00 | 0.0 |
| Water Pumping | M3 | م 3 | 2500 | 0.035 | 86.9 |
| Pollen | No | عدد | 0.00 | 0.000 | 0.0 |
| Packages | boxes | عبوة | 1853 | 0.10 | 185.3 |
| Transport | ton | طن | 18.53 | 2.50 | 46.3 |
| Management | Man/month | رجل شهر | 0.06 | 115.61 | 6.9 |
| Permanent labor: | Man/month | رجل شهر | 1.16 | 75.36 | 87.7 |
| Energy | RO./month | شهر | 12.00 | 2.998 | 36.0 |
| Maintenance of pump and Irrig. System | RO/year | | 1.00 | 7.12 | 7.1 |
| **TOTAL VARIABLE COST** | **Dh.** | **درهم** | | | **856.8** |
| Depreciation of owned machinery | RO | ريال | 1.00 | 0.15 | 0.1 |
| Land rent | RO | ريال | 1.00 | 7.80 | 7.8 |
| Depreciation of Irrig. System | RO | ريال | 1.00 | 49.83 | 49.8 |
| Interest on Irrig. System | RO | ريال | 1.00 | 14.95 | 14.9 |
| **TOTAL FIXED COST** | **RO.** | **ريال** | | | **72.7** |
| **TOTAL COSTS/feddan** | **RO./fed.** | **ريال/فدان** | | | **929.6** |
| **TOTAL COSTS/ton** | **RO./ton** | **ريال/طن** | | | **50.2** |
| **Profit Account** | | | | | |
| **Gross Margin/feddan** | **RO./fed.** | **ريال/فدان** | | | **347.6** |
| **Net Profit/feddan** | **RO./fed.** | **ريال/فدان** | | | **274.9** |
| **Net Profit/ton** | **RO./ton** | **ريال/طن** | | | **14.83** |
| **Value Added/feddan** | **RO./fed.** | **ريال/فدان** | | | **435.3** |
| **Returns to Water** | **RO./$M^3$** | ريال/م3 | | | **0.145** |
| **Average product of water** | **Kg/$m^3$** | كجم/م3 | | | **7.412** |

***Table 1:*** Author's work in the preparing policy analysis matrix for agriculture sector in Oman, a FAO work, 2007

## 7.4 Remote sensing based water accounting

Remote sensing technology will allow rapid water accounting over large areas, such as a river basin. Although there are satellite images available without cost (from Google, for example), projects need cloud-free, high-resolution images that allow detailed visuals of an individual farm.

In India National Remote Sensing Centre (NRSC), is responsible for remote sensing satellite data acquisition and processing, data dissemination, aerial remote sensing and decision support for disaster management. It uses this technology among other sectors, in agriculture and water (figure 4 & 5). For agriculture it uses space technology and land-based observations for

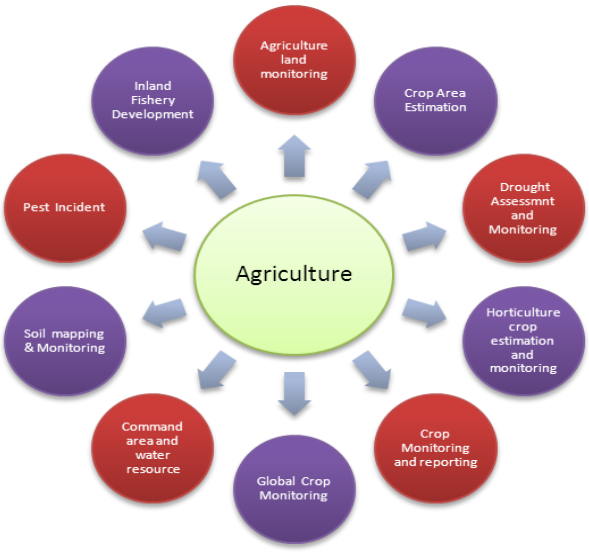



***Figure 4:*** Indian Remote Sensing for Agriculture

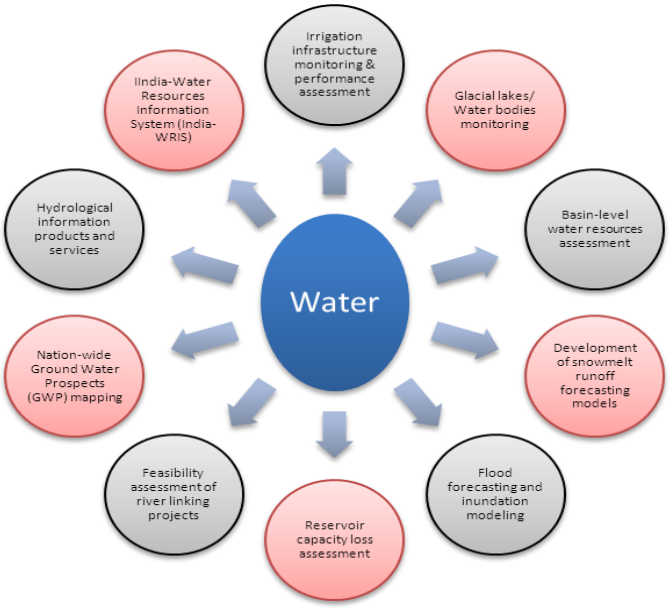



***Figure 5:*** Indian Remote Sensing for Water

1. Generating regular updates on crop production statistics and providing inputs to achieve sustainable agriculture.

2. For water accounting, satellite images that provide valuable information about the land use, irrigation infrastructure, hydrological flows and ground water profiles.

3. Geographical Information System (GIS) with customized decision support tools facilitate integrated analysis of different datasets and their visualization.

In Pakistan Space and Upper Atmosphere Research Commission (SUPARCO) is mandated to:

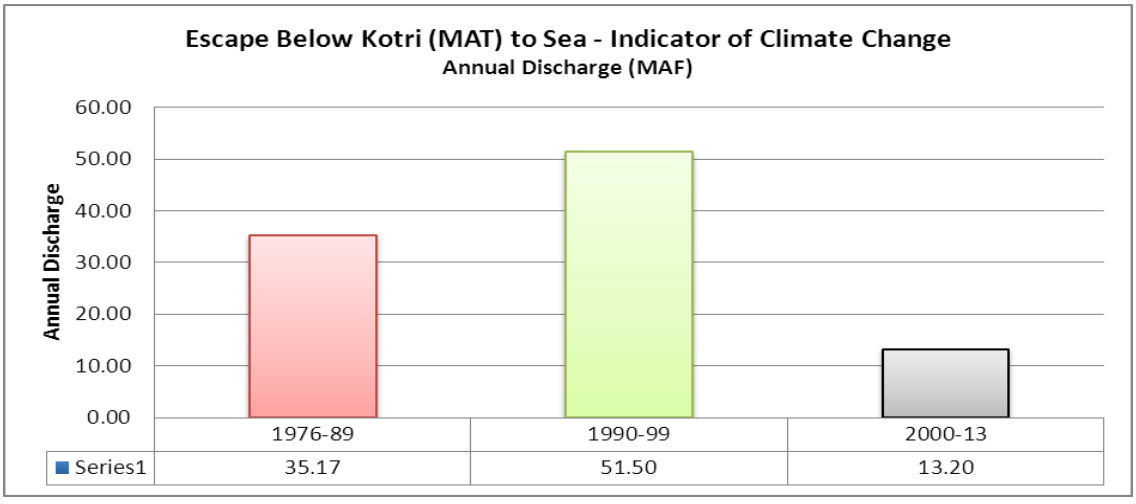
1. Undertake research and conduct pilot studies based on the applications of Satellite Remote Sensing (SRS) data and

2. Geographic Information System (GIS) technology to natural resources surveying, mapping and environmental monitoring.

3. Provides critical information on glaciers which are enormous reservoirs of fresh water and their melted water is an important resource which feed rivers in Pakistan. Glacier depletion, especially recent melting can affect agriculture, drinking water supplies, hydro-electric power, and ecological habitats. More so it might be contributing to climate change (see box 1).

**Box 1** – Impact of climate change on water resources of Pakistan

Water from glacier melt can also have a more immediate impact on Pakistan's economy that depends mainly on this source. Melting of seasonal snowfall and permanent glaciers has resulted not only in reduction of water resources but also caused flash floods in many areas of Pakistan. The impact is analyzed using fresh-water outflow to the sea as an indication of the total river flow for period 1976 t0 2013[1]. From the year 1975/76-1989/90 (post Tarbela) the average flow into the sea per year has been 35.17 MAF/year. However from the year 1990/91 to 1999/2000, this flow has been 51.50 MAF/year i.e. about 42 % greater. However the period 2000 to 2013 there is significant fall in outflow only 13.20 MAF, indicating possible impact of climate change (figure-1). It is already significantly affecting both water availability and agriculture sustainability. Using GIS technology provides a better information management on this key resource.



Another very useful study using RS technology[1] presents water accounts of Indus river basins using four sheets (i) a resource base sheet, (ii) an evapotranspiration sheet, (iii) a productivity sheet, and (iv) a withdrawal sheet. The results for one selected year (2007) showed that total annual water depletion in the basin (501 km3) plus outflows (21 km3) exceeded total precipitation (482 km3). The water storage systems that were affected are groundwater storage (30km3), surface water storage (9 km3), and glaciers and snow storage (2km3). Evapotranspiration of rainfall or "landscape ET" was 344 km3 (69 % of total depletion).

The results suggest that the Indus basin is nearly a closed basin in which more than 95 % of the available water is used. Agriculture depleted 297 km3, or 59 % of the total depletion, of which 85 % (254 km3) was through irrigated agriculture and the remaining 15 % (44 km3) through rainfed systems. Due to excessive soil evaporation in agricultural areas, half of all water depletion in the basin was non-beneficial. Based on the results of this accounting exercise loss of storage, low beneficial depletion, and low land and water productivity were identified as the main water resources management issues.

On the supply side, precipitation falls short of meeting the water demand. This leads to significant reduction in storage, especially groundwater storage (29.8 km3 in 2007). If the rapid decline in GW storage that occurred in 2007 is representative of other years, it may have major implications for the sustainability of the basin, considering the crucial role that GW plays in the basin's food security. While agriculture accounts for 59% of total water depletion it has a considerably low productivity, especially in terms of water use.

Therefore, to improve the situation and reach sustainability in the water-food nexus, it is wise to provide attention to water and land productivity improvement. This will in turn result in increased production with reduced water consumption. The results suggest that of the main opportunities for reducing water depletion is through decreasing wasteful soil evaporation in agricultural areas, particularly in irrigated land.

## 8. The LUMS Initiative

Center for Water Informatics & Technology (WIT), SBA School of Science & Engineering, LUMS, Lahore, Pakistan, is planning to launch initiatives to rationalize water use in agriculture (e.g. taking the cases of rice and sugarcane) and second to cross-pollinate water economics with water informatics technologies (e.g. remote sensing ) for on-farm water management.

## 9. Need to rationalize water use in agriculture

The link between water and food policy is getting the importance it deserves. A latest report by Water Resource Group 2030 on "Charting Our Water Future: Economic Framework to inform decision-making" supported by private sector clearly spells out that reforms in agriculture are key to solving the problem of growing excess water demand. The argument is simple, agriculture use the bulk of water (95% in case of Pakistan), rationalizing water use, starting with agriculture has the potential to save significant water and cost. Addressing the issues would requires among others policy options, improving productive (more crop per drop) and allocative efficiency (more value per drop) of water use in agriculture. To be more specific, using policy analysis matrix as a tool (extensively used by WIT researcher in other countries) it will look at how to enhance water productivity in general and agriculture sector in particular. It would initiate

[1] P. Karimi, W. G. M. Bastiaanssen, D. Molden, and M. J. M. Cheema (2013) Basin-wide water accounting based on remote sensing data: an application for the Indus Basin, Hydrol. Earth Syst. Sci., 17, 2473–2486, 2013 www.hydrol-earth-syst-sci.net/17/2473/2013/ doi:10.5194/hess-17-2473-2013© Author(s) 2013. CC Attribution 3.0 License

all important research, often being left out in our policy formulation process, how to rationalize water use in rice and sugarcane.

## 10. Use of Remote Sensing to monitor agronomic best practices and its economics

Timely availability of information in rice and sugarcane farming is vital for taking informed decisions on qualitative and agronomic issues. Remote sensing is instrumental in generating regular updates on crop production, statistic and providing inputs, which are not only economically crucial for rice and sugarcane growers and industry but also a prerequisite for sustainable crop cultivation. Moreover, it is inevitable to adopt smart agriculture for tailoring our modern technology practices, demand based irrigation scheduling, nutrients budgeting, advance crop planning and diversification not only at farm level but also at policy level in crop value chains. For example, the interest of the private sector is to make rice farming profitable and competitive. Remote sensing offers the possibility of not only monitoring the quantity and quality of input used but also in at least cost mode, thus making agriculture profitable.

WIT will deploy its expertise to help private sector track rice growth in selected area comparing three available technologies (1) Satellite based imaging (2) Unman Aerial Vehicle (UAV) based imaging and (3) in-situ electronic sensors. Based on each technology, detail crop water budgets would be prepared its economics would provide whole set of useful information for researcher and policy maker alike.

## 11. Conclusion

There is a need for proper accounting and auditing of water resources, taking stock beyond the simple accounting of volumes and flows, there is also a need to focus on issues relating to accessibility, uncertainty and governance of water.

On water accounting, Pakistan's water shortage cycle is getting worst over time as the Kharif season demands three times more water than Rabi, causing water shortage of 13.12 MAF, creating more storage at all levels is option has been delayed for long time at very high cost to the economy.

Three issues on water flow need to be more research and policy direction, concept of return flow is not well understood in terms of water saving, second water needed to preserve of ecology and environments be properly accounted for in water allocation at all levels, how cop with decline in water flow due to climate changes.

Modern harvesting techniques followed in different parts of the world can enable Pakistan to capture almost up to 90% of the total rainfall. This potential needs to be exploited on a more scientific basis to meet the future food and fodder requirements.

Ground water monitoring is another successful community based groundwater program requires collection of representative samples in relevant locations in a timely manner. It hinges on the involvement of key stakeholders—farmers and farmer organizations—in the monitoring process. This can be successful in Pakistan if the Government takes an active initiative in installing the appropriate technology (solar and IT based) and is also cost affective. Water accounting at micro or farm levels can be done by preparing simple crop and water budgets. Water Budgeting is key to developing a program that can be initiated on a pilot basis in Pakistan.

As water scarcity grows, the issue of water rights at all level would come. The debate should focus more on whether water rights should be tied to the land or can be traded separately. However, it is acknowledged that there are technical limitations to the system to work; it has to be redesigned to provide water on demand, and make sure social problems are not created through large scale sale of water rights by the small farmers.

Remote sensing technology will allow rapid water accounting over large areas, such as a river basin. Based on the results of this accounting exercise in Pakistan clearly identifies loss of storage, low beneficial depletion, and low land and water productivity as the main water resources management issues.

WIT center at LUMS has started some initiatives to rationalize water use in agriculture and the cross-pollination of water economics with water informatics technologies (remote sensing) for on-farm water management. We expect both private sector, international donors and Government to support this initiative at all levels.

## 12. Acknowledgement

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**Paper No. 775**

# A WAY FORWARD FOR FLOOD RISK MANAGEMENT IN PAKISTAN

**Dr. Qazi Tallat Mahmood Siddiqui**

# A WAY FORWARD FOR FLOOD RISK MANAGEMENT IN PAKISTAN

By

**Dr. Qazi Tallat Mahmood Siddiqui[1]**

## 1. INTRODUCTION

Despite many years of experience and highly developed techniques, flood even now continues to play havoc in most parts of the planet. The frequency of occurrence of floods in the region in general and in Pakistan in particular has considerably increased since past several years, because of global warming and rapid climate change. That is why Pakistan has faced series of flood events during the past 6-7 years i.e. since 2010, which indicates that flood has now become a regular feature in the country. This is exacerbated by the inadequate surface water storage capacity for absorbing flood peaks, chronic and increasing threat of encroachments in flood plains, inadequate discharge capacity of some of Barrages/Bridges, inadequate budget allocation under PSDP and Provincial ADP for execution of flood projects, weakness in flood defenses due to improper maintenance of existing flood protection structures and importantly a distorted natural drainage network.

There is a lack of effective coordination among institutions involved in flood management, caused in part by limitations of technical capacities such as dissemination of early warning, disaster preparedness measures, emergency response and structural measures for flood mitigation. The local communities do not have enough disaster preparedness information and there is lack of general awareness raising, sensitization and education of the masses regularly affected by floods, focusing especially on populations residing within the active flood plains along major, secondary and tertiary rivers.

The riverine floods are generally caused due to heavy concentrated rainfall in catchment areas of major & other rivers, which are sometimes augmented by snowmelt due to high temperature and generate exceptionally high flood flows in main rivers & their tributaries flowing across the country. The torrential rains are caused due to monsoon currents originating from Bay of Bengal and resultant depressions (strong weather system) often cause heavy downpour in the catchment areas of major and other rivers including hill torrents, which is sometimes augmented by the Westerly Wave from Mediterranean Sea.

## 2. THE NATURE OF FLOOD HAZARD

The riverine floods take hours or even days to develop, giving ample reaction time to locals to prepare/evacuate. However, flash floods generate quickly in mountainous regions with little warning/reaction time for locals. Flash floods can be extremely dangerous, instantly turning a babbling brook into a thundering wall of water and sweeping everything on its way downstream. Floods occur in all types of rivers and their tributaries. Localized flooding may be caused or exacerbated by drainage obstructions such as landslides, ice, debris, or dam failure. The increase in flow may be the result of sustained rainfall, rapid snow melting, Monsoon/Depression (Weather System) or tropical cyclones. Rapid flood events including flash floods, more often occur on smaller rivers, rivers with steep valleys or rivers that flow for much of their length over impermeable terrain. The cause may be localized convective precipitation

[1] Pakistan National Committee of ICID (PANCID)

(intense thunderstorms) or sudden release from an upstream impoundment created behind a dam, landslide or glacier.

Disaster experts classify floods according to their likelihood of occurring in a given time period. A hundred-year flood, for example, is an extremely large, destructive event that would theoretically be expected to happen only once every century. But this is a theoretical number. In reality, this classification means there is a one-percent chance that such a flood could happen in any given year. Over recent decades, possibly due to global climate change, hundred-year floods have been occurring worldwide with frightening regularity.

Climate change is considered to be a critical global challenge and recurring flood events have demonstrated the growing vulnerability to climate change. The impacts of climate change range from affecting agriculture to further endangering food security, to rising sea levels and the accelerated erosion of coastal zones, increasing intensity of natural disasters like floods & droughts, species extinction and the spread of vector-borne diseases.

It is generally recognized that complete prevention from floods is humanly impossible, but protection from flood is feasible and is a vital necessity. By proper planning, means can be devised to harness the fury of floods to safeguard human life and property. Devoid their destructive power, floods can be used in the service and the welfare of a community.

## 3. FLOODS IN PAKISTAN

Pakistan is a country with diverse type of land and fluctuating pattern of climate. Climate is usually considered hot and dry in Pakistan but it has shown significant obvious variations in last few years. Many districts and urban centers located along the rivers banks are ever on a great risk to confront with various types of floods i.e. riverine flood, flash flood and urban floods particularly in Punjab & Sindh provinces. The floods cause damages to hundred thousand acres of fertile agricultural lands, standing crops and affect adjoining abadies with monetary loss in billions of rupees. Major direct flood damages in the country are caused to agricultural lands, standing crops, urban and rural abadies, besides, other private & public property.

The riverine floods are generally caused due to heavy concentrated rainfall in the rivers catchments, during monsoon season, which is sometimes augmented by snow melt flows. Monsoon currents originating from Bay of Bengal and resultant depressions (weather system) often result in heavy downpour in the Himalayan foothills, which occasionally generate destructive floods in main rivers and their tributaries. Sometimes exceptionally high flood flows in major rivers are generated due to formation of temporary natural dams by landslide or glacier movement and their subsequent collapse. Flooding of the Indus River and its tributaries represents the greatest hazard in Pakistan. Floods occur usually in summer season (July - October). Therefore, damages to agriculture sector are mainly to the standing Kharif crops. However, in some cases the inundated lands do not dry up in time and ultimately affecting sowing Rabi crops.

The major rivers (Indus, Jhelum, Chenab, Ravi, Sutlej) and secondary rivers (Kabul, Swat etc.) cause flood losses by inundating low lying areas round the rivers bed by damaging irrigation and communication network, besides, land erosion along the rivers banks. In the upper part of the Indus Basin (Punjab & Khyber Pakhtunkhwa), floodwater spilling over the high banks of the rivers generally turns back to the main rivers channel. In the lower parts of the country i.e. Lower Indus Basin (Sindh province), River Indus is flowing at ridge i.e. higher elevation than adjoining lands. That is why flood embankments have been provided along both sides of the river. The flood water, if breaches the embankments do not return to the main river channel. This largely extends the area and period of inundation resulting in more damages to abadies, standing crops and other private as well as public infrastructure.

## 4. HISTORY OF FLOOD EVENTS IN PAKISTAN

So far, Pakistan has faced 24 severe flood event i.e. 1950, 1955, 1956, 1957, 1959, 1973, 1975, 1976, 1977, 1978, 19981, 1983, 1984, 1988, 1992, 1994, 1995, 2010, 2011, 2012, 2013, 2014, 2015 & 2016, the 2010 floods were worst ever in the country. The floods of various magnitudes since 1950 to 2016 affected vast areas in the four provinces including Gilgit-Baltistan, FATA & Azad Jammu & Kashmir. Owing to adverse impacts of climate change, in the recent years, vulnerabilities of communities to coastal & urban flooding have also increased. Flood damages are caused mainly due to riverine flooding in main rivers and flash floods in Secondary & Tertiary Rivers/Hill Torrents, Coastal flooding due to Cyclone & urban flooding due to torrential rains and inadequate storm drainage facilities, besides, GLOFs in northern parts of the country.

The unprecedented floods of 2010 were the worst floods in history of the country in which about 1985 people lost their lives, 1,608,184 houses were damaged/ destroyed, 17,553 villages were affected and total area of 160,000 $Km^2$ was affected. The Sindh province, particularly southeastern parts of the province was severely affected due to unprecedented rains and inadequate drainage facilities during Monsoon Season-2011. The torrential rains during 2012 rains/floods affected Southern Punjab, Sindh & Balochistan provinces. About 571 people lost their lives, 636,438 houses were damaged/ destroyed, 14,159 villages were affected and a total area of 4,746 Sq.km was affected. About 333 people lost their lives during 2013 rains/floods, around 8,297 villages with land area of around 4,483 Sq.km was affected. The floods of 2014, affected cropped area of about 2.415 million acres (9,779 square kilometers) affecting 4,065 villages, claiming about 367 lives, fully damaging 107,102 houses and population of about 2.600 million was affected. The floods 2015, affected more than 1.933 million populations, 4,634 villages (damaging 10,716 houses) and claiming about 238 lives all over the country. Flood flows triggered by torrential rains affected various parts of country, especially Chitral valley in Khyber Pakhtunkhwa, Punjab, Balochistan and some parts of Sindh & Federally Administered Areas (Gilgit-Baltistan, FATA & AJK). Moderate to heavy downpour in upper catchments of major rivers and their tributaries generated flood flows, which caused losses to human lives and damages to private and public infrastructure. 2016-rains/ floods affected 43 villages, claiming 153 lives, 113 injured and damaging 1452 houses. Pakistan has suffered a cumulative financial loss of more than US$ 38.171 billion during the past seventy years. Around 12,502 people lost their lives, some 197,273 villages damaged/ destroyed and an area more than 616,598 Sq.km was affected due to major flood events. The historical flood events experienced in the past and their damages are given in the **Table-1**.

***Table-1:*** HISTORICAL FLOOD EVENTS IN PAKISTAN

| Sr. No. | Year | Direct losses (US$ million) | Lost lives (No) | Affected villages (No) | Flooded area (Sq-km) |
|---|---|---|---|---|---|
| 1 | 1950 | 488 | 2,190 | 10,000 | 17,920 |
| 2 | 1955 | 378 | 679 | 6,945 | 20,480 |
| 3 | 1956 | 318 | 160 | 11,609 | 74,406 |
| 4 | 1957 | 301 | 83 | 4,498 | 16,003 |
| 5 | 1959 | 234 | 88 | 3,902 | 10,424 |
| 6 | 1973 | 5134 | 474 | 9,719 | 41,472 |
| 7 | 1975 | 684 | 126 | 8,628 | 34,931 |
| 8 | 1976 | 3485 | 425 | 18,390 | 81,920 |

| Sr. No. | Year | Direct losses (US$ million) | Lost lives (No) | Affected villages (No) | Flooded area (Sq-km) |
|---|---|---|---|---|---|
| 9 | 1977 | 338 | 848 | 2,185 | 4,657 |
| 10 | 1978 | 2227 | 393 | 9,199 | 30,597 |
| 11 | 1981 | 299 | 82 | 2,071 | 4,191 |
| 12 | 1983 | 135 | 39 | 643 | 1,882 |
| 13 | 1984 | 75 | 42 | 251 | 1,093 |
| 14 | 1988 | 858 | 508 | 100 | 6,144 |
| 15 | 1992 | 3010 | 1,008 | 13,208 | 38,758 |
| 16 | 1994 | 843 | 431 | 1,622 | 5,568 |
| 17 | 1995 | 376 | 591 | 6,852 | 16,686 |
| 18 | 2010 | 10,000 | 1,985 | 17,553 | 160,000 |
| 19 | 2011 | 3730 | 516 | 38,700 | 27,581 |
| 20 | 2012 | 2640 | 571 | 14,159 | 4,746 |
| 21 | 2013 | 2,000 | 333 | 8,297 | 4,483 |
| 22 | 2014 | 440 | 367 | 4,065 | 9,779 |
| 23 | 2015 | 170 | 238 | 4,634 | 2,877 |
| 24 | 2016 | 6 | 153 | 43 | - |
| 25 | 2017 | - | 172 | - | - |
| Total | | 38,171 | 12,502 | 197,273 | 616,598 |

## 5. FLOOD CONTROL PLANNING IN PAKISTAN

Flood management planning in Pakistan is being carried out to essentially cover the following three specific objectives:

i. To reduce or eliminate damages to existing properties;

ii. To prevent future increase in damages; and

iii. To mitigate the residual hazards.

In Pakistan, flood control planning is a complex problem and calls for great ingenuity and experience on the part of the planners. The nature of flood problems varies in each of the four provinces and federally administered areas due to varying physiographic, climatic, demographic, and socio-economic conditions. Even the characteristics of catchment areas of various rivers differ from each other. Flood problems relating to various provinces are given as under:

### Punjab

In Punjab, the flood protection marginal bunds have been generally constructed either to protect Headworks and other irrigation structures, or to safeguard certain towns, villages & adjoining agricultural lands. Due to general topography of the area, pre-determined breaching sections have been provided in the Right Marginal Bunds (RMBs) for operation for safety of Headworks/ barrages in case of exceptional high flood flows i.e. likely to exceed the designed

level. In order to protect areas from erosion, spurs have been constructed in critical reaches. These spurs have protected vast areas and in some cases even large tracks of eroded lands have been reclaimed.

## Sindh

The Indus River flows on a ridge in Sindh Province and generally, surrounding areas (outside the flood embankments) are lower than the river bed; hence, water once leaving the Indus River does not return to the main channel. The escaped water thus causes greater damage to widespread areas, and it persists for a longer period even after flood peaks are over *(Refer Tori Bund, M.S Bund breaches during 2010-Flood Season)*.

Sindh provinces is situated at tail end, hence, drain out all rivers and if flood protection measures adopted in the upper Sindh are not properly planned, severe damages are likely to occur in the Province. In most of the reaches, a double line of flood embankments has been constructed on both sides of the river from Guddu to few kilometers short of Arabian Sea. These flood embankments have been further compartmentalized to contain widespread inundation.

## Khyber Pakhtunkhwa

In Khyber Pakhtunkhwa, the floods are mainly due to flash flood flows in secondary rivers (Kabul, Swat, Panjkora, Kurram etc.) and major hill torrents/flood flow generating nullahs having steep bed slopes, which greatly increase flood velocity and severely erode the banks. In Khyber Pakhtunkhwa, mostly flood protection walls/embankments and short spurs have been constructed to save the areas from spill action and erosion.

Around 40 spurs having considerable shank length and Marginal Bund have been constructed along the right bank of Indus River “Chashma Barrage – Ramak Reach” for protection of D.I. Khan City and adjoining area from devastating flood flows of Indus River. A large number of spurs and flood embankments/flood protection walls in critical locations have also been constructed along Kabul, Swat, Panjkora, Kurram rivers and their tributaries including flood flows generating nullahs/hill torrents.

## Balochistan

Due to peculiar physiographic and climatic characterizes in Balochistan, the bed slopes of rivers and nullahs in Balochistan are very steep; hence, generate flash flood flows with high velocity causing banks erosion and inundations of low lying area along the banks of rivers and their tributaries. Mostly flood protection walls/embankments & short spurs have been constructed for protection of orchards, agricultural lands and abadies. Flood flows regulators/ flood diversion structures have also been constructed to dissipate the thrust of flood water and use the same for agriculture in the area.

## Gilgit-Baltistan, FATA & AJK

The bed slopes of rivers and nullahs in Gilgit-Baltistan, FATA and AJ&K are very steep. The flash flood flows generated in main rivers and their tributaries cause severe banks erosion. Flood Protection walls and short spurs in PCC & gabion crates are constructed in order to check the spill and erosive action of flood flows in rivers/hill torrents. The main purpose of such interventions is to provide protection to abadies, agricultural lands and other private and infrastructure.

Existing flood protection facilities are summarized in Table-2

***Table-2:*** EXISTING FLOOD PROTECTION INFRASTRUCTURE

| Sr. No. | Provinces / Federal Line Agencies | No. of Protection Works |
|---|---|---|
| 1 | Punjab | 1,185 |
| 2 | Sindh | 261 |
| 3 | Khyber Pakhtunkhwa | 784 |
| 4 | Balochistan | 260 |
| 5 | Gilgit-Baltistan | 30 |
| 6 | FATA | 209 |
| 7 | AJ&K | 13 |
| | Grand Total | 2,742 |

## 6. IMPACT OF GLOBAL WARMING & CLIMATE CHANGE ON FLOOD MANAGEMENT

Global warming causes climate change, which is a serious issue for the entire world. It is a serious threat to the third world as its impacts will not be felt equally across our planet. Developing countries including Pakistan are much more vulnerable to the impacts of climate change. The melting rate of glaciers in South Asia has increased, which may cause floods in Pakistan and surrounding countries in the coming years. Pakistan economy has faced significant losses due to environmental damages and degradations.

Pakistan is amongst the top ten countries on the globe experiencing frequent and intense climate change events such as floods, droughts, cyclones, heavy rains, extremely high temperatures, etc. The average global temperature has increased due to increasing concentrations of carbon dioxide and other greenhouse gases in the atmosphere for last many years. During the last century, it increased by 0.6 degree Centigrade and is likely to increase further by 1.0 $^{o}$C to 4.0 $^{o}$C till the end of the current century.

The most recent extreme climate events witnessed by Pakistan are floods hitting various parts of the country during the monsoon season. Pakistan has experienced flooding almost every year since 2010 which caused huge damages to life and property. The water security of the country is also threatened by the climate change. The increasing temperatures in the northern mountains of the country were likely to result in glacier melting, thereby affecting the flows of Indus River System.

## 7. INTEGRATED APPROACH IN FLOOD MANAGEMENT

Flood management plays important role in protecting people and their socio-economic activities in flood plains from flooding. The development in the river basins has been closely linked with successful implementation of flood control projects. In the past, exposure to flood risks has been handled largely through structural measures. However, strategies that rely largely on structural solutions unfortunately alter the natural environment of the river, which may result in loss of habitats, biological diversity and ecosystem productivity.

Further, structural approaches are bound to fail the moment an extraordinary or unforeseen event occurs. These traditional approaches, where the risks are merely transferred spatially, are likely to generate conflicts and inequities. Environmental degradation has the potential to threaten human security, including life and livelihoods, and food and health security.

This realization has recently led to calls for a paradigm shift from traditional flood management to Integrated Flood Management.

Integrated Flood Management (IFM) is a concept that addresses issues of human security against flood risks and sustainable development within the framework of Integrated Water Resources Management (IWRM). Such an integrated approach to flood management can play an important role in sustainable development and poverty reduction. Integrated Flood Management aims at minimizing loss of life from flooding while maximizing the net benefits derived from flood plains.

## 8. FLOODS AND THE DEVELOPMENT PROCESS

Historically, flood plains have been the preferred places for socio-economic activity as is evident from the very high densities of human settlement found there. Floods are a natural phenomenon, with both negative and positive impacts, and, generally, should not be considered a hindrance to economic development. Floods play a major role in replenishing wetlands, recharging groundwater and support agriculture and fisheries system, making flood plains preferred areas for human settlements and economic activities. Extreme demands on natural resources due to population growth have forced people and their property to move closer to rivers in many parts of the world. Further, flood control and protection measures have encouraged people to utilize newly protected areas extensively, thereby increasing flood risks and consequent losses.

Recurrent and extreme flooding, however, pose grave risks to development and have negative impacts on lives, livelihoods and economic activity and can cause occasional disasters. Flood disasters result from the interaction between extreme hydrological events and environmental, social and economic processes. These disasters have the potential to put development back by five to ten years, particularly in developing countries. The spiraling economic losses in developed countries also have given rise to grave concerns.

The balancing of development needs and risks is essential. The evidence worldwide is that people will not, and in certain circumstances, cannot abandon flood-prone areas. There is a need, therefore, to find ways of making life sustainable in the floodplains. The best approach is to manage floods in an integrated manner.

## 9. BACKGROUND AND SALIENT FEATURES OF NATIONAL FLOOD PROTECTION PLAN -IV

Pakistan has been frequently experiencing devastating floods in the past. Flood damages are caused mainly due to riverine flooding in major rivers and flash floods in secondary and tertiary rivers including hill torrents, besides, coastal & urban flooding. The 2010 floods were the worst ever riverine floods in the history of Pakistan, when floods caused over 1985 deaths, affected about 20 million people, 1.6 million houses were damaged/destroyed, 17,553 villages and land area of around 160,000 sq.km were affected. Keeping in view the future challenges, there is a dire need to adopt modern techniques for better flood management.

Since its establishment, FFC has so far prepared and executed three National Flood Protection Plans i.e. National Flood Protection Plan-I (1978-88), National Flood Protection Plan-II (1988-1998) & National Flood Protection Plan-III (1998-2008) through Provincial Irrigation Departments and Federal Line Agencies. After the large scale damages as a result of 2010 floods followed by 2011 & 2012-floods/rains, the need for investment in flood sector has gained importance. Federal Flood Commission initiated working on formulation of National Flood Protection Plan-IV (2016-25) in May 2013. The plan has been prepared in close consultation with all federal as well as provincial stakeholders. The planning strategy for development of National Flood Protection Plan-IV, in line with the current practices worldwide, is focused on

integrated flood management planning laying more emphasis on the non-structural measures, such as;

i. Reservoir operations
ii. Flood forecasting and early warning
iii. Flood risk zoning
iv. Watershed management
v. Flood proofing and insurance
vi. Disaster management and other measures

However, considering the uniqueness of Indus Basin with its large integrated network of dams, barrages and canals, which form the lifeline of agriculture economy of the country, the protection of this infrastructure as well as the irrigated area that it feeds and which has been the focus of previous three Flood Protection Plans, the maintenance, up-gradation and in some cases new construction especially in light of 2010 floods, has not been ignored.The above guidelines have been adopted while formulating the Plan. The estimated cost of the Plan with category wise breakdown into structural and non-structural measures is Rs 177.61 Billion and it provides for:-

i. Rehabilitation and enhancement of flood passing capacity of barrages and bridges
ii. Restoration of adequate conveyance capacity within the river and urban channels by removing bottle necks and encroachments
iii. Up-gradation of Flood Early Warning System (FEWS-Pakistan) for inclusion of
    a. Catchment area upstream of Tarbela dam,
    b. Updated existing river and floodplain geometry,
    c. Study on Radar calibrations,
    d. Enhancement in reliability of Quantitative Precipitation Forecast (QPF) through meteorological studies to enhance lead time, and
    e. Training of PMD professionals
iv. Expansion and up-gradation of existing gauging network, Radar network, telemetry network under PMD and WAPDA.
v. Repairing, strengthening and up-gradation of existing flood protection works
vi. Operation and Maintenance (O&M) of existing flood infrastructure
vii. Comprehensive studies for all existing breaching sections to ascertain their effectiveness and possible flow paths, flow depths, velocities and inundation extents of breach flood flows
viii. Formulation and implementation of watershed management policy for re-forestation, soil conservation and improvement in land use in the watersheds by undertaking necessary legislation at national level as well as provincial level

ix. A draft "River Act" for the rivers flood plains has been formulated during current NFPP-IV studies keeping relevant stakeholders on board for removing encroachments, permanent settlements and undue developments in the floodplains so that flood damages can be reduced

x. Capacity building and training of FFC, PIDs, NDMA, PDMAs, and other relevant agencies.

Category-wise summary for Cost of Structural and Non-structural measures is presented in Table-3:

***Table-3:*** FLOOD PROTECTION MEASURES UNDER NATIONAL FLOOD PROTECTION PLAN – NFPP-IV

| Description | | Cost in Million Rupees |
|---|---|---|
| **Structural Measures** | | |
| 1. | Completion of on-going Project/Liability | 751 |
| 2. | Construction of Proposed Flood Protection Works | 90,992 |
| 3. | Flood Management Structures Across Hill Torrents and Flood Generating Nullahs | 26,371 |
| 4. | Feasibility & Detailed Design Studies of Barrages and Hydraulic Structures. | 1,500 |
| 5. | Master Planning, Feasibility Studies, and Detailed Designing Studies. | 3,000 |
| 6. | Physical Hydraulic Model Study for Major Railway Bridges and Improvements of Existing Flood Protection Facilities of Pakistan Railway. | 450 |
| 7. | Physical Hydraulic Model Study for Selected Reaches of Major Rivers. | 200 |
| 8. | Measures for GLOFs & Land Sliding in Hilly Areas. | 1,000 |
| 9. | Remodeling & Proper Maintenance of Drainage System. | 9,763 |
| 10. | Coastal Flood Protection Works. | 1,622 |
| 11. | Flood Mitigation, Channelization and Execution of the Lai Nullah Project (Only Flood Component). | 16,000 |
| 12. | Studies for Proper Town Planning in Future and Improving the Existing storm Drainage System of Urban Areas. | 1,000 |
| 13. | Provision of Annual Funds under Provincial ADPs for Flood Fighting | 5,000 |
| Sub-Total (I) | | 157,649 |
| **Non-structural Measures** | | |
| 1. | Up-gradation & Expansion in Existing Flood Forecasting and Warning System of PMD | 4,205 |
| 2. | Up-gradation, Installation & Expansion in Existing Gauging System of WAPDA. | 2,297 |

| Description | | Cost in Million Rupees |
|---|---|---|
| 3. | Study to be conducted for removal of Encroachments in major Rivers & Hill Torrents and Procurement of LiDAR's. | 800 |
| 4. | Study and Implementation Cost of Development of Watershed Management in Upper Catchment Areas of Rivers & Hill Torrents. | 4,500 |
| 5. | Disaster Management Activities by NDMA, Rescue and Relief. | 6,500 |
| 6. | Feasibility/Technical Studies for Ramsar Sites | 30 |
| 7. | Capacity Building for All Institutions Dealing with Flood Management in the Country. | 1,380 |
| Sub-Total (II) | | 20,012 |
| Sub-Total (I+II) | | 177,661 |

## 10. CONCLUSION

It is the need of the hour to implement flood mitigation measures under National Flood Protection Plan in all the provinces and federal line agencies as a stitch in time saves nine.

**Paper No. ~~775~~776**

# PLANNING, DESIGN, CONSTRUCTION AND MONITORING OF FLOOD BUNDS

**Engr. Syed Mansoob Ali Zaidi**

# PLANNING, DESIGN, CONSTRUCTION AND MONITORING OF FLOOD BUNDS

By:

**Engr. Syed Mansoob Ali Zaidi[1]**

## 1. ABSTRACT

Flood protection Bunds have a long history dating back to the adoption by humans to living along the river streams to harvest the benefits of harnessing river flows, for irrigated agriculture and other miscellaneous uses. Initially the Bunds were in the form of micro earthen dividers between protected crops and river flow spills. With the civilization developing along the river streams, the Bunds sizes increased and the issues of design, construction, reinforcement and maintenance of Bunds were established which kept on improving all along. This paper presents the current methodology for planning, design, construction and monitoring of flood Bunds.

The design of Bunds provides the following geometrical parameters for construction:

a) The crest level;

b) The crest width;

c) Freeboard;

d) Side slopes;

e) Protective arrangements for flood levels water ways, velocity and scour

The crest level is established by taking into account, the flood levels, the rise in levels due to wave run-up, river set, wind setup and any emergency rise in flood levels. The crest width is mainly regulated by the requirement of crest width by transportation needs and space required for stocking of stone or other flood fighting materials. The side slopes are instrumental in providing the required seepage flow path for safety of the Bund. The design slopes are checked for stability against all loadings and seepage. Foundations soils if needed are strengthened by compaction, reworking, key trenches or seepage cut-offs. The shortcomings of original design are mitigated in the current design exercises.

## 2. INTRODUCTION

Flood protection by means of Bunds is a common and ancient practice followed globally. Historically, it may be said that in the olden days, the humans dwelled along the banks of rivers and for their livelihood they used the flood plains for growing food crops. This led to the growth of villages, towns and cities in the flood plains. In order to protect their land and dwellings, people started construction of Bunds locally.

A flood Bund is defined as “a man-made structure, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control, or divert the flow of water so as to provide protection from flooding”.

The development of the concept of `state` and the growth of civilization gave further impetus to the construction of flood Bunds for providing protection in longer stretches along the river/stream banks but for a considerable time these Bunds were constructed by private

[1] Ex-Secretary Irrigation and Power Department, Punjab/Senior Technical Advisor to M/s. National Development Consultants (Pvt) Ltd. Lahore

agencies aided by the local people. It was only about 200 years back that the construction of Bunds was taken over by the Government. The length of flood Bunds existing in Pakistan is over 6807 kilometres (4228 miles).

## 3. PLANNING

I. Topographic Surveys

Topographic survey in the area proposed for construction of the Bund is carried out to facilitate identification of Bund alignment.

II. Alignment

The alignment of a flood Bund should optimally address:

1) Follow the ridge (high contour line) to the possible extent.
2) Avoid as far as possible the weak foundation areas.
3) Keep the Bund at an optimal distance from the active river channels. In addition to technical conditions, the socio – economic political conditions have to be negotiated with.
4) Both anchor points should be at a safe elevation.
5) Ensure protection of the important salient intended to be provided with protection.
6) Avoid tortuous routes and sharp curves / bends.
7) Proximity of villages should be avoided if possible as it generally, encourages heavy trespass.
8) Try to avoid heterogeneous or other weak soils incorporation in the body of the bund.
9) If the above cannot be avoided, the nearest location of desired soils should be identified and put on record.
10) As an alternative, the design team should be apprised of the situation to enable them include proper measures in the design.

III. Investigations

A) Hydrological Studies

The hydrological studies are carried out for the reach of the river for determination of Probable Max Flow (PMF) with frequency analysis, and the flood water levels anticipated to be achieved the flood containing embankment. The max flood levels so determined form the basis of fixation of crest level of the embankment for a maximum flood flow of 100 years return period.

B) Geotechnical investigations to identify the formation soil characteristics and soil category available along the Bund alignment are to be carried out comprising:

a. Grain size analysis
b. Atterberg limits
c. Permeability

d. Soil classification

e. Bulk density

f. Compressive strength

g. Shear strength

C) Geophysical investigations

a. Electrical resistivity profiling to reveal soil stratification.

b. Ground penetrating radar can be used to detect cavities, weak soil strata and buried objects.

## 4. TYPES OF FLOOD BUNDS

1. Marginal Bunds for barrages and bridges
2. Bunds Protecting Strategic location
3. Bunds along open rivers for protection of crops and rural populace
4. Bunds along sea shores

## 5. DESIGN OF BUNDS

The geometrical parameters of Bunds determined through design exercises are:

a) The crest level

b) The crest width

c) Freeboard

d) Water side slope

e) Country / land side slopes

f) Protective arrangements for flood levels, water waves, velocity and related local scour

### 5.1 The Crest Level and Freeboard

While designing the Bunds, the crest level is to be kept higher than the designed flood level by a certain margin, dictated by the requirement of safety against over topping which can be caused due to various factors. The elements causing rise in water levels are wave action, wind setup, river set (or super elevation) at the curves. A suitable safety margin of 1 ft is to be added to the crest level determined by addressing the above elements.

The formulae used for calculations of various elements referred above are:

### 5.2 Wave Run-Up (or Ride)

The run-up (or ride) of a breaking wave in shallow water measured vertically above the mean water surface level may be estimated by using Hunt's equation:

$$\frac{R}{H} = \frac{K \tan \alpha}{\left(\frac{B}{T}\right)\left(\frac{H}{2g}\right)^{0.5}} \qquad [1]$$

where:

R = wave run-up (or ride) (ft)

H = wave height (ft)
K = surface roughness coefficient for the embankment slope (ft)
= 2.3 for smooth surface
= 1.8 for earthen surface
T = wave period (seconds)
$\alpha$ = embankment slope angle with horizontal (degree)
g = acceleration due to gravity (ft./sec$^2$)

## 5.3 Wind Set-Up

An appreciable rise in water level may be caused on the slope of reservoir or pond Bunds by wind action in shallow water. The wind set-up can be estimated by Zui-Der-Zee formula:

$$S = \frac{U^2 F}{1400\,D}\cos\theta \qquad [2]$$

where:

S = set-up above still pond level (ft)

U = wind velocity (miles/hour)

F = fetch (miles)

D = average water depth (ft)

θ = angle of fetch and wind (degree)

For combined effect of wind set-up and storm wave the total effective rise in water level is equal to wind set-up plus two thirds of wave run-up.

## 5.4 River set or super elevation

At a curve the deepest point of the river cross-section is near the concave (or outer) bank and the water surface is higher than that at the convex (or inner) bank. The “river set” may cause a super elevation of the water surface at the concave (or outer) bank and this can be estimated by Schoklitsch formula:

$$h = 2.3\frac{V^2}{g}.\log\frac{R_2}{R_1} \qquad [3]$$

Where:

h = river set (ft)

V = average velocity at the upstream straight reach (ft/sec)

g = acceleration of gravity (ft./sec$^2$)

$R_1$ = radius of curvature of convex (or inner) bank (ft)

$R_2$ = radius of curvature of concave (or outer) bank (ft)

To determine an appropriate freeboard above the design flood level, the wave run-up, wind set-up and river set are to be added and one foot additional be provided for contingent requirement as a factor of safety i.e.

F (freeboard) = Wave Run-up + Wind Set + River Set + 1.0 ft

Taking into consideration the current practices on freeboard, influenced by different hydraulic and wind conditions, the recommended minimum freeboard provisions for various flood protection structures are listed in the following table:

**Table 1**

| Flood protection structure | Hydraulic/Wind conditions | Recommended minimum freeboard in(ft)* |
|---|---|---|
| *Bunds along open river* | Current flow, sediment deposition, storm wave and wind set-up | 6.0 |
| Marginal Bunds (at barrages, and strategic locations) | Current flow, sediment deposition, storm wave and wind set-up | 7.0 |

*(*) The recommended minimum freeboard values include one foot additional freeboard for contingent requirement as factor of safety.*

## 5.5 Crest width

The crest width requirement is basically calculated with reference to the seepage flow through the body of a well compacted and well maintained embankment and is no more than 8'-12' but in view of the movement of official traffic and storage space required for stone and other flood management materials, the widths in practice are kept as the following:

a) Bunds along open-river, protecting agricultural lands and normal infrastructure / Bunds with wetting channels – 20 ft.

b) Bunds protecting component/strategic salient / with or without wetting channels – 25 ft

c) Coastal Bunds are designed as Bunds protecting strategic salients.

## 5.6 Side Slopes

Requirement for operation of earthmoving machinery and human access to the crest the best slope is 3:1. However in general practice the water side slope is fixed as 3:1 and the land side as 2:1 which normally satisfy the field needs.

## 5.7 Slope stability

The side slopes of an embankment must be stable under all conditions of construction, design flood discharge, rapid flood draw-down low flow level and earthquake forces. The stability depends on the strength of the fill soil and foundation characteristics.

Slope stability analysis requires a comparison between the state of stress in the soil and soil shear strength along an assumed failure plane. The detailed slope stability analysis is carried out according to Method of Slices, by using Simplified Bishop Method with relevant computer software. The needed information includes:

- Geometry of embankment
- Soil properties
- Design flood level and low water level of the river

- Phreatic line and pore water pressure
- Surcharge on the embankment
- Earthquake loads

### 5.7.1 River Flow Levels

The design high flood level and low flow level of the river are established for the particular embankment site to determine the saturation line in high flood condition and subsequent draw-down (low flow condition).

### 5.7.2 Phreatic line and pore water pressure

### a) Saturation line

The position of emergence of saturation line on the landside slope is not influenced by the permeability of the homogeneous fill material. The less pervious material takes longer to attain the steady state position, but the ultimate position of the phreatic line in all cases remains the same. Location of the phreatic line in the embankment is determined by using Casagrande's solution, illustrated in the following two Casagrande equations:

$$S = \sqrt{h^2 + d^2}$$

$$a = \sqrt{d^2 + h^2} - \sqrt{d^2 - h^2 \cot^2 \alpha} \qquad \textbf{[4]}$$

Where:

S = length of parabolic line of seepage (or phreatic line) from riverside water entrance point to the toe of landside slope (ft)

h = design flood depth on riverside slope (ft)

d = horizontal distance from water entrance point to the toe of landside slope (ft)

a = landside slope angle with horizontal (degree)

$\alpha$ = length of surface of seepage upward from the toe on landside slope (ft)

The phreatic line is the boundary between saturated and dry portion of the embankment (Figure-1).

### b) Pore water pressure

Pore pressure ratio ($r_u$),which is defined as the ratio of pore water pressure to vertical load at the base of each slice, is used for the slip circle analysis for all given conditions except for riverside slope at flood draw-down condition. For this condition only pore water pressure is used. The pore water pressure equation is:

$$u = r_u \gamma h \qquad [5]$$

where:

u = pore water pressure

$r_u$ = pore pressure ratio

$\gamma$ = total unit weight of soil (weight above and below the phreatic line)

h = depth of soil from top surface

The Simplified Bishop Method computer package may be used to compute the pore pressure ratio. For common soils the value of "$r_u$" ranges between 0.3&0.1

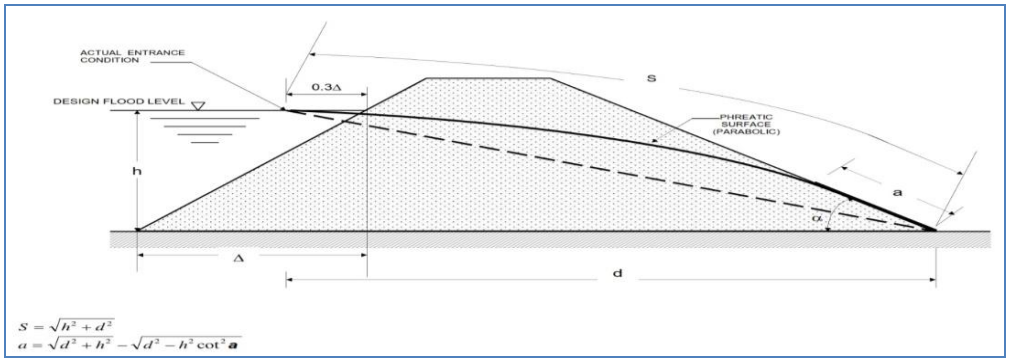



***Figure-1:*** Embankment phreatic surface (Casagrande's solution)

### 5.7.3 Surcharge

A surcharge of 975 kg/m$^2$ (200 lbs. per ft$^2$) on the top surface of embankment is applied for stability analysis.

### 5.7.4 Earthquake load

The acceleration due to earthquake is selected on the basis of fifty percent (50%) reduction for the horizontal and vertical components for calculation by the pseudo-static method.

The earthquake forces act at the cente~~r~~re of gravity of the slice.

The OBE values for particular scheme ~~is~~ are adopted on the basis of specific related earthquake zone criterion.

### 5.7.5 Critical conditions for stability analysis

The stability analysis of an embankment is carried out by using laboratory test values and characteristics of selected materials for determining the "Safety Factors" under critical conditions. The minimum safety factors against shear failure are:

***Table 2:*** Minimum safety factors against Shear Failure

| Condition | Minimum safety factor | |
|---|---|---|
| | Without earthquake | With earthquake |
| During and end of construction | 1.4 | 1.2 |
| Rapid river draw-down | 1.3 | 1.1 |
| River low flow level | 1.2 | 1.0 |
| Design flood | 1.5 | 1.2 |

*Source:* HASKONING and Delft Hydraulics

For overall stability of the embankment, during construction stage and under different river flood levels, the proposed slopes of the embankment is analysed for the stability under the following three loading conditions:

- Gravity
- Seepage (or pore water pressure)
- Earthquake

The potential failure surface is analysed for following four critical conditions against the pore water pressure by considering loadings with and without earthquake:

- End of construction
- Design flood level with steady seepage
- Flood draw-down
- Assuming fully launched stone apron

## 5.8 Foundation stability

The stability of side slopes of embankment depend upon the strength of the fill material and foundation characteristics. The foundation soil determines the rate of under seepage, formation of boils on the landside natural surface, the settlement of the embankment and stability of side slopes. If required, the foundation is treated for its strengthening / protection by:

- Reworking of the foundation soil
- Grouting to the required extent
- Inverted filter or geo-textile layer at the foundation of revetment / gabion based hydraulic structure
- Where required, provision of a suitable cut-off

Distribution of foundation pressures is also checked against the bearing resistance of the formation soils.

The investigation essentially provides the following information:

- Material type and zoning
- Permeability of river bed material
- In-situ density
- Hardness of cobbles and boulders
- Bearing pressure / capacity

## 5.9 Seepage Control

Bunds have to face seepage through the body or and the foundation:

### 5.9.1 Foundation Under-seepage

Without control under-seepage in pervious foundations beneath Bunds may result in:

a) Excessive pressure beneath an impervious top stratum on the land side

b) Sand boils

c) Piping underneath the body of the Bund

Under-seepage problems are most acute where a pervious substratum underlies a previous embankment and extends both riverside and landside of the Bund. Principal seepage control measures for foundation under-seepage are:

i) Cut off trenches

ii) River side impervious blankets

iii) Land side seepage berms

iv) Pervious toe trenches on the downstream toe

v) Pressure relief wells on the downstream toe

vi) Vertical wall cut-off's

vii) Non-woven go membranes

- **Cut offs**

  A cut off beneath a bund to block seepage through pervious foundation strata is the most positive means of eliminating seepage problems. Positive cut offs may consist of excavated trenches back filled with compacted earth usually located near the riverside toe. Since the cut off to be effective must penetrate 95% or more of thickness of pervious strata it is not economically feasible to construct cut offs where pervious strata are of considerable thickness.

- **Riverside Blankets**

  Bunds are frequently situated on foundations having natural covers of relatively fine grained impervious to semi impervious soils overlying pervious sands gravels. The surface strata constitutes impervious or semi impervious blankets when considered in connection with seepage control. If these blankets are continuous and extend inward for a considerable distance they can effectively reduce seepage flow and seepage pressure land side of the bund.

- **Land side Seepage Berms**

  If uplift pressure in pervious deposits underlying an impervious top stratum landward of a bund become greater that the effective weight of top stratum heaving and rupturing of top stratum may occur resulting in sand boils. The construction of land side berms can eliminate this hazard by providing:

  a) the additional weight needed to counteract these upward seepage forces and

  b) the additional length required to reduced uplift pressure at the toe of the berm to tolerable values

  Seepage berms may reinforce an existing impervious or semi impervious top stratum or if none exists be placed directly on pervious deposits.

### 5.9.1.1 Types of seepage berms

Four types of seepage berms have been used, with selection based on available fill materials, space available landside of the levee proper and relative costs.

1. Impervious berms. A berm constructed of impervious soils restricts the pressure relief that would otherwise occur from seepage flow through the top stratum, and consequently increases uplift pressures beneath the top stratum. However, the berm can be constructed to the thickness necessary to provide an adequate factor of safety against uplift.

2. Semi-impervious berms. Semi-impervious material used in constructing this type of berm should have an in-place permeability equal to or greater than that of the top stratum. In this type of berm, some seepage will pass through the berm and emerge on its surface. However, since the presence of this berm creates

additional resistance to flow, subsurface pressures at the levee toe will be increased.

3. Sand berms. While a sand berm will offer less resistance to flow than a semi-impervious berm, it may also cause an increase in substratum pressures at the levee toe if it does not have the capacity to conduct seepage flow landward without excessive internal head losses. Material used in a sand berm should be as pervious as possible, with a minimum permeability of 100 x 10-4 cm per sec. Sand berms require less material and occupy less space than impervious or semi-impervious berms providing the same degree of protection.

4. Free-draining berms. A free-draining berm is one composed of random fill overlying horizontal sand and gravel drainage layers (with a terminal perforated collector pipe system), designed by the same methods used for drainage layers in dams. Although the free-draining berm can afford protection against under seepage pressures with less length and thickness than the other types of seepage berms, its cost is generally much greater than the other types, and thus it is rarely specified.

### 5.9.1.2 Pervious Toe Trench

Where a levee is situated on deposits of pervious material overlain by little or no impervious material, a partially penetrating toe trench, can improve seepage conditions at or near the levee toe. Where the pervious stratum is thick, a drainage trench of any practicable depth would attract only a small portion of the seepage flow and detrimental under seepage would bypass the trench. Consequently, the main use of a pervious toe trench is to control shallow underseepage and protect the area in the vicinity of the levee toe. (Figure-2)

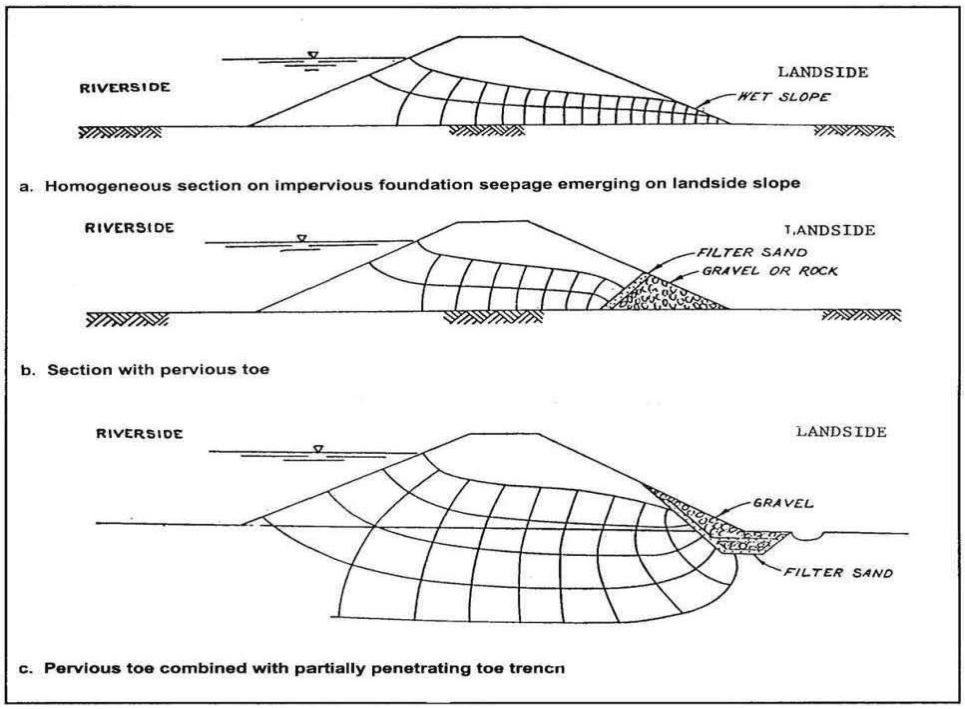



***Figure 2:*** Embankment with through seepage, USACE Manual EM 1110-2-1913

### 5.9.1.3 Pressure Relief Wells

Pressure relief wells may be installed along the land side of bunds to reduce uplift pressure which may otherwise cause sand boils and piping of foundation material. Wells accomplish this by intercepting and providing controlled outlets for seepage that would otherwise emerge uncontrolled land side of the bund. Pressure relief wells are used where pervious strata under a bund are too deep or too thick to be penetrated by cut off or toe drains. (Figure-3)

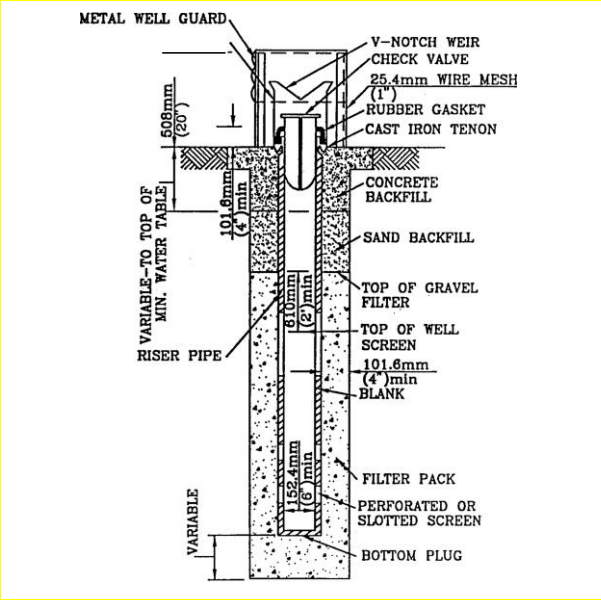



***Figure 3:*** Typical Relief Well (USACE EM-1110-2-1913)

### 5.9.2 Seepage through Embankments

Should the seepage through an embankment emerge on the landside slope, it can liquefy fine grained fill in the vicinity of the landside toe, cause sloughing of the slope, or even lead to piping under the sand or silty materials. Seepage exiting on the landside slope would also result in high seepage forces, decreasing the stability of the slope. In many cases, high water stages do not act against the Bund long enough for this to happen, but this can happen, if heavy precipitation occurs with the highest flood flows and levels. If landside stability berms or berms to control under seepage are required because of foundation conditions, they may be all that is necessary to prevent seepage emergence on the slope. On the other hand, if no berms are needed, landside slopes are steep, and flood stage durations and other pertinent considerations indicate a potential problem of seepage emergence on the slope, provisions should be incorporated in the Bund section such as an impervious cut-off of concrete or horizontal and/or inclined drainage layers or toe drains to prevent seepage exiting on the landside slope.

### 5.9.2.1 Pervious Toe Drain

A pervious drain will provide a ready exit for seepage through the bund and can lower the phreatic surface sufficiently so that no seepage will emerge on the land side slope. A pervious toe can also be combined with partially penetrating toe trenches. (Figure-4)

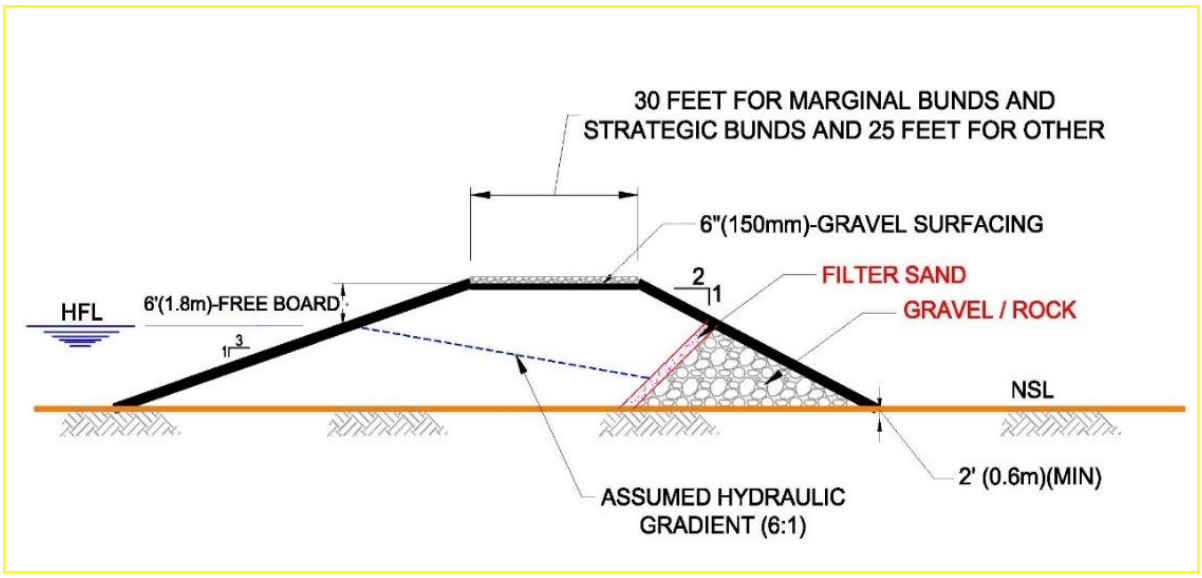



***Figure 4:*** Cross section of flood Bund with pervious toe

### 5.9.2.2 Horizontal Drainage Layers

Horizontal drainage layers serve the same purpose as a pervious toe but are advantageous in that they can extend further under the bund requiring a relatively small amount of additional material. They can also serve to protect the base of the bund against uplift pressures where shallow foundation under seepage is occurring. (Figure-5)

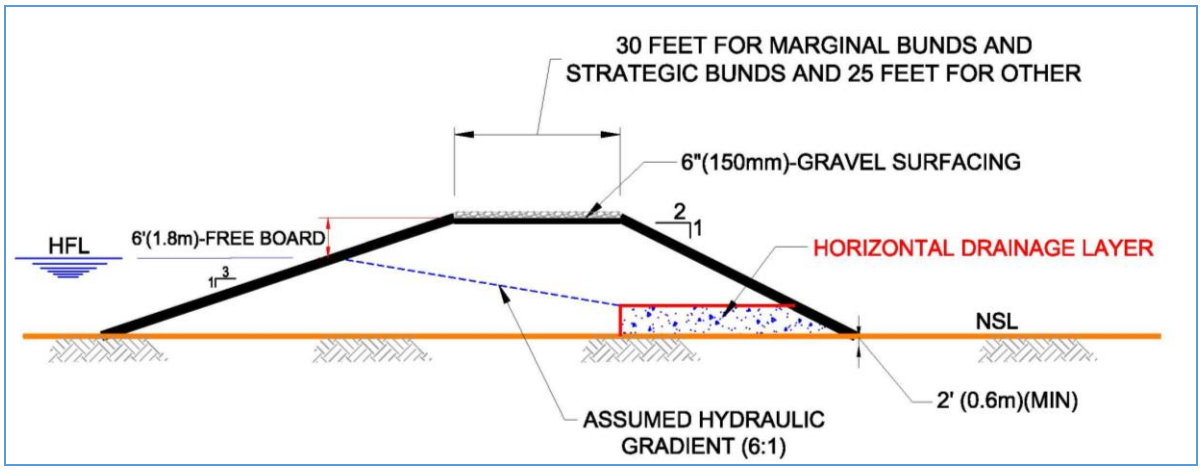



***Figure 5:*** X-section of flood bund with horizontal drainage layer

### 5.9.2.3 Inclined Drainage Layers

An inclined drainage layer is one of the most plosive means of controlling internal seepage and is used extensively in earth dams. It is rarely used in bund construction because of added cost but might be justified for short bund reaches in important locations where land side slopes may be steep and other control measures are not considered adequate and bund will have high water against it for prolonged periods. (Figures-6&7)

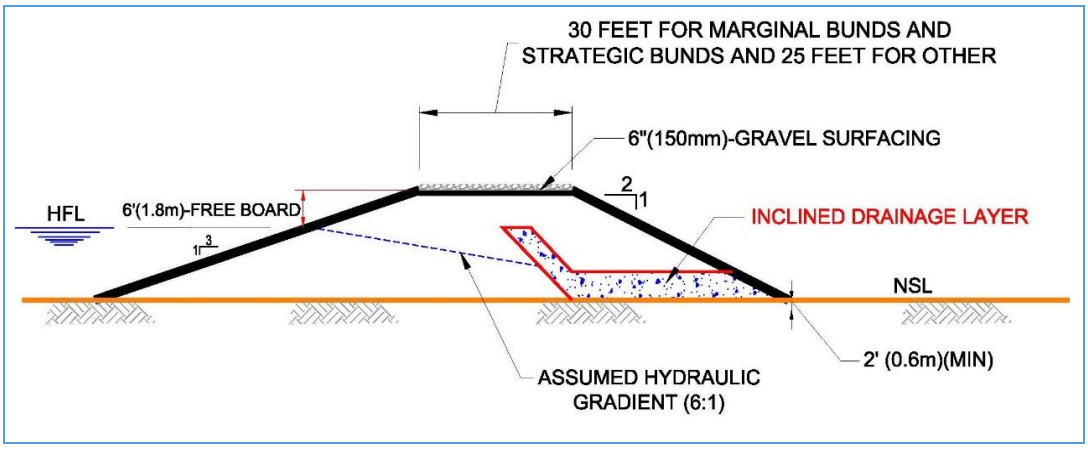



***Figure 6:*** X-section of flood bund with inclined drainage layer homogeneous embankment

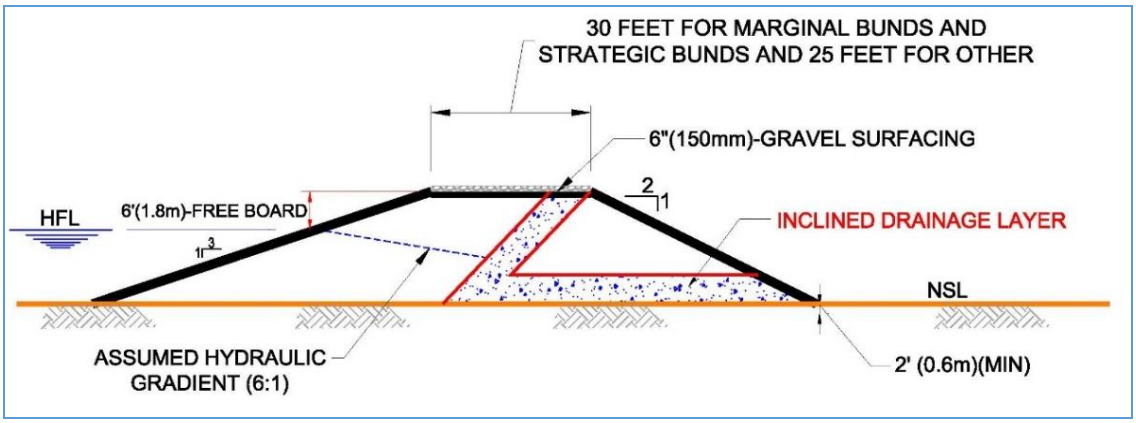



***Figure 7:*** Cross section of flood Bund with inclined drainage layers

### 5.9.2.4 Impervious Cut-off within the Body of Embankment

This is a good option where the embankment body contains on reliable and porous soils liable to rodent action. (Figure-8)

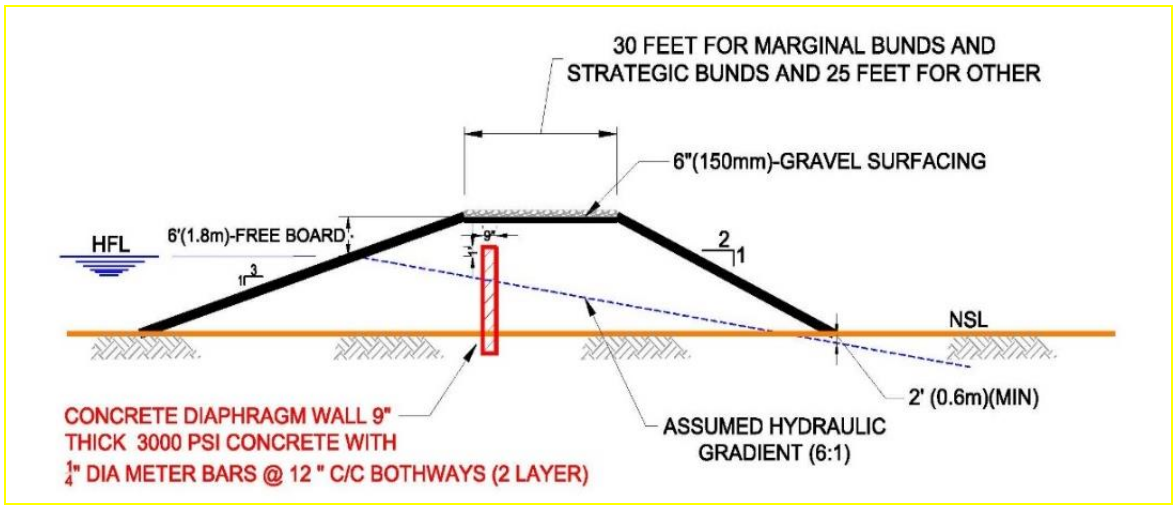



***Figure 8:*** Cross section of flood Bund with impervious wall cut-off

### 5.9.2.5 Design of Drainage Layers

The design of pervious toe drains and horizontal and inclined drainage layers must ensure that such drains have adequate thickness and permeability to transmit seepage without any appreciable head loss while at the same time preventing migration of finer soil particles.

### 5.9.2.6 Compaction of Drainage Layers

Placement and compaction of drainage layers must ensure that adequate density is attained, but should not allow segregation and contamination to occur. Vibratory rollers are probably the best type of equipment for compaction of cohesionless material although crawler tractors and rubber-tired rollers have also been used successfully. Saturation or flooding of material as the roller passes over it will aid in the compaction process and in some cases has been the only way specified densities could be attained. Care must always be taken to not over compact to prevent breakdown of material or lowering of expected permeabilities.

After the size of flood embankment has been established, it is tested against hydraulic gradient and if found necessary, back berm (or pushta) is provided to keep the hydraulic gradient within the cross-section. The hydraulic gradient is an assumed line of saturation in the embankment body. Normally, for loamy soils with natural compaction, it is assumed as 6:1 and it should lie at least 2 ft below NSL at the toe of the Bund. A minimum cover of 4 ft is provided above hydraulic grade line in made up fill. (Figure-9)

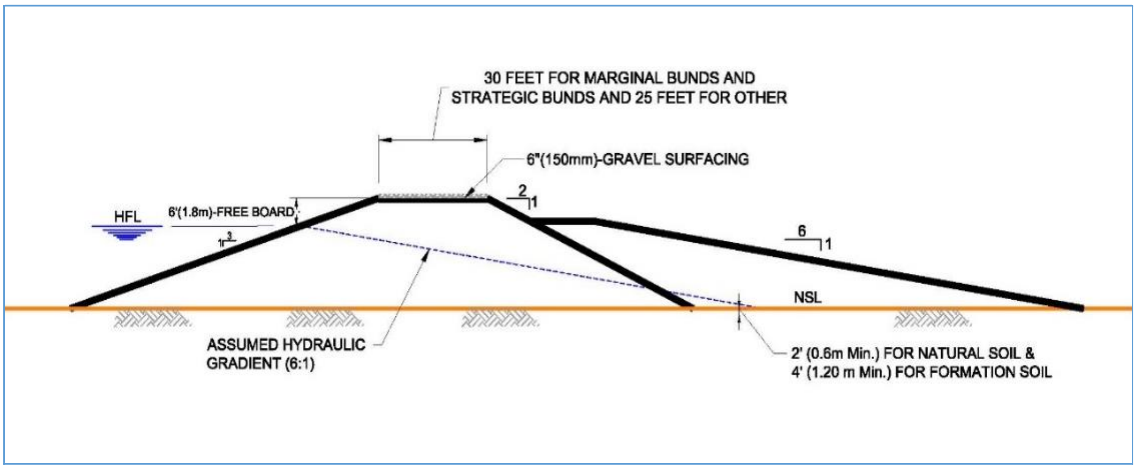



***Figure 9:*** Cross section of flood Bund with supportive back berm (pushta)

For embankments that remain in contact with flood water for a prolonged period, the phreatic line will be established for designing of the embankment to the phreatic line concept.

The embankments can thus be divided in the following two categories:

- Embankments which come in contact with the flood water for short duration.
- Embankments which remain in contact with the flood water for long periods.

For the first category, the embankments should be designed on the basis of hydraulic gradient line. The second category of embankments will be designed by ensuring stability of slopes and preventing their failure through piping.

## 6. SLOPE PROTECTION

The best form of protection system for the Bund slopes against waves and current attack is a properly designed protective layer of hammer dressed stone pitching underlain by transition spall and filter layers. A common requirement for protection is the bank body having a stable

slope. This slope is normally dictated by the angle of repose and physical characteristics of the bank material which influence the slope stability. Mostly it is not feasible to flatten the bank slope to satisfy soils stability criteria. For a steeper bank slope, a stone protection has to be provided on the slope. It provides the strength and stability required together with the flexibility, not weakened by slight movement of the embankment resulting from settlement or other adjustments. The local damage or loss is easily repaired by addition of stone where required.

For Bunds located along the barrage pond or bank of a wide river, the action of waves or current or a combination of both are considered for stone protection on the bank slope. The stone protection for all other hydraulic structures and river banks slope are designed against transverse forces due to the velocity and turbulence of current action.

For all type of bank slopes the stone size gradation of 18 kg (40 lbs) to 113 kg (250 lbs) is used. The stone material is composed of angular quarried stone eighty (80) percent of which by weight are equal and larger than $W_{50}$. The remaining twenty (20) percent quantity may be lighter rock ($W_{15}$ to $W_{30}$) to fill the voids between larger stones.

## 6.1 Protection of Upstream Slope

One of the major causes of failure of earthen Bunds is the damage caused by wave action, during floods when the embankments hold substantial depth of spill water. Protection of the Up Stream slope, therefore, becomes necessary for safety of the embankment. The methods employed for protection of Up Stream slope are introduced in the following paragraphs:-

## 6.2 Temporary (Limited Duration) Measures

### 6.2.1 Khaji Mats

Khaji Mats are stitched together with 6 inches over laps and the large mat so prepared is laid on the bank slope and anchored in place with small stakes (Figure 10&11). This method is used mainly during emergency and un-expected storms as delaying measure only and can at best simply reduce the damages. Sometimes gunny bags filled with earth are also used in extreme emergency.

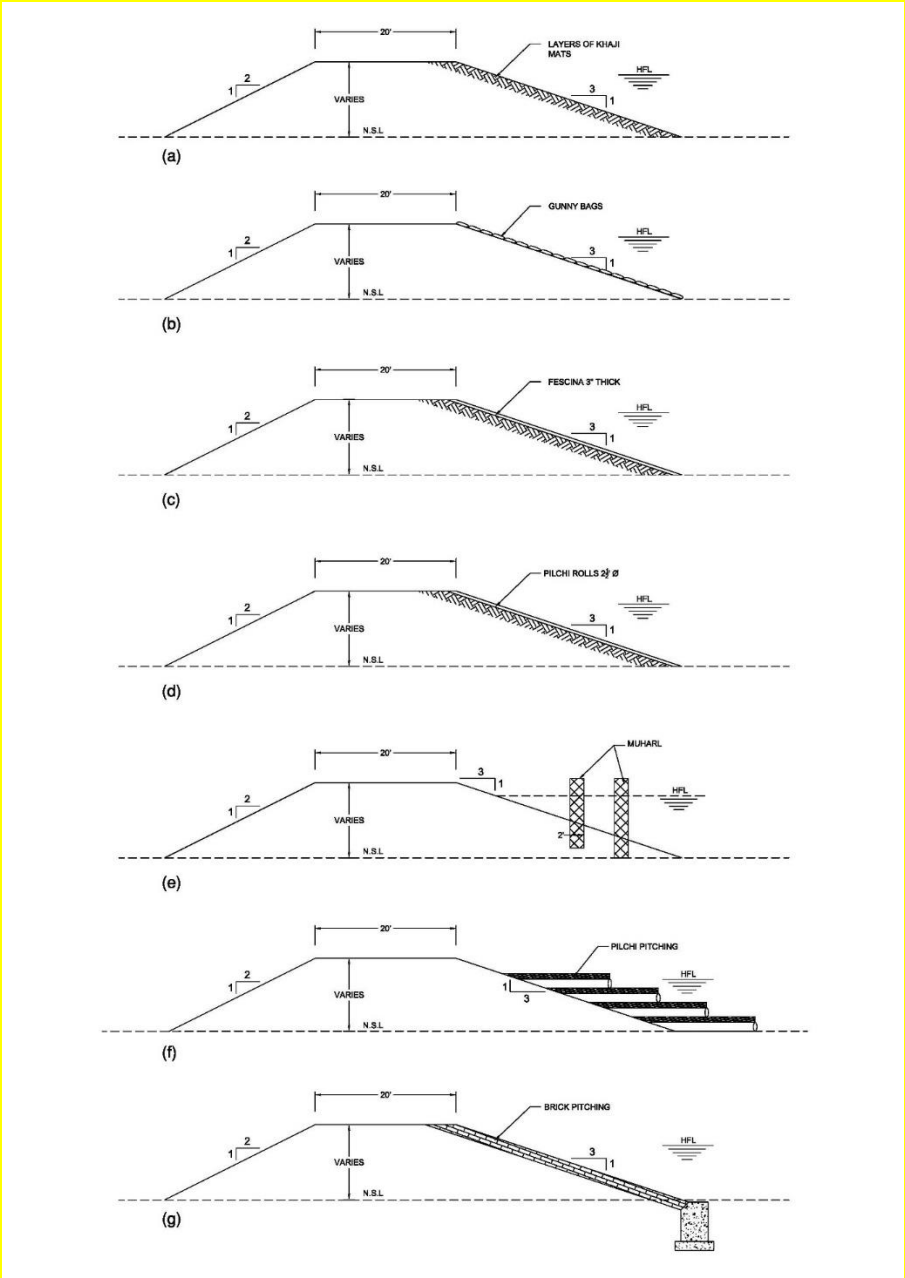



***Figure 10:*** Slope protection measures (temporary)

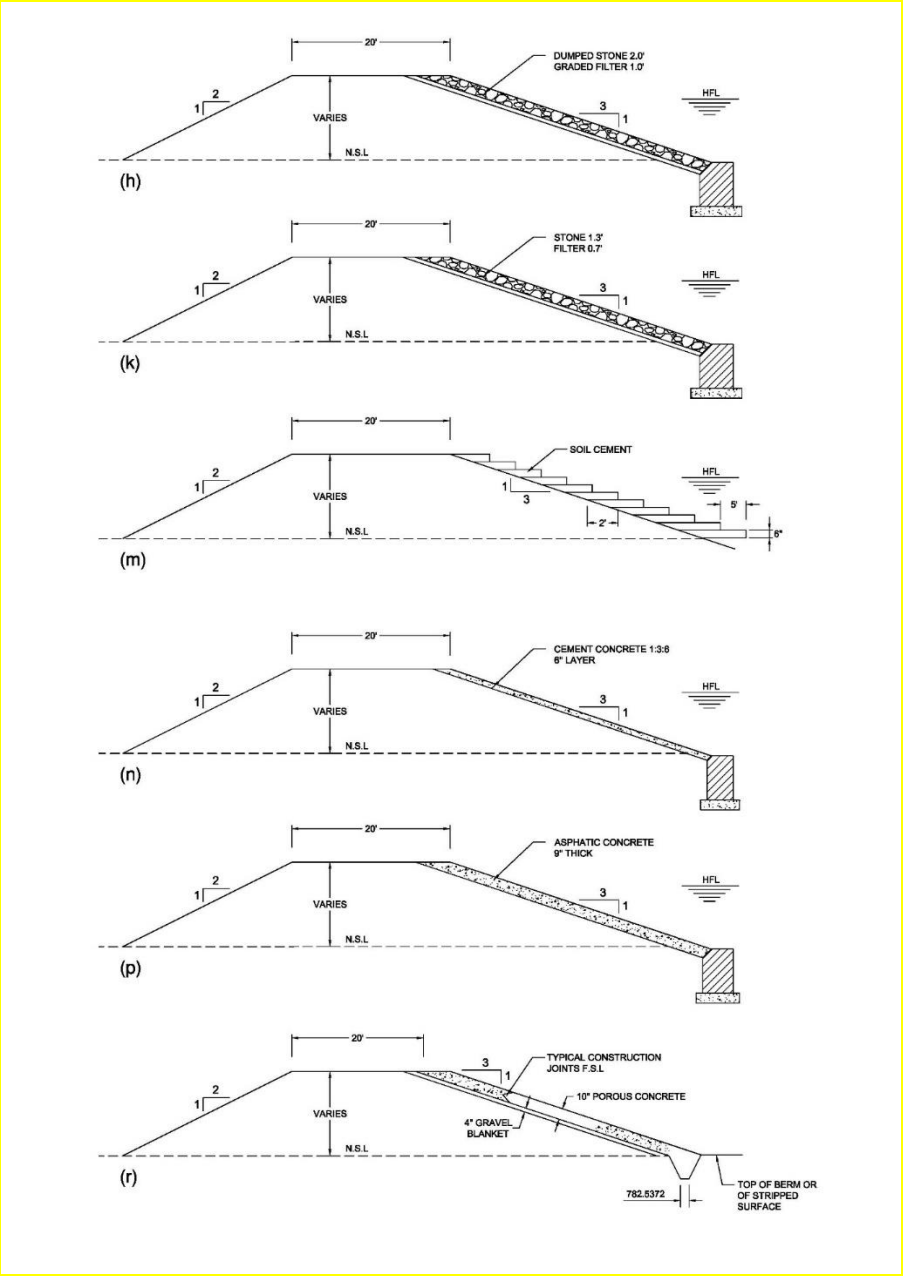



***Figure 11:*** Slope protection measures (permanent)

### 6.2.2 Fascine Covering

Pilchi, Sarkanda or brush wood is woven in the form of a mat and this mat is then laid on the bank slope and anchored with rope and stakes (Figure 10).This is quite a cheap protection but can only reduce the damages. Its life is also limited and it loses its usefulness just after one or two storms.

### 6.2.3 Pilchi Rolls

Pilchi, Sarkanda or Brush wood is tied into around bundles with diameter varying between 1 ft to 2½ ft, and these rolls are placed on the slope at water level parallel to the embankment and anchored with timber stakes (Figure 10). This type is more useful than fascine

but only for a few storms of moderate wind velocities. The author has seen such rolls broken and churned up into pulp due to pounding action of waves during high velocity storms.

### 6.2.4 Longitudinal Stakes and Bushing Protection

This consists of one or two rows of stakes with brush wood or pilchi rolls of 6 inch dia whettled in between (Figure 10). In Sind province this type is termed as MUHARI.

### 6.2.5 Pilchi Pitching (Revetment)

This is a semi-permanent type of protection and is widely used. However, the extent of protection afforded to the embankment slope sometimes does not justify the cost. It also gets damaged, (most of the time irreparably) under intense action and usually losses its usefulness in about 2 years time. (Figure 12)

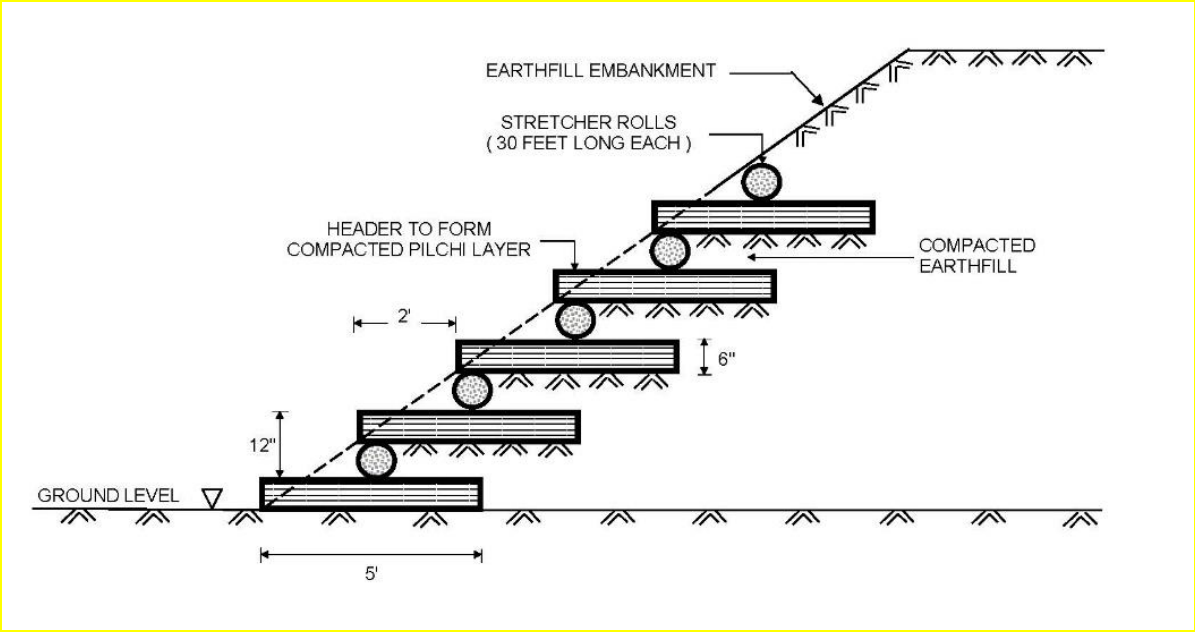



***Figure 12:*** Pilchi pitching

## 6.3 Permanent Measures

### 6.3.1 Brick Pitching

Brick pitching with brick laid on end over a flat course has been tried at some places. It deteriorates with time and lets the water seep through to earth backing. This creates hydrostatic pressure behind the brick pitching which results in buldging and finally spawling off and especially so when the spill depth reduces rapidly (Quick Draw Down Case).

A similar protection was provided to 2nd defence bund along right bank of D.G. Khan Canal at Taunsa Barrage, but it collapsed in the very first season, it had to face the full fury of wave action during monsoon of 1967, and was later replaced with Stone pitching.

The Brick pitching also has a disadvantage that the wave ride increases due to smoothness of the pitched surface.

### 6.3.2 Dumped Stone Rip–Rap

It is one of the best methods for protection of water face slopes. It consists of angular Stone dumped randomly over a properly placed graded filter consisting of rock material ranging between ¼ inch to 3 inch ring (Figure 11). The efficacy of this type depends on:-

1. Quality of Stone
2. Weight, size and shape of individual Stones. Angular but not flaky, pieces are better than rounded boulders.
3. Thickness of the protection and the slope on which laid.
4. Behaviour of the filter behind the dumped Stone.

The individual stones must be of sufficient weight to resist displacement by wave action which is not necessarily a function of the height of the embankment. It is wrong to suppose that large size stones are needed only on higher structures, while smaller size considering the wind velocity, direction, depth and the fetch causing wave action. The required weight of the individual pieces can be determined theoretically by methods which provide that the force exerted on Rip-Rap Stone cannot be greater than that of a current flowing at velocity equal to that of the particles in the wave. The stone sizes so calculated compare favourably with experience on prototypes studied by U.S. Bureau of Reclamation. The gradation limits of Rip–Rap Stone on 3:1 slope is given in the following table used by USBR:-

**THICKNESS & GRADATION LIMITS OF RIPRAP ON 3:1 SLOPE**

| Reservoir Fetch (Miles) | Normal Thickness (Inches) | Maximum Size | Gradation, Percentage of Stones of Various Weight (Pounds) | | |
|---|---|---|---|---|---|
| | | | 40 to 50 | 50 to 60 | 0 : –10 |
| | | | Percent greater than | Percent from - to | Percent less than |
| 2.5 & Less more than | 30 | 2,250 | 1,250 | 75-1250 | 75 |
| 2.5 | 36 | 4,500 | 2,250 | 100-2250 | 100 |
| Note:- 1) sand and rock shall be less than 5 percent by weight of the total riprap material.<br>2) The percentage of this size material shall not exceed an amount which will fill the voids in larger rock. | | | | | |

Sherad et al in their publication Earth and rock dams recommend the following data for rip-rap and filter beneath.

### 6.3.3 Recommended Riprap Design Criteria

| Maximum Wave Height (feet) | Minimum average Rock Size (D50) (Inches) | Layer Thickness (Inches) |
|---|---|---|
| 0-2 | 10 | 12 |
| 2-4 | 12 | 18 |
| 4-6 | 15 | 24 |
| 6-8 | 18 | 30 |
| 8-10 | 21 | 36 |

MINIMUM THICKNESS OF SINGLE LAYER FILTER UNDER RIPRAP

| Computed Wave Height (feet) | Minimum Filter Thickness (Inches) |
|---|---|
| 0-4 | 6 |
| 4-8 | 9 |
| 8-10 | 12 |

Dumped rip-rap is, however, very expensive and should be provided where absolutely necessary.

### 6.3.4 Hand placed Rip-Rap (Stone Pitching)

Stone pitching consists of Stone Laid by hand and packed properly over a graded filter similar to the one required for dumped rip–rap. The usual thickness adopted varies between 1.5 ft to 3 ft, 2 ft is the usual average adopted. The usual thickness of filter is 0.7 ft: and the balance 1.3 ft is occupied by top Layer of large stones. This is quite effective in protecting the earthen slope but calls for a larger free board due to increased wave run- up on account of smoother surface. (Figure 11)

### 6.3.5 Soil Cement Cover

Of late, soil Cement is being considered as an economical facing material for embankments where the cost of Stone protection is too high. A reasonable firm foundation is required so that deformation after placement is insignificant. It is generally placed and compacted in horizontal step–like layers of 8 ft: width. Each succeeding layer is stepped back by a distance equal to the product of the compacted layer thickness (Usually six inches) and the embankment slope (Figure 11). Soil containing 10 to 25% material passing No. 200 sieve (ASTM) can be safely used by adding one bag of cement per Cyd. of compacted layer.

### 6.3.6 Cement Concrete Paving

Concrete paving can be used successfully where the Sub Grade is suitable and settlement in the body or foundation of the embankment can be taken as negligible. Monolithic pavement behaves better than that laid in the form of panels, with expansion joints in between. The water finds its way through the joints to the back of the pavement and damages it through hydrostatic pressure. Settlement in the Sub Grade also results in cracks and similar failure. Usually a six inches layer suffices if the Sub Grade is firm. In case of panelled construction, all joints have to be sealed and panels reinforced. In such cases adequate provision for back drainage must also be made (Figure 11).

### 6.3.7 Asphaltic Concrete

Asphaltic concrete being comparatively flexible than cement concrete is a better substitute, but in this case also provision for back drainage is a must to eliminate up lift in case of rapid draw down. The usual thickness of asphaltic concrete is taken as 9 to 12 inches (Figure 11).

### 6.3.8 Porous Concrete Slab

Porous concrete has also been in use for quite some time. When properly made it is comparatively strong and almost equally pervious as crushed stone protection. It consists of coarse aggregate 3/4 inch to ¼ inch with one barrel of cement per Cyd. This is best suited for mild climates and where stone rip- rap is excessively expensive (Figure 11).

### 6.3.9 Design of Slope Protection

Till recently the structural measures for protection of slope of flood Bunds has been confined to hammer dressed stone pitching designed on the basis of the following relationships:

- General Recommendation by Punjab Irrigation Research Institute (IRI)

  Stone thickness = 2.5(ft) = 0.76 (m)

  Spall/Filter layer = 0.75(ft) = 0.23 (m)

  Total thickness = 3.25(ft) = 0.991 (m) say 1.0 (m)

- Spring (1903)

  Spring prepared a thickness table based on channel slope and grain size of the alluvium through which the river flows (refer Table 3).

***Table 3:*** Thickness of stone pitching

| **Fall per mile (in)** | 3 | 9 | 12 | 18 | 24 |
|---|---|---|---|---|---|
| **Sand classification** | Thickness of stone pitching (in) | | | | |
| Very coarse | 16 | 19 | 22 | 25 | 28 |
| Coarse | 22 | 25 | 28 | 31 | 34 |
| Medium | 28 | 31 | 34 | 37 | 40 |
| Fine | 34 | 37 | 40 | 43 | 46 |
| Very fine | 40 | 43 | 46 | 49 | 52 |

*Source:* Spring and Khosla (1903/1936)

- Gales dimensions

  Stone thickness = 3.25(ft) = 0.99 (m)

  Spall/Filter layer = 0.75(ft) = 0.23 (m)

  Total thickness = 4.0(ft)(1.2 m)

- C.C. Inglis Formula

  $T = 0.06(Q)^{1/3}$ **[**6**]**

  where:

  T = thickness of pitching (ft)

  Q = discharge (cusecs)

  This equation yields normal results for discharges up to 8,500 cumecs (300,000 cusecs) but the thickness increases abnormally for discharges above 11,300 cumecs (400,000 cusecs).

- Sethi Formula

  $T=K(Q/f)^{1/3}$ **[**7**]**

where:

T = thickness of pitching (ft)

Q = discharge (cusecs)

K = constant

f= Lacey's silt factor

This equation yields slightly high but consistent results subject to correct selection of the constant K.

Considering the merits and short- comings of the above methods and general river alluvium the recommendation of IRI (total thickness = 0.99 m (3.25 ft) appears to be most rational. This thickness has been provided at some spurs and has held very well.

Spring's table yields results similar to those of the IRI for rivers in plains with fine sediments and a ruling slope of 1 in 5000.

However, the design of slope protection is carried out for both velocity and wave wash effects.

### 6.3.10 Protection against flow currents

Two empirical methods by Isbash and Maynord can be used to determine weight and size of the stone in hand placed rip rap or stone pitching. The larger stone size ($D_{50}$) resulting from either method may be used.

### 6.3.11 Wave Attack Protection

For the embankment slopes which are subject to wave action, it is necessary to predict wave characteristics for determining the free board and design of stone pitching for embankment protection. First the wave characteristics are determined and then applied for establishing the protection stone size.

#### *a) Wave Run-up*

The wind velocity, duration and direction at the location of the Bund and adjacent topography affect the generated wave heights. The wave run-up is measured above the still water level that would theoretically exist if there is no wave action. Waves prediction methods are based on semi-empirical relationships, as discussed in the following paragraphs:

i. Wind Data

Normally, wind data to establish the design wind velocity over land ($U_{land}$) is obtained from the nearest meteorological station.

ii. Effective Fetch (Fe)

The characteristics of wind-generated waves are influenced by the distance that wind travels over the water surface in the fetch direction. The value of "$F_e$" ranges from 1.6 km (1 mile) to 16 km (10 miles) depending upon the maximum straight line length of water surface to which the face of the embankment is exposed.

iii. Significant Wave Height and Period

With the established "$U_{water}$" and "$F_e$" the significant wave height and period in deep water are computed from the following methods;

- Saville Method :

$$\frac{gH_s}{U_w^2} = 0.0026\left(\frac{gF_e}{U_w^2}\right)^{0.47} \quad [8]$$

$$\frac{gT_s}{U_w^2} = 0.46\left(\frac{gF_e}{U_w^2}\right)^{0.28} \quad [9]$$

where:

$H_s$ = significant wave height, the average of the highest 1/3 waves in the spectrum (ft)

$T_s$ = significant wave period, the average interval between successive crests or troughs of groups of the higher waves (s)

$F_e$= effective fetch (ft)

$U_w$= wind velocity overwater surface (ft/s)

g= acceleration due to gravity (ft/s$^2$)

- Bretschneider Method

$$H_s = 0.0555(U_w{}^2 F_e)^{0.5}$$
$$T_s = 0.50(U_w{}^2 F_e)^{0.25} \quad [10]$$

where:

$H_s$ = significant wave height(ft)

$T_s$= significant wave period( s)

$U_w$= wind velocity (knot)

$F_e$= effective fetch (nautical mile) (1.15 statue mile = 1.0 nautical mile)

Shallow water wave-height ($H_s'$) = 0.65 $H_s$

- Creager and Justin Method

$$H = \frac{F^{0.37}.U^{0.48}}{3.41} \quad [11]$$

where:

H = wave height (ft)

F= fetch (mile)

U = wind velocity (mile/hour)

The following table shows the wave heights (in ft) with reference to reservoir/ pond fetch in miles and wind velocity in mph.

***Table 4:*** Wave height versus fetch and wind velocity

| Fetch Km (mi) | Wind velocity Km (mi/h) | Wave height Meter (ft) |
|---|---|---|
| 1.6 (1) | 80 (50) | 0.82 (2.7) |
| 1.6 (1) | 120 (75) | 0.91 (3.0) |
| 4.0 (2.5) | 80 (50) | 0.98 (3.2) |
| 4.0 (2.5) | 120 (75) | 1.1 (3.6) |
| 8.0 (5) | 80 (50) | 1.13 (3.7) |
| 8.0 (5) | 120 (75) | 1.31 (4.3) |
| 16.0 (10) | 80 (50) | 1.37 (4.5) |
| 16.0 (10) | 120 (75) | 1.65 (5.4) |

***Source:*** Seville et. Al. (May 1962)

## b) Wave breaking on slope

When a wave strikes on the front slope of embankment, it breaks completely or is partially reflected creating standing waves which are sometimes twice the height of incident waves. To avoid these standing waves the front slope has to completely break the wave train. The front slope competency can be checked by the following equation:

$$\tan\alpha = \frac{8}{T_s}\left(\frac{H_s}{2g}\right)^{0.5} \qquad \textbf{[12]}$$

where:

α= bank slope angle to the horizontal (degree)

$H_s$ = significant wave height (ft)

$T_s$ = significant wave period (seconds)

g= acceleration due to gravity (ft/$s^2$)

As on rivers we have to deal with mostly the shallow water conditions, there is no change in the height of waves i.e.

$$\boldsymbol{H_1 = H_o \; and \; L_o = \frac{gT^2}{2}} \qquad \textbf{[13]}$$

$$\therefore \frac{H_o}{L_o} \simeq \frac{\tan^2\alpha}{5.1} \text{ ---------- (After Iribarren and Nogales)} \qquad [14]$$

As on rivers we have to deal with mostly the shallow water conditions, there is no change in the height of waves i.e.

$$H_1 = H_o \; and \; L_o = \frac{gT^2}{2} \qquad \textbf{[15]}$$

$$\therefore \frac{H_o}{H_o} \simeq \frac{\tan^2\alpha}{5.1} \ldots\ldots (After\ iribarren\ \&\ Nogales) \qquad \textbf{[16]}$$

## c) Wave Run-up or Ride

The run-up "R" of a breaking wave, measured vertically above the mean water surface level, is given by "HUNT" as

$$\frac{R}{H} = \frac{K \tan_{\alpha}}{\left(\frac{B}{T}\right)\left(\frac{H}{gg}\right)^{0.5}} \quad \textbf{[17]}$$

where

R = Wave run up (ride)

H = Wave height

K = Surface roughness coefficient for embankment slopes usually assumed as 2.3'

$\gamma$ = density of water – cbs/ft

g = gravity acceleration – ft/sec$^2$

d = Bank slope angle to the horizontal (degrees)

## d) Protection Stone Size

The semi-empirical Hudson and Jackson formula for determining the required weight of the stone is:

$$W_{50} = \frac{W_r H^3}{K_{RR}\left(S_r - 1\right)^3 Cot\theta} \quad \textbf{[18]}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi W_r}\right)^{1/3} \quad \textbf{[19]}$$

where:

$W_{50}$ =median stone weight in gradation by weight (lb)

$W_r$=unit weight of the stone= 165 (lbs/ft$^3$)

H=design wave height (ft)

$S_r$=specific gravity of stone = $W_r$ / $W_w$

$W_w$=unit weight of water = 62.5 (lbs/ft$^3$)

θ=slope measured to horizontal (degree)

$K_{RR}$ =stability coefficient for graded angular stone = 2.0 for double stone units armoured layer

$D_{50}$ =equivalent spherical diameter of stone having the same weight as $W_{50}$ (ft)

The value of "H" may be determined using the following equations,

**$H = 1.25\ H_s$** **[20]**

where:

$H_s$ = significant wave height (ft)

## 6.4 Stone pitching protection

The slope protection will consist of hammer dressed stone pitching underlain with transition layer of well graded rock spall and filter layer of well graded gravel or crushed stone from toe to the crest level.

### 6.4.1 Stone Pitching Layer

All stones should be contained within the thickness of stone pitching layer. The larger stones should not protrude above the general surface of the stone pitching. The minimum thickness of stone pitching layer is $=1.5D_{50} \geq$ 1.5 ft(0.45 m). [21]

### 6.4.2 Rock Spall Transition Layer

150 mm (6 inches) thick rock spall transition layer is provided between the stone pitching layer and filter layer. The spall layer should be well graded from 50 mm to 100 mm (2 to 4 inches). Not more than fifteen (15) percent should be larger than 100 mm (4 inches) and not more than five (5) percent should be smaller than 50 mm (2 inches).

### 6.4.3 Filter Layer

To prevent infiltration of the material susceptible to draining into the filter, the following requirement are to be met:

$$\frac{D_{15}\text{ percent size of filter material}}{D_{85}\text{ percent size of material being drained}} \leq 5 \text{ (stability)} \quad [22]$$

~~$D_{85}$ percent size of material being drained~~

$$\frac{D_{50}\text{ percent size of filter material}}{D_{50}\text{ percent size of material being drained}} \leq 25 \text{ (segregation)} \quad [23]$$

~~$D_{50}$ percent size of material being drained~~

where:

$D_{15}$, $D_{85}$, $D_{50}$ = material size for which 15%, 85%, 50% respectively are finer than

To assure that the filter material is much more permeable than the material being drained the following requirement is met:

**Permeability:**

$$\frac{D_{15}\text{ percent size of filter material}}{D_{15}\text{ percent size of material being drained}} \geq 5 \text{ (permeability)} \quad [24]$$

~~$D_{15}$ percent size of material being drained~~

The permeability of a soil is approximately proportional to the square of its $D_{15}$ particle size. Therefore, the criterion given by above equation assures that the filter material is at least twenty five (25) times more permeable than the material being drained.

The filter layer of gravel or crushed stone should be well graded from 50 mm (2 inches) down. Not more than fifteen (15) percent should be larger than 50 mm (2 inches) and not more than ten (10) percent should be smaller than 1.5 mm (1/16 of an inch).

### 6.4.4 Geotextile Filter

Geotextile filter may be used on bank slope protection where gravel or crushed rock filter materials are not readily available or are not cost effective. However, the overland flow beneath

the geotextile can cause the development of gullies in bank material and bulges in the geotextile.

## 6.5 Stone Apron

The function of stone apron is to launch and cover the scoured subaqueous slope below pitching level and prevent the possibility of damaging effect of scour reaching close to the structure, with a possibility of undermining.

### 6.5.1 Stone Apron Design

To design a stone apron, it is necessary that the volume of stone should be sufficient to cover the local scour slope length in its fully developed state and also fill partially the scour hole at toe of the subaqueous slope.

### i. Launching Slope

The launching slope of apron maybe assumed as the angle of internal friction ($\phi$) of the river bed subsurface saturated soil located near side slope of the Bund. The representative values of "$\phi$" for various soil types in saturated condition are listed in the following table:

Table 5:Values of internal friction angle "$\phi$" for different saturated soils

| **Soil type** | **"$\phi$" (Saturated conditions)** |
|---|---|
| Gravel | 35º |
| Coarse sand | 28º |
| Silty sand | 25º |
| Fine sand | 28º |
| Clayey sand | 20º |
| Silt | 20º |

*Source:* Bolhales – Foundation analysis and design (1980)

### ii. Volume of Stone Apron

The volume of stone to be laid in apron should be enough to cover (or protect) the side slope of the scour hole with a reasonable thickness, sufficient to ensure safety of stone pitching. Some of the empirical/research methods for determining the value of stone apron presently used in Pakistan are presented in the following:

- Spring's (1903) formula for the side slope 2H:1V of the scour hole:

  Cover thickness = 1.25 T

  Length of slope = 2.24 D

  $Q_s$ = 1.25 T x 2.24 D = 2.8 DT(per ft) **[25]**

  where:

  D = scour depth below the normal bed level (ft)

T = thickness of stone pitching necessary to protect sand surfaces of various grades of sands and water surface slopes of rivers, refer Table 3.

- Pakistan Engineering Congress Golden Jubilee Publication (1963)

  **$Q_s$ =7 D(per ft)** **[**26**]**

  where:

  D = scour depth below the normal bed level (ft)

  Kalabagh Barrage Criteria

  **$Q_s$ = 10 D(per ft) (To cover a slope of I in 3 with a thickness of 3.25 ft)** **[**27**]**

  where:

  D = scour depth below the normal bed level (ft)

The material commonly found in the bed of alluvial rivers in plains of Pakistan is silty sand, with an average angle of internal friction ($\phi$) of twenty three (23) degrees in saturated conditions, which conforms to a slope of 2.36H:1V. Adopting a 3.25 ft thickness, recommended by IRI, the quantity of stone required works out to $3.25x(5.57)^{1/2}x$ D = 8.33 D or say 8.5 D.

## 6.6 Width of Stone Apron

Spring (1903) recommended normal width of stone apron=1.5 D.

Where:

D = scour depth below the normal bed level (ft)

### 6.6.1 Design of Slope Protection

Till recently the structural measures for protection of slope of flood works has been confined to hammer dressed stone pitching designed on the basis of the following empirical relationships:

- Old Manual of Irrigation Practices, Punjab (1943)

  Stone thickness = 1.3(ft)

  Spall/Filter layer = 0.7(ft)

  Total thickness = 2.0(ft)

- IRI - A General Recommendation

  Stone thickness = 2.5(ft)

  Spall/Filter layer = 0.75(ft)

  Total thickness = 3.25(ft)

- Spring (1903)

  Spring prepared a table 5.15 based on channel slope and grain size of the alluvium through which the river flows.

- Gales

  Stone thickness = 3.25(ft)

Spall/Filter layer = 0.75(ft)

Total thickness = 4.0(ft)(1.2 m)

- C.C. Inglis Formula

  $T = 0.06(Q)^{1/3}$ **[**28**]**

  where:

  T = thickness of pitching (ft)

  Q = discharge (cusecs)

  This equation yields normal results for discharges up to 300,000 cusecs (8,500 cumecs) but the thickness increases abnormally for discharges above 400,000 cusecs (11,300 cumecs).

- Sethi Formula

  $T=K(Q/f)^{1/3}$ **[**29**]**

  where:

  T = thickness of pitching (ft)

  Q = discharge (cusecs)

  K = constant

  f= Lacey's silt factor

  This equation yields slightly high but consistent results subject to correct selection of the constant K.

Considering the merits and short- comings of the above methods and general river alluvium the recommendation of IRI (total thickness = 3.25 ft (1.0 m)) appears to be most rational. This thickness has been provided at some spurs and has held very well.

Spring's table yields results similar to those of the IRI for rivers in plains with fine sediments and a ruling slope of 1 in 5000.

However, the design of slope protection is carried out for both velocity and wave wash effects utilizing the latest state-of-the-art and scientific knowledge available internationally and discussed in the subsequent sub-sections.

Sample calculations for a typical case are included at the end of this section as **Annexure A**.

### 6.6.2 Protection against flow currents

Two empirical methods given below should be used to determine the slope protection stone size against current attack. The larger stone size ($D_{50}$) resulting from either method may l be used.

a) Isbash Method

The Isbash equation given in the expanded form for determining the stone size on a slope for low turbulence is:

$$V = 1.20\left[2g\left(\frac{\gamma_s - \gamma_w}{\gamma_w}\right)\right]^{1/2}\left(1 - \frac{Sin^2\theta}{Sin^2\phi}\right)^{1/4}(D_{50})^{1/2} \quad \textbf{[30]}$$

where:

V= average velocity (ft/sec)

$\gamma_s$ = specific weight of stone= 165 (lbs/ft³)

$\gamma_w$= specific weight of water= 62.5 (1bs/ ft³)

θ =bank slope angle to the horizontal (degree)

ϕ = angle of repose for the stone (degree) (normally 40º)

$D_{50}$ = equivalent spherical diameter of stone having the same weight as $W_{50}$ (ft)

$W_{50}$ = weight of stone 50% weight of total material containing stone of less weight (lb)

g = acceleration of gravity (ft/s²)

The Isbash equation can be further expanded as:

$$D_{50} = \left(\frac{6W_{50}}{\pi\gamma_s}\right)^{1/3} \quad \textbf{[31]}$$

$$W_{50} = 0.0219\frac{V^6.\gamma_s}{g^3}\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^3\left(1 - \frac{Sin^2\theta}{Sin^2\phi}\right)^{-3/2} \quad \textbf{[32]}$$

b) Maynord Method

The basic Maynord equation for determining the stone size on a slope for low turbulence is:

$$D_{30} = S_f.C_S.C_v.C_t.d\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5}\left(\frac{V}{(K_1.g.d)^{0.5}}\right)\right]^{2.5} \quad \textbf{[33]}$$

where:

$D_{30}$ = stone size for which 30% is finer than (mm)

$S_f$= safety factor= 1.1

Cs= stability coefficient for incipient failure = 0.30 for angular stone

$C_v$= velocity distribution coefficient= 1.0 for straight channels and inside of bends

= 1.283 – 0.2 log (R/W) for outside of bends (1.0 forR/W $\geq$ 26)

R= center-line radius of bend(ft)

W= water-surface width at upstream end of bend( ft)

$C_t$= blanket thickness coefficient = 1.0

d=discharge depth at 20% upslope from toe of bank slope.

$K_1$ = side slope correction factor

$=-0.672 + 1.492 \cot \theta - 0.449 \cot^2 \theta + 0.045 \cot^3 \theta$

$\theta$= bank slope angle to the horizontal (degree)

$\gamma_s$= specific weight of stone= 165 ( 1bs/ft$^3$)

$\gamma_w$ = specific weight of water = 62.5 ( 1bs/ft$^3$)

g= acceleration of gravity (ft/s$^2$ )

V= average velocity (ft/s)

The expanded Maynord equation is:

$$D_{30} = 0.41d\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{0.5}\left(\frac{V}{(K_1.g.d)^{0.5}}\right)\right]^{2.5} \quad \textbf{[34]}$$

$$D_{50} = D_{30}(D_{85}/D_{15})^{0.32} \quad \textbf{[35]}$$

where:

$D_{50}$ = average stone size

$D_{30}$, $D_{85}$, $D_{15}$ = stone sizes for which 30%, 85%, 15% respectively are finer than

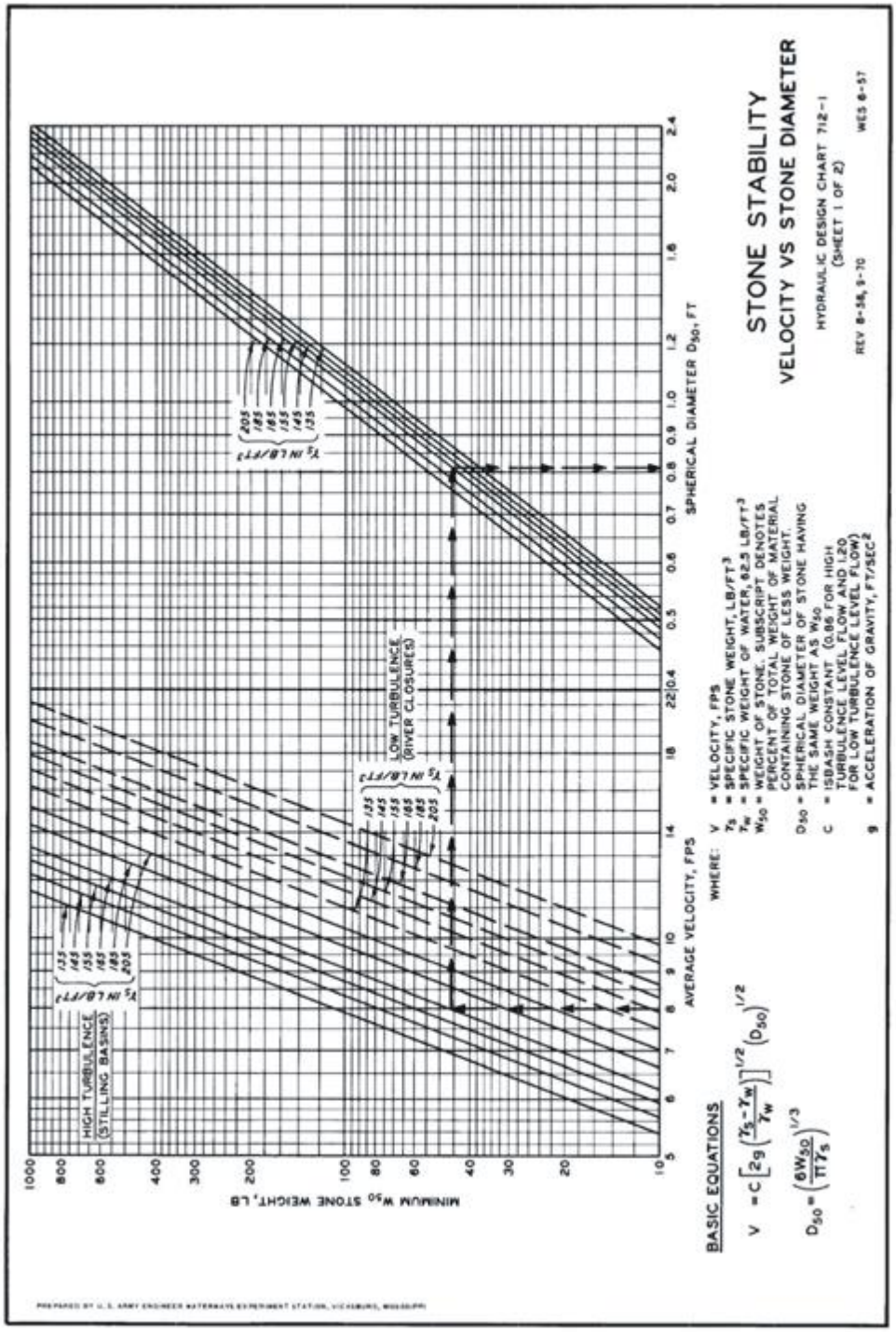



***Figure 13:*** Typical details of stone pitching protection and stone apron

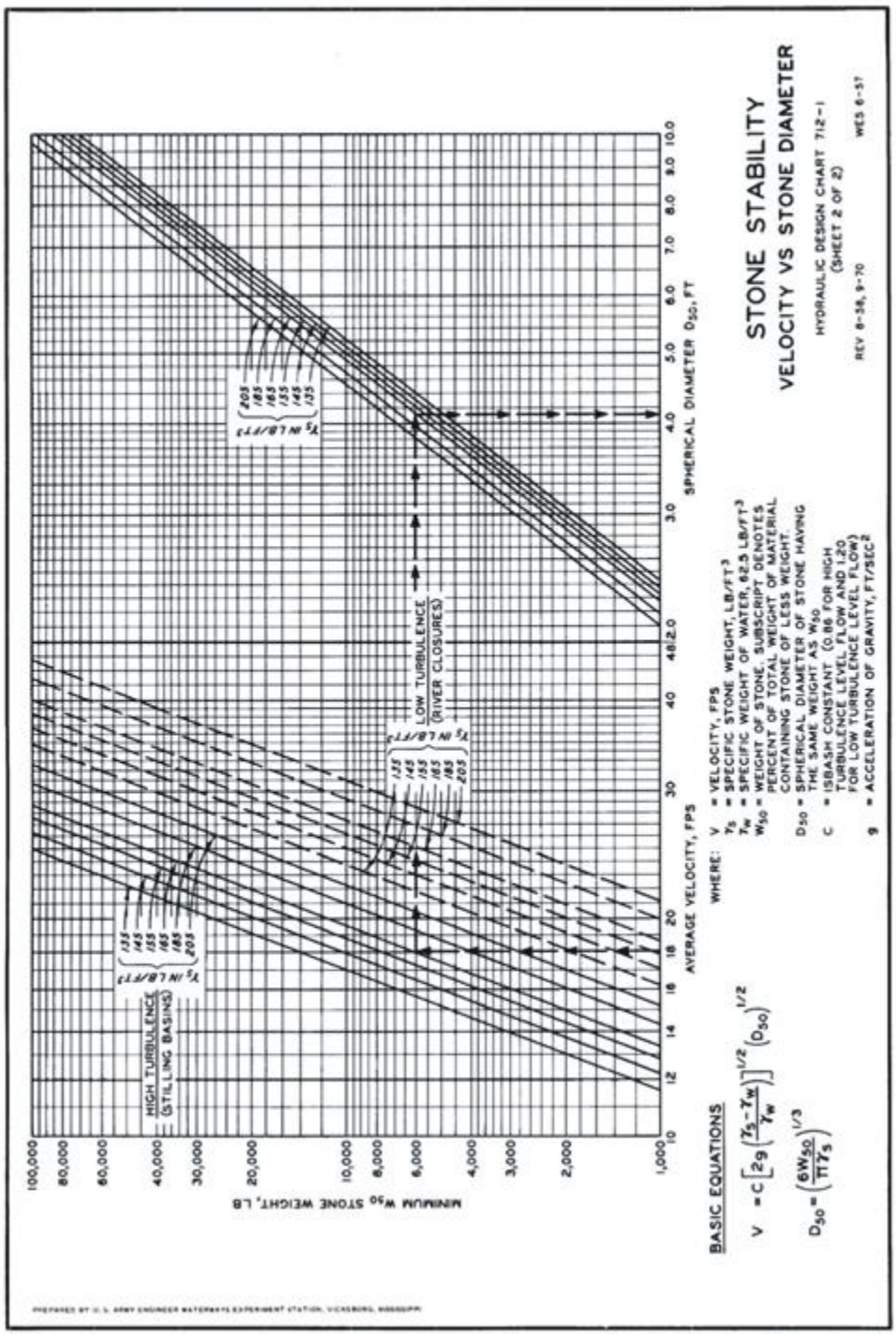



***Figure 14:*** Typical details of stone pitching protection and stone apron

## 6.7 Minimum Stone Size

The function of stone apron is to launch and protect the scoured subaqueous slope below pitching level. The stone size ($D_{50}/W_{50}$) to be laid in apron should be at least of same size or larger than that determined for the slope pitching.

$$D_{50} = D_{30}\left(D_{85} / D_{15}\right)^{0.32} \qquad [36]$$

where:

$D_{50}$ = average stone size

$D_{30}$, $D_{85}$, $D_{15}$ =stone sizes for which 30%, 85%, 15% respectively are finer than

The size gradation of apron stone shall be established from an envelope as shown in Figure-15.

| | | |
|---|---|---|
| $W_{100}$ (max | = | $4.0W_{50}$(min |
| $W_{100}$ (min | = | $2.0W_{50}$(min |
| $W_{50}$(max | = | $1.5W_{50}$(min |
| $W_{15}$(max | = | $W_{50}$(min |
| $W_{15}$(min | = | $0.4W_{50}$(min |

Typically, a stone size of 18 kg to 113 kg (40 lbs to 250 lbs) is used.

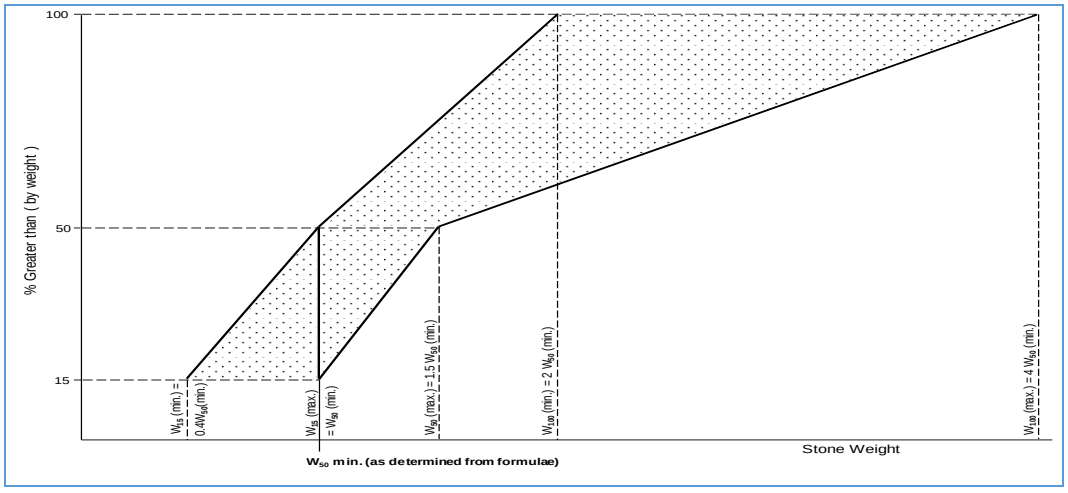



***Figure 15:*** Gradation envelope for apron stone

Sample calculation for design of slope protection appear as **Annexure-A.**

### 6.7.1 Recommendations

The following are recommended:-

1) The existing criteria for river face slopes of earthen embankments need revision. We may either construct the Water face slopes in conformity with stable beach slope or the existing slopes may be provided with groper protection.

The slope protection measures may be executed in two stages:

i) Trees with good foliage and medium height, or other plants like Gulabasi may be grown in a thick belt in front of the embankments at a distance of 10 ft from the toe, but Gulabasi if grown or other bushes may not be allowed to spread on to the slopes.

ii) Permanent protection on Up Stream slope in the form of stone pitching with proper filter or a layer of cement concrete (1 : 3 : 6) 6" thick may be provided but in case of concrete the free board will have to be increased by 0.5 ft specially on

account of reduction in friction co-efficient of slope protection and consequent increase in wave run up.

The existing criterion for free board may also have to be revised. The following standards are recommended:-

a. On River Indus = 7.5 feet

b. Oil River Chenab and Ravi = 7.0 feet

c. On River Jhelum & Sutlej = 6.0 feet

It is hoped that the above recommendations will go a long way in ensuring the safety of earthen embankments against wave action.

## 7. CONSTRUCTION OF BUNDS

The construction of Bunds bears a high importance towards effectiveness of Bunds in flood protection. The Bunds are designed with the assumption that the Bunds will be constructed with homogeneous isotropic soil which in general is not the case. Very few Bunds and those too in developed countries are constructed with a homogeneous soil body because this requires selection of soil which is in some cases exhorbitantly expensive.

The specifications for earthwork in Bund construction are generally written with the assumption that the soil to be used will be homogenous and isotropic. Since this is not normally achievable, special construction methodologies like blending, reworking and special drainage support, some of which have already been explained in some earlier paragraphs, are required.

In general, the soil in the bank body should be isotropic or proper measures included in design. Stratification in the Bund section of highly impermeable and highly permeable soil should be avoided through blending, reworking or selection of soil to be used in the embankment body. Heterogeneous masses will require special measures to be built in the Bund body.

However, the Bunds must be constructed in full conformity with the prescribed specifications to be capable of providing desired protections against flood waters.

## 8. STRENGTHENING / REMODELING OF BUNDS

Reinforcement / strengthening / remodeling of Bunds is required if,

a) The initial Bund design or construction were substandard.

b) There is a marked change in flood causative factors which increase the flood magnitude, flood water levels requiring change in design and additional care in placing the reinforcing earth and other interventions.

Actually, the Bunds constructed in developing countries in the past were constructed with the locally available earth and with inadequate compaction. Such Bunds require extra care in strengthening effort. Some explanation of such efforts appears in the design write-up.

## 9. INSTRUMENTATION IN BUNDS

Generally, Bunds lower than 8 meters do not require any instrumentation network except in cases where the Bund section in composed of heterogeneous / Anisotropic soils, lines of standpipe piezometers / water level recorders are provided to ascertain the line of saturation / hydraulic gradient line and the point of exit on the downstream slope of the Bund. In consideration of the observation of piezometric water levels, decision for remedial measure is made and implemented.

# 10. MONITORING, SAFETY OF FLOOD BUNDS / BUNDS

## 10.1 Need for Effective Surveillance and Good Governance

The current situation is that the inadequately designed Bunds remain dry for a major part of the year and develop all sorts of weaknesses. These Bunds therefore need enhanced and effective surveillance through detailed in depth inspection and analysis for their safety. The procedure currently in vogue leaves much to be desired.

It is high time now that the existing design methods are upgraded to state of the art design methodology and the existing inspection procedures amended.

The general governance of flood Bunds is quite weak specially in developing countries. Although all countries have some sort of organization for flood control but performance is below the required level. Creation of properly authorized / empowered organizations like FEMA in USA with added responsibility of strengthening and maintenance is call of the day.

## 10.2 The Inspection Team

It is generally considered that flood bunds do not to require maintenance in dry periods. This is incorrect because continuous repairs, rehabilitation and surveillance are the key to safety of these structures. Based on the above, a casual inspection by a randomly picked team is inadequate. The flood Bunds being important need a competent inspection.

A three person team comprising a competent leader, a hydraulic engineer and a geotech engineer will hopefully be a good evaluation outfit, although some site specific scenarios may require a change in the constitution of the inspection team.

### 10.2.1 Scope of Works for the Inspection Team

The team is assigned a detailed inspection and analysis for the flood Bunds and is expected to submit a comprehensive report containing observations, findings, conclusions and recommendations to the owner within 15 days of the inspection date.

## 10.3 Collection of Data

The inspection team should collect the data required for assessment of safety and may be but not limited to the (i) Current approved L-section and (ii) Designed cross sections

### 10.3.1 Existing Data

Top levels, Top widths, Side slopes, Observations of mile gauges along the Bunds, Water levels for various discharges, and NSL on both sides. The data which is not available will be observed by the staff incharge under the directions of the inspecting / evaluating team.

## 10.4 Safety Inspection and Evaluation

The safety inspection and evaluation team inspects the flood Bund on a pre-set date. The maintenance staff incharge of the flood Bund also remains available during the visit / inspection. The safety evaluation team is expected to carry out the tasks listed in the following. The team may however amend the task program if dictated by site specific conditions:

a) Review the design according to the current state of the art design methods/procedures, point out any deficiencies to be made up and suggest suitable action to withstand the accepted PMF (Probable Maximum Flood).

b) Carryout physical inspection of the Bund and look for:

   i) Top Levels

      If lower than designed, may result in overtopping.

*Solution: Redesign and strengthen the Bund raising the top to safe elevation.*

ii) Unwanted Vegetation and Debris

Vegetation roots can trigger and allow undue and damaging seepage. Vegetation and debris also retard the process of identifying problems like leakages, covered damages and implementation of their remedial measures.

*Solution: The unwanted vegetation and debris should be removed and not allowed to reappear.*

i) Encroachments

Unauthorized / unlawful structures on the slopes or top of a Bund and excavations at or close to the Bund toe weaken the Bund. Many a time Bunds have failed at such spots.

*Solution: All encroachments should be removed and the Bund checked for any defaults.*

ii) Slope/Section Stability

Slides, slumps, and cracks are serious problems resulting in failure.

*Solution: Redesign the Bund and implement the new design.*

iii) Settlement

Settlement of Bund top reduces the flood protection level and control.

*Solution: Investigate the causes and redesign accordingly. The design should be implemented expeditiously to ensure Bund safety.*

iv) Land Subsidence

General land subsidence, though rare is common to some typical soils and results in lowered flood protection levels.

*Solution: Consider re-alignment of the Bund or strengthening the section.*

v) Damaged Armour

Damaged stone or other protection renders the Bund vulnerable for failure during ensuing floods.

*Solution: Restore and strengthen the armour in accordance with the prevailing conditions.*

vi) Flood Wall Damage

In some unique locations, the Bund cannot be raised above a certain elevation and a flood wall is provided to raise the freeboard to adequacy level. In such cases cracks, tilting or arching in a flood wall indicate wall failure and lowered level of flood protection.

*Solution: Rehabilitate the flood wall.*

vii) Runnels Caused by:

a) Old tree roots

Old tree roots embedded in the body of the Bund get decayed with time and provide circular channels for rapid seepage which results in soil loss and breach.

Solution: Locate the tree root channels and block preferably by re-working the soil at the site.

b) Solutioning of Soluble Materials

Soluble materials if present in the body of the Bund are dissolved in water resulting in cavities/slits which combine to form a larger hole resulting in failure.

Solution: Rework or replace the soil. Provision of an impervious cut-off wall will also work.

c) Rodent burrows, holes and dens

The burrowing animals create holes and dens in the Bund as their living place which on reaching near the water level result in failure. Since these damages are localized and are difficult to clearly identify, they are sometimes termed as “Wild Cards”. The best way to explore and identifying these faults is the use of Ground Penetrating Radar (GPR).

*Solution: For better results, procurement of GPR and necessary staff training is required. Rodent caused cavities have to be refilled and in some cases, a solid cut-off may be needed.*

d) Piping action through foundation alluvial materials.

This phenomenon sometimes is exhibited through boils and springs on the country side of the Bund.

If not addressed properly the phenomenon can and mostly results in undermining of the Bund body and a breach.

*Solution: If only the body of the Bund is involved, an impervious cut-off like clay puddle core, masonry wall, a concrete wall, cellular or solid will be the answer.*

*In the case of foundation, if the defective soil depth is small, replacement will work otherwise jet grouting may be required.*

e) Erosion by river flow.

The river flow occurring alongside the Bund, depending on its velocity can cause erosion of riverside slopes and scour at the toe of the Bund and can damage the slope protections and launching aprons, thus endangering the safety of the Bund with ultimate failure.

*Solution: Protection has to be enhanced.*

f) Erosion by wave action.

Water waves generated by high winds or other elements like floating vessels, while breaking against the Bund body cause loss of soil from

body of the Bund through dynamic hammering and suction of the receding snout/lip of the wave. This action continues indefinitely and results in weakening of the damaged slopes and ultimately failure.

Wave action intensity is directly proportional to depth, fetch of water and wind velocity.

*Solution: Provide or strengthen the armour and freeboard.*

g) Ravines caused by heavy rains, if not repaired expeditiously can also result in breaches. Formation of ravines / rain cut-off is common in sandy soils.

Solution: Open and refill the rain cuts/ravines properly.

h) Excessive seepage exhibited by springs / boils spewing clear or muddy water can also end up in piping action and Bund failure.

Solution: If only the body of the Bund is involved, an impervious cut-off like clay puddle core, masonry wall, a concrete wall, will be the answer.

In the case of foundation, if the defective soil depth is small, replacement will work otherwise jet grouting may be required.

i) Accidents

Accidents like trees falling off, leaving a hole, heavy traffic accidents and accidental collision of heavy equipment causing damages to the Bund, which sometimes can be heavy, trigger Bund failure.

*Solution: Precautions should be taken to avoid the accidents and necessary arrangements should be kept available to repair the damages expeditiously as the occur.*

j) Vandalism

Vandalism is basically the act of damaging some others property wilfully and sometimes even with no serious motives.

*Solution: The security and watch has to be strengthened adequately.*

k) Sabotage.

Sabotage is a planned act with inimical motives towards causing damages to public properties to the extent of total destruction.

*Solution: The security and watch has to be strengthened adequately.*

The observations/findings of the evaluation team can be briefly shown in a checklist for quick overview. A sample generic format for such a checklist appears as **Annexure-B.** It may however be kept in mind that the sample is by no means the last word. It is basically a guide, which can be amended by the evaluation team to suit the site specific requirements.

## 10.5 Frequency of Safety Inspections and Monitoring

The safety inspection in a normal or wet year should be carried out twice in a year; A pre flood inspection in April / May and a post flood inspection in October/November. Special Inspections can also be instituted after unusual incidents. In flood less years, i.e. October/November, visit can be dropped/omitted.

## 10.6 Inspection, Monitoring and Evaluation Report

After inspection, analysis and view sharing the committee will formulate its inspection/evaluation report and submit it to the owner/requesting authority within 15 days of the Bund inspection. This report will normally contain An Introduction, Location of Bund, Analysis for Design Parameters, Physical Inspection Results i.e. observation and findings, Conclusions and Recommendations. The report will also be signed by all the team members and the incharge of the facility or authorized representative.

**Annexure-A**

# SAMPLE CALCULATIONS FOR FREEBOARD AND STONE PROTECTION ON EMBANKMENTS

## *1. Wave Height and Wave Run-Up*

Assume average depth (D) = 10.0 ft ( shallow )

Assume reach of fetch = 4.0 mile

= 4.0 ÷ 1.15 = 3.48 (nautical mile)

Wind ratio = $\frac{U_{water}}{U_{land}}$ for 4.0 mile = 1.28

Assume wind $U_{land}$ = 50 mph

Design wind $U_{water}$ = 50 x 1.28 = 64 mph

= 64 ÷ 1.15 = 55.65 knot

### *1.1 Significant Wave Height and Wave Period*

Use Bretschneider Method for shallow water:

$$H_s = 0.0555(U_w{}^2 F_e)^{0.5}$$

$$T_s = 0.50(U_w{}^2 F_e)^{0.25}$$

where:

$H_s$ = significant wave height (ft)

$T_s$ = significant wave period (s)

$U_w$= wind velocity (knot)

$F_e$ = effective fetch (nautical mile)

$H_s$ = 0.0555 (55.65 $^2$ x 3.48 )$^{0.5}$ = <u>5.76 ft</u>

$T_s$ = 0.5 (55.65 $^2$ x 3.48 )$^{0.25}$ = <u>5.09 second</u>

Shallow Water Wave Height = $H_s{}'$ = 0.65 $H_s$

= 0.65 x 5.76 = <u>3.75 ft</u>

### *1.2 Wave Length*

$L_o = 5.12\ T_s{}^2$

$L_o$ = 5.12 x 5.09 $^2$ = <u>132.65 ft</u>

*1.3 Wave Run-Up ( R )*

Wave steepness = $\frac{H_s'}{L_0}$ = $\frac{3.75}{132.65}$ = 0.03

From Figure 6.8, for $\frac{H_s}{L_0}$ = 0.03 and embankment slope = 2H:1V :

$\frac{R}{H_s}$ = 2.37

$R$ = 2.37 x $H_S$ = 2.37 x 3.75 = 8.9 ft (vertical)

***2. Check Wave Breaking on Slope***

Check the front slope competency by satisfying the following equation:

$$\tan\alpha = \frac{8}{T_s}\left(\frac{H_s}{2g}\right)^{0.5}$$

where:

ɵ = bank slope angle to the horizontal (degree)

$H_S$ = significant wave height (ft.)

$T_S$ = significant wave period (s)

g = acceleration of gravity (ft/s$^2$)

Assumed embankment slope = 2H : 1V

(with stone pitching protection)

$$\tan\alpha = \frac{8}{5.09}\left(\frac{3.75}{2\times 32.2}\right)^{0.5} = 0.38$$

Required embankment slope for wave breaking = 2.6 H : 1V < 2H : 1V (assumed)

The complete wave breaking will not happen with embankment slope of 2H : 1V. For economic purposes the partial deflection of waves will be acceptable.

***3. Wind Set-Up***

Use Zui -Der -Zee Formula (see Section 5 ) :

$$S = \frac{U^2 F}{1400D}Cos\theta$$

where:

*S* = wind set-up above still pond level (ft.)

U = wind velocity (mile/hour)

F = fetch (mile)

D = average water depth (ft.)

Θ = angle of fetch and wind (degree)

Assume wind $U_{land}$ = 50 mph

F = 4.0 mile

D = 10.0 ft

Θ = 80º

$$S = \frac{50^2 \times 4}{1400 \times 10} Cos\, 80^o = \underline{0.12 \text{ ft}}$$

***4. Required Freeboard***

Required freeboard = 2/3 wave run up + wind set-up + 1.0ft. (see Section 5)

= 2/3 x 8.9 + 0.12 + 1.0

= 7.0 ft

Adopted freeboard = 7.0 ft (2.0 m)

***5. Stone Pitching Size and Layer Thickness***

Use Hudson and Jackson Formulae:

$$W_{50} = \frac{W_r H^3}{K_{RR}\left(S_r - 1\right)^3 Cot\theta}$$

$$D_{50} = \left(\frac{6W_{50}}{\pi W_r}\right)^{1/3}$$

where:

$W_{50}$ = median stone weight in gradation by weight (lb)

$W_r$ = unit weight of the stone (lb/ft$^3$)

H = design wave height (ft)

$S_r$ = specific gravity of stone = $W_r$ / $W_w$

$W_w$ = unit weight of water (lb/ft$^3$)

Θ = slope measured to horizontal (degree)

$K_{RR}$ = stability coefficient for graded angular stone

$D_{50}$ = equivalent spherical diameter of stone having the same weight as $W_{50}$(ft)

$W_r$ = 165 (lb/ft$^3$)

H = 1.25$H_s$' = 1.25 x 3.75 = 4.69 ft.

$W_w$ = 62.5 (lb/ft$^3$)

$S_r$ = $W_r/W_w$ = 165 ÷ 62.5 = 2.64 ft.

Θ = 26.57$^o$ (bank slope = 2H:1V)

$K_{RR}$ = 2.0 ( for double stone layer )

$$W_{50} = \frac{165 \times 4.69^3}{2 \times (2.64-1)^3 Cot(26.57^o)}$$ = 965 lb ( 437 kg )

$$D_{50} = \left(\frac{6 \times 965}{\pi \times 165}\right)^{1/3}$$ = 2.24 ft. ( 0.68 m )

Thickness of stone pitching layer = 1.5 $D_{50}$

= 1.5 x 2.24

= 3.36 ft. (1.0 m )

**Annexure-B**

# SAMPLE CHECKLIST FOR SITE INSPECTION

Name of the facility – (say) XYZ Bund

In-charge Division, Circle, Zones: (say) X-Division, Y-Circle & Z-Zone

Date of Inspection ……

Design data (i) Top width; (ii) Top RL; (iii) Side slope; (iv) HFL; (v) Free board etc.

Analysis Carried out ……..

Comments on Analysis ……..

Observations & Findings

| | |
|---|---|
| a) Review on Design with respect to latest standards | |
| b) Phreatic line development | |
| c) Physical conditions (visual) | |
| d) Top width (compared with design) | |
| e) Top RL (compared with design) | |
| f) Side slopes (compared with design) | |
| g) HFL (compared with design) | |
| h) Free Board (compared with design) | |
| i) Patrol road and section maintenance | |
| j) Unauthorized traffic | |
| k) Slumping or settlement | |
| l) Body materials suitability determined through available data or investigations and grain size analysis at various depths and locations. | |
| m) Any visible/apparent runnels, caused by old tree roots | |
| n) Rodent, holes, burrows and dens | |
| o) Evidence of solutioning | |
| p) Evidence of piping | |
| q) Evidence of heavy seepage, springs and boils on the land side | |

| | |
|---|---|
| r) Erosion by flow | |
| s) Is the bund prone to wave action | |
| t) Erosion by wave action | |
| u) Evidence of greater permeability of foundation materials (say in creek crossings) | |
| v) Rain cuts and ravines | |
| w) General land subsidence | |
| x) Damages to stone or other protections | |
| y) Evidence of compressible or expansive (plastic) soils in foundation. | |
| z) Evidence of Misuse | |
| aa) Accidents | |
| bb) Vandalism | |
| cc) Sabotage | |
| dd) Flood watching arrangements | |
| ee) Gauges observation system | |
| ff) Flood warning and information system | |
| gg) Investigation carried out | |
| hh) Analysis of observations and investigations | |

Conclusions and Remarks ……

Recommendations ……

Signatures

| Name & Designation | Incharge of the Bund inspected | Member Hydraulics | Member Geo-tech | Team Leader |
|---|---|---|---|---|

Date:

**Paper No. 777**

# TARBELA 4TH EXTENSION HYDROPOWER PROJECT

**Dr. Izhar ul Haq & M. Mushtaq Ch.**

# TARBELA 4TH EXTENSION HYDROPOWER PROJECT

By:

**Dr. Izhar ul Haq[1], M. Mushtaq Ch.[2]**

## Abstract

Tarbela the largest earthfill dam 475 ft. high was constructed on river Indus under Indus Basin Replacement Works in 1974. Its gross storage was 11.6 MAF. Four units of 175 MW and six units of 175 MW each were installed on Tunnels $T_1$ and $T_2$ in seventies and early eighties. In early nineties four units of 432 MW each were installed on $T_3$ third tunnel. $T_4$ and $T_5$ were used for irrigation releases. In order to meet the power shortages, three units of 470 MW each are being installed on Tunnel 4. The design was prepared by JVC lead by Mott MacDonald and the construction is being carried out by Sino Hydro. $T_4$ extension power house is located at the back of the existing powerhouse near the western hill.

The intakes of both Tunnel 3 and Tunnel 4 are being constructed at higher level in the right abutment and joined to the existing tunnels by a vertical shaft. This would reduce the silt entry to the turbines and avoid the risk of the tunnels being clogged by onslaught of the sediment delta which is already higher than the intake invert level of $T_3$ & $T_4$ (1160 ft.) Right hand Low level outlet of $T_4$ would also be retained to flush out the sediments and meet the irrigation indent if it is more than the outflow capacity of the power house. This paper describes the salient features of the T4 extension project and the problems faced during its construction.

## Introduction

Tarbela 4th Extension Hydro Power Project ($T_4$) would add 1410 MW to the existing 3478 MW generation capacity of Tarbela Dam Project. Initially meant for irrigation discharges, Tunnel 4 release structure (flip bucket) has been demolished to allow the new power station to be located away from the water spray from the existing relief valves of Tunnel 3 power house and the westerly hill side. The new water release and sediment flushing structures have been incorporated to the right of the power house.

Due to the increasing sedimentation in reservoir, the intake structures of both the tunnels $T_3$ and $T_4$ are also being raised to RL415 m (1362 ft.).

An inception report was prepared in 1992 during execution of the Tarbela3rd Extension Project. In June-2010, Consultancy contract for detailed design and tender documents was awarded to a Joint Venture comprising of Mot Machonald of U.K; Coyne et Bellier of France with local firms MM Pakistan and ACE. The design & tender documents were essentially completed in 2012. The construction work was awarded on the basis of ICB to Sinohydro, now Power Construction Company of China Ltd.(PCCC), in September 2013 to be completed in 4 years.

The water release capabilities of the as built power generating facilities are much greater than originally anticipated; with 165% of the originally envisaged capacity currently installed.

Tarbela 4th Extension is a moderate extension scheme to an already substantial hydro scheme, rather than a green field with all its front end civil works and associated impacts. As such the

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overriding economic driver is to maximize the additional energy output from the extra units. The project is nearing completion and expected to be inaugurated in February 2018.

## Location of the Powerhouse

**Fig-1** shows the layout plan of Tarbela Dam existing Powerhouse. The Tarbela 4$^{th}$ Extension power house is located at the west of the existing powerhouse and u/s of the existing $T_4$ flip bucket outlet. It has been founded about 50m below the road level on graphitic to chloritic schist just below dolerite and quartzite, almost same types of foundation rocks as the existing powerhouse.

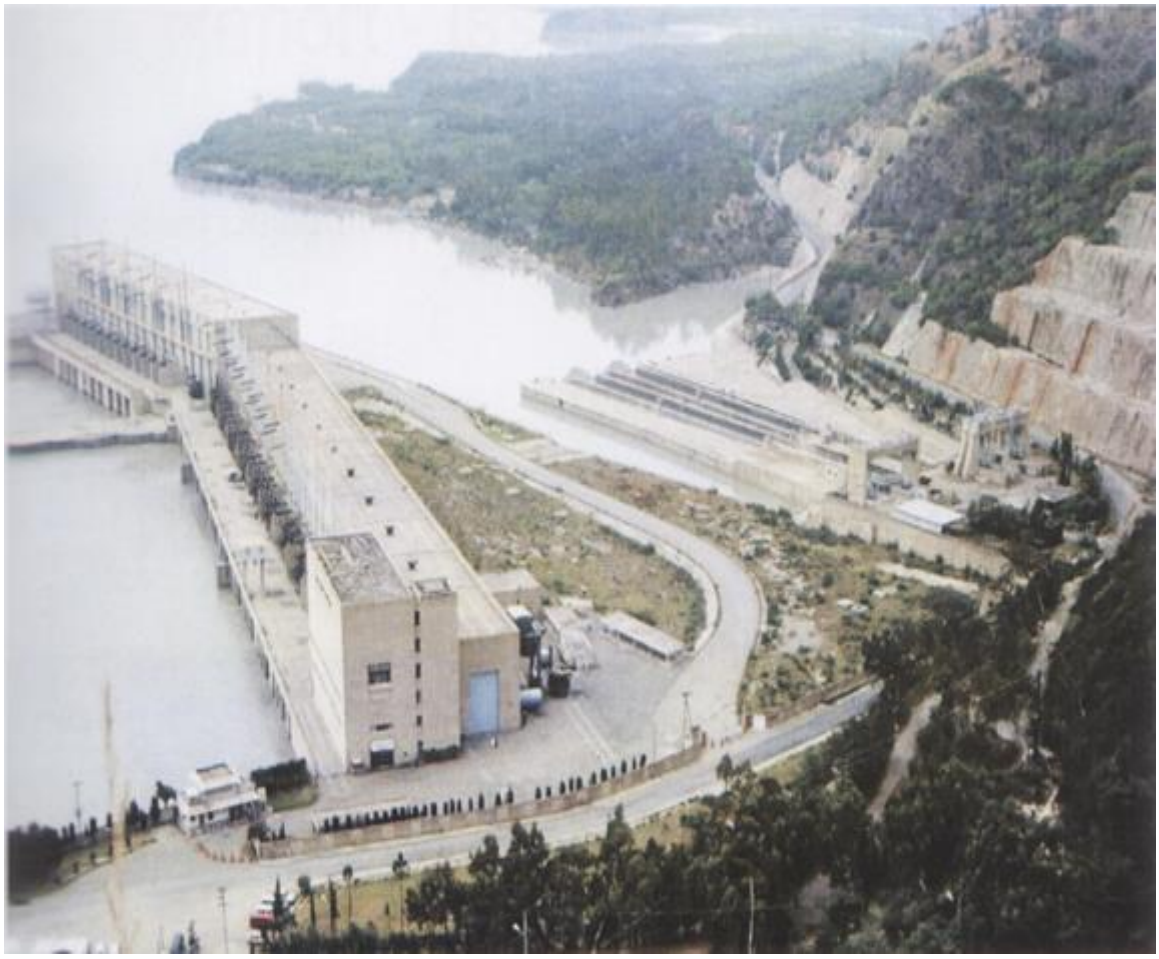


***Figure: 1:*** Layout Plan of Tarbela Dam Existing Powerhouse

The foundation is a wide spread RCC raft. Consolidation grouting was carried out underneath the foundation. A grout curtain at the periphery and drainage arrangement has also been installed under the foundation. After mathematical and physical modeling to study the hydraulic behavior of the flow during operation, Y branch connection to the existing Tunnel 4 has been made as shown in the **Fig-2.** It was found that the penstock design was feasible and the maximum stresses in the reinforcement plates were acceptable.

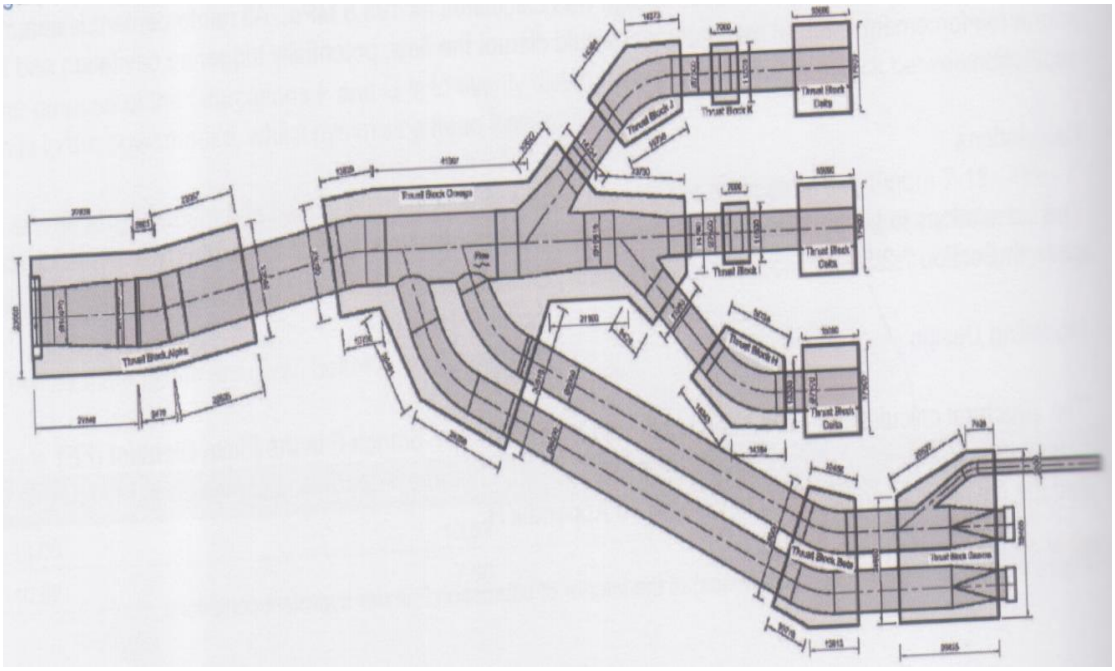


***Figure 2:*** Manifold Y Branch Connection to the Existing Tunnel 4 & Penstocks laid on thrust blocks

## Sedimentation

Average discharge of river Indus at Tarbela Dam is 60 MAF per annum. River Indus brings about 200 M.Tons of sedimentation at the Tarbela dam site. Its live storage when completed in 1975 was 9.679 MAF. Due to sedimentation, it is losing an average of 0.1 MAF of storage every year and presently its live storage is 6.174 MAF. The original river bed has gone up by about 200 ft. The pivot point of delta is at a distance of about 10km (5.45 miles) from the main embankment and its level is 1386 ft (2016 Hydrographic Survey). The movement of the delta towards the dam depends on the operation of the reservoir. If it is depleted and operated at level lower than the delta pivot point, then it gets eroded and moves towards the intakes. It is projected that the pivot point will reach the dam in about 10 to 12 years. Initially the minimum operating reservoir level was 1320 ft. It has been gradually raised and currently it is fixed at 1386 ft. Average %age values of sand, silt and clay from Besham Qila during months of June to September are sand 22%, Silt 58% and clay 20% respectively.

The sediment level at range line which is closer to dam is about 1244 ft (2006 survey). The intake level of $T_3$ and $T_4$ is 1160 ft. In order to protect the overwhelming of the intakes from the sediments and reduce the wear and tear of the turbines, it was decided to raise the intakes level of $T_3$ and $T_4$.

The data regarding the likely quantities of sediment to be released from the reservoir was studied which indicated the life of the turbine would be longer than was previously anticipated. It was concluded that turbine erosion would not be an issue. Therefore, turbine coating and provision for runner removal from bottom was not required.

The finalized design has relocated the power house which has reduced the costs and significantly increased the benefits. New flushing gates have been included within the power house complex that would allow the low level intakes to be flushed if they become blocked by sediments from the reservoir.

## Construction of Power House

Before starting the excavation for the power house, a cellular coffer dam 206.66m long and 32m high was constructed starting from the d/s end of the existing power house to the right bank **(Fig-3)**. It was filled with river run boulders and gravels for stability and grouted with the bed rock for water tightness. The cell in contact with the power house slipped in the first attempt but was steel anchored and got stabilized. The Coffer dam also served as a temporary diversion of road on the d/s toe of the dam leading to Topi, village.




***Figure 3:*** D/S Cellular Cofferdam

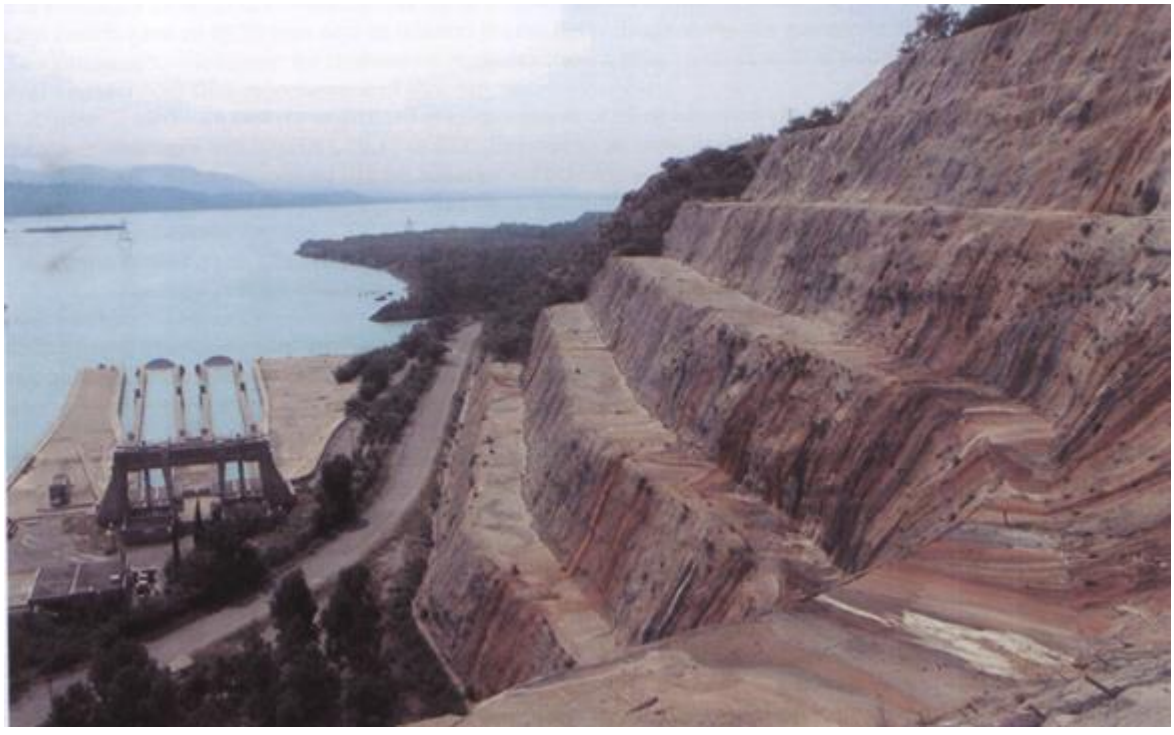


***Figure 4:*** View of the stabilized Outlet Slopes of Right Abutment

## Tunnel $T_4$

$T_4$ reinforced concrete lined portion is 1135m long and its 448.31m from d/s end is steel lined. The d/s control structure radial gates were dismantled. Its d/s most structure, flip bucket and the d/s wall of the $T_3$ old channel provided a safe enclosure from d/s water for excavation of the power house pit even before the completion of the cellular coffer dam. In 1982 a landslide had occurred on the d/s right abutment. The slope was cut back and stabilized by shotcrete lining, rock bolts/anchors and drainage holes (**Fig-4**). Excavation on the d/s and western hill was made carefully in benches stabilizing the slope. The new low level out let has been located on the right bank underneath the erection bay of the power house. There is bifurcation from the head race tunnel to the penstocks for the three units 15, 16, 17 and the low level outlets, as shown in **Fig-2**. **Fig-5** presents a picture of the T4 powerhouse.



***Figure 5:*** T4 Powerhouse

## Raised Intake of $T_4$

Due to the increasing sedimentation of the Tarbela reservoir, the Tarbela 4th Extension hydropower project included the raising of the Tunnel 4 intake and also Tunnel 3 intake to ensure their future operation. This involved huge excavation of the right abutment and the construction of vertical shaft to connect to the u/s end of the tunnel.

The raised intake arrangement consists of a large excavation, 'U' shaped in plan in the basic rock of the right abutment containing the two intake shafts. **Fig-6** shows plan of raised intake structures. Two large concrete intake structures, each consisting of 10 radial piers with roof and horizontal supports, sit atop the shafts to streamline the flow and prevent vortices. **Fig-7** shows physical model of the structures. The structures will also provide support for the intake bulkhead gates and the trashrack. The concrete lined shafts drop vertically into Tunnel 3 and 4. When the existing intakes are decommissioned, a concrete plug will be constructed and vertical bend formed within each tunnel. The internal diameter of intake shaft is 13.72 m (45 ft.).

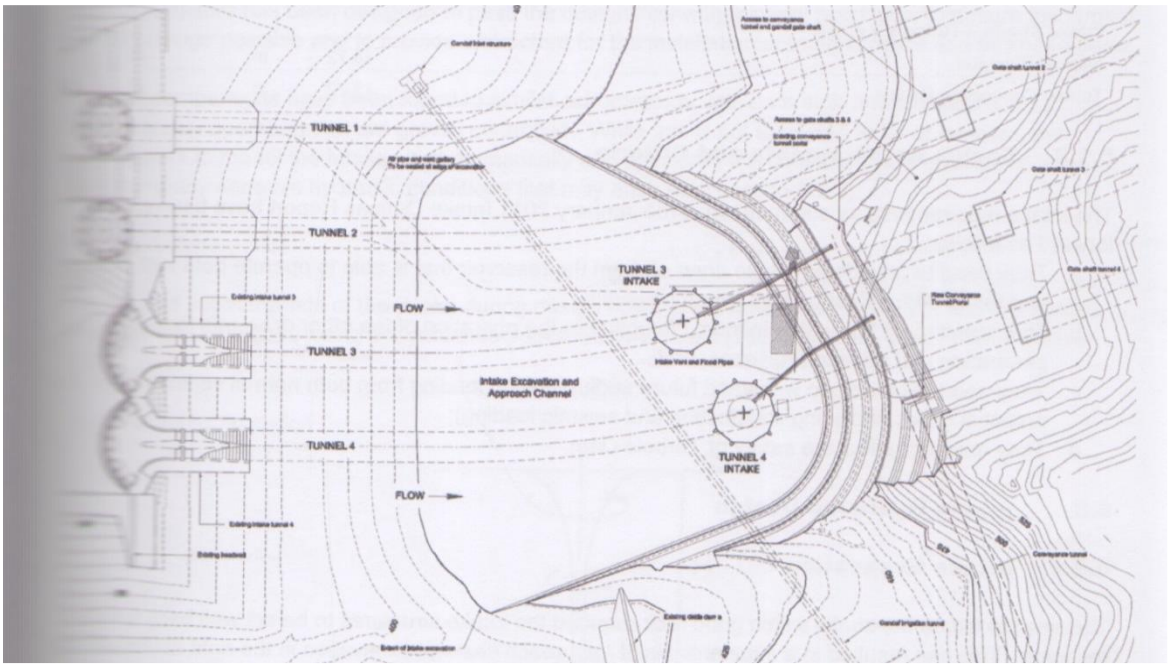



***Figure 6:*** General Arrangement of Raised Intake Structure of T3 and T4

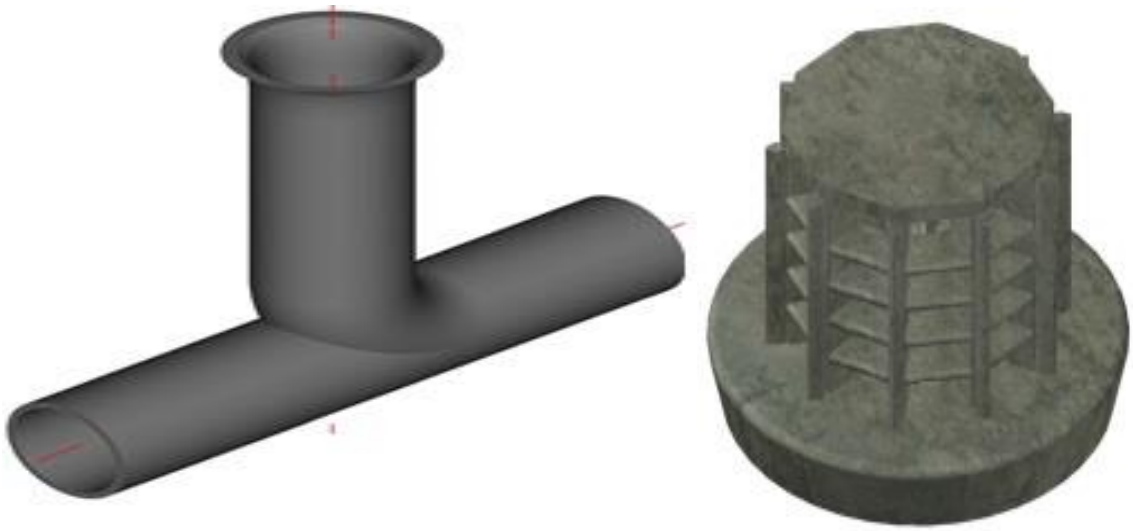


***Figure 7:*** Tunnel-4, Shaft and Raised Intake 4 Model

The intake has been located in good rock, set back from the reservoir necessitating channel excavation in the right abutment. The inlet excavation has been designed to provide approach velocity of less than 1 m/s. The optimum geometry, considering the new minimum operating level and the design velocity, consists of a channel width of 200 m at the reservoir and depth of 21.5 m to supply water in stream lines to the rear of the inlets; a width of 20 m from the rock slope has been kept.

The inlet rock slopes have been designed at a slope of 4v:1 H with 5 m wide benches at 15 m height intervals.

The required submergence of the intake during design flows to prevent vortex formation and associated water cover has been calculated as 20.5 m.

Trash racks are required to catch debris that would otherwise damage the plant and have been designed with a trashrack opening width of 0.4 m.

A 600 mm dia vent pipe has been provided to allow the structure to be dewatered and refilled during maintenance. The vent pipe extends from the inside face of the bell mouth intake, through the intake foundation to a point above the maximum reservoir level.

Both numerical and physical modeling of the reservoir was carried out. The construction of intakes is constrained by a number of factors including the reservoir water level, generation requirements and availability of special construction equipment.

In first year of $T_4$ hydropower construction period, when the reservoir draw down started, excavation of the u/s rock slope and d/s power house pit was initiated. Controlled blasting was rigorously monitored and allowed as these were close to the u/s grout curtain, structures of the tunnels and the existing power house. Dump trucks disposed of the muck via leading approach roads to the d/s of the main dam. Guniting, steel mesh and rocks bolts were installed where necessary.

A reinforced Coffer dam (12.5m high) starting from elevation near the u/s shore line was built to stop flooding of the pit upto reservoir elevation 461m **Fig-8**. This provided about 8 months working period each year during filling and depletion of the reservoir. Consolidation and curtain grouting was done under the Coffer dam to check the seepage of water into the pit. Leakage and seepage of water into the pit was less than anticipated and was pumped out. Each year Tarbela reservoir was filled to its full capacity El 1550 ft and thus the raised intake pit was filled with water. As the reservoir receded after the draw down to the coffer dam base level, valves installed in it were opened to empty the pit. The remaining water was pumped out and the excavation restarted. **Fig-9** shows the excavation of the pit and the U/S Coffer dam.




***Figure 8:*** U/S Coffer Dam



***Figure 9:*** Excavated Pit and U/S Cofferdam

The vertical shaft was excavated by raised boring method. From the center of the starter pit of the shaft, one meter dia bore hole was drilled through rock into the Tunnel 4. **Fig-10** presents raised boring scheme& shaft. Thick reinforcement and concrete lining of the $T_4$ was cut carefully for connection of the shaft with it (**Fig-11**). Area of the Tunnel 4 under the shaft was filled with earth to avoid any damage to it during the falling debris from the bore hole. The shaft was excavated by drill and blast method and the muck was dropped through the borehole into the tunnel from where it was disposed off by dumpers exiting through the d/s end of the tunnel. This expedited the excavation and mucking of the shaft instead from the top. Shot creting, rock bolting and anchoring where required was done side by side. The black rock being of good quality, there was no water and stability problem. After completion of the shaft excavation, it was concreted with reinforcement and connection to the existing tunnel was made.

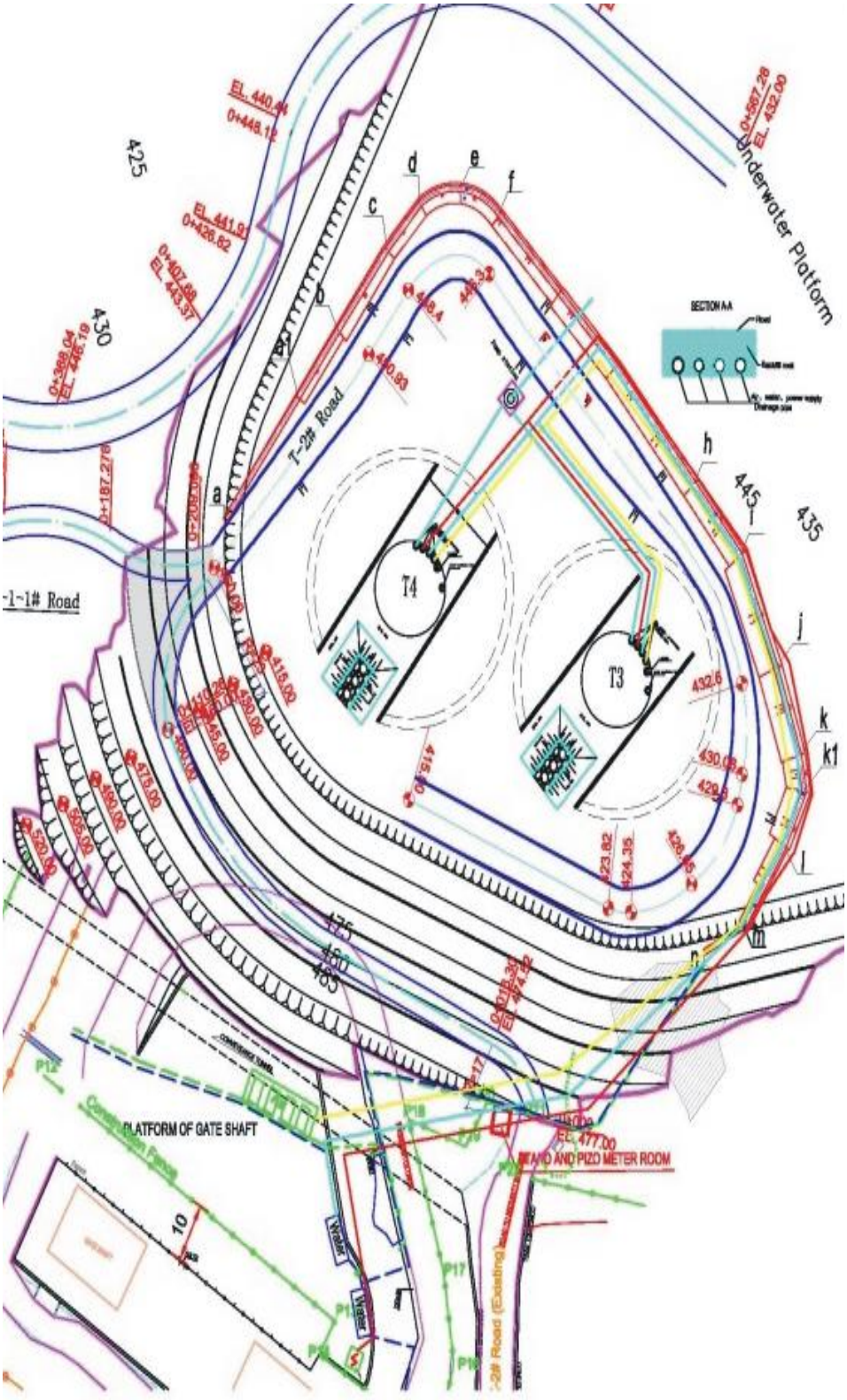



***Figure 10:*** Raised Boring Scheme for excavation of T4 vertical shaft



***Figure 11:*** Connection of vertical shaft with Tunnel-4

## Construction Problems and Solutions:

The construction of T4 extension project represents a unique set of activities that must take place in time to avoid delays. The success of the project was judged by meeting the criteria of cost, time, safety, resource allocation, and quality as determined by the Client.

The Consultants and the contractor of Tarbela 4$^{th}$ Hydropower Project faced a number of challenges which were solved professionally with team work. A few are listed below.

### 1. Replacing of micro piles with anchor bolts

Micro piles were designed to support the vertical excavation of the right side of penstock area at Powerhouse. However, due to short time, limited working space and difficulty in piling, the matter was revisited by the Consultant. It was suggested and decided to put Anchor bolts with shot crete for support of the vertical slope cutting. Anchor bolts with shot crete were used and the method was successful due to which valuable time was saved.

### 2. Cellular Concrete

Powerhouse area was excavated deep up to EL.297m due to which a lot of water seepage problem was faced especially under the LLO excavated area. The site team decided to go with cellular concrete before placing mass concrete in foundation. The method remained successful to control the continuous seepage during execution of concrete pouring at site.

### 3. Temporary Plug for Shaft

When it was known that the super structure of T4 shaft could not be completed in due time, then after quite a number of discussions and meetings, temporary plug for T4 shaft was designed

and installed. **Fig-12** shows temporary plug on T4 shaft. The plug worked successfully during the high reservoir period. With the temporary plug a shaft in place and the low level intake closed, the reservoir was filled and the work on T4 powerhouse construction continued.

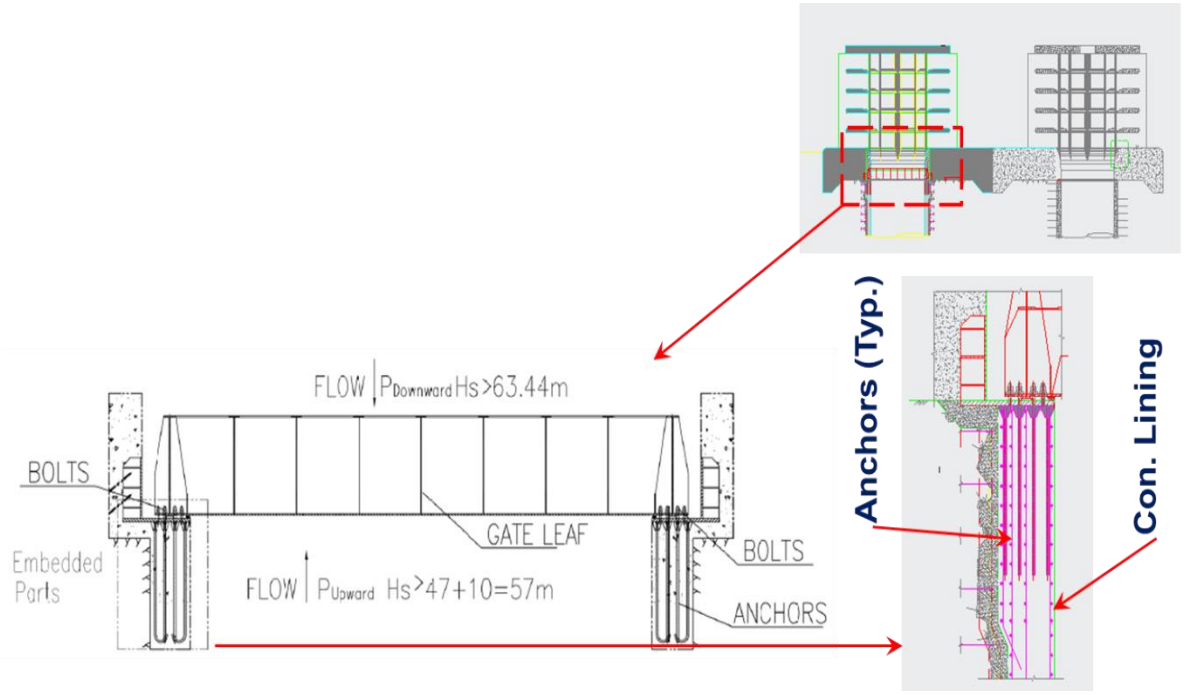



***Figure 12:*** Temporary plug on T4 Shaft.

## 4. Modification of Gantry Girder Structure at Switchyard

A problem was faced at switchyard due to misunderstanding between design and site survey team. Foundations of the extended switchyard were aligned 1m away (31m instead of 30m) from the existing switchyard. Time was short and the site management took decision to modify the steel gantry girder to save time and to avoid dismantling of the new foundations.

## 5. Use of existing foundations in T3 switchyard

The old foundations were to be dismantled and new ones to be constructed in T3 switchyard for installation of new structures and equipment. Some of the old foundations were tested and the results were found quite satisfactory especially in Bay 12 where construction was done. Those foundations were used after little modification at the top to suit the new equipment installation.

## 6. Vigilant Monitoring

Continuous monitoring and frequent contract management meetings by the Consultants and Client with the contractor led to resolution and completion of intricate problems in time.

## Conclusions

- The T4 extension to Tarbela would provide significant additional energy at a relatively low cost with high economic rate of return.
- The opportunity of raised intakes of T3 and T4 would prolong the life of the turbines and mitigate the risk of overwhelming of the tunnels by sediment delta.
- Construction of RCC Coffer dam instead of an earthfill dam every year during the Construction period helped save time and cost.

- Raised boring method helped cut down cost and time in construction of vertical shaft.
- Cellular concrete placement method controlled the excessive seepage problem in the foundation.
- Temporary plug of vertical shaft helped same time, fill the reservoir and continue the remaining works in T4 Powerhouse.
- Retention of low level outlet would help to meet irrigation demand, flushing of intakes area and generation of electricity at low reservoir level.
- Meticulous planning and continuous monitoring was necessary to achieve the milestones during the execution of the project.
- T4 units being more efficient would not only generate power during high flows but also replace other units during their repair.
- T4 construction experience would benefit the construction of T5.

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**Paper No. 778**

# CLIMATE CHANGE IMPACT ON WATER RESOURCES OF PAKISTAN

**Muhammad Munir Ch., Maqsood Shafique Qureshi & Dr. Allah Bakhsh Sufi**

# CLIMATE CHANGE IMPACT ON WATER RESOURCES OF PAKISTAN

By

**Muhammad Munir Ch[1]., Maqsood Shafique Qureshi[2], and Dr. Allah Bakhsh Sufi[3]**

## SUMMARY

The water scarcity has reached a crucial stage in Pakistan, and it is considered as one of the most water-stressed countries in the World. It is apprehended that country has become water scarce due to high population, estimated over 200 million in 2017. The water storage capacity has been reducing due to continuous sedimentation of its existing reservoirs. It is apprehended that by the end of year 2017, storage capacity of reservoirs would be reduced by **27%**. Under this scenario, construction of mega multi-purpose storage dams is assuming highest priority to sustain irrigated agriculture which is the backbone of Pakistan's economy and to meet the growing water needs of all water utilizing sectors. For meeting these requirements, a number of dams are planned on River Indus including under construction Dasu Hydropower Project. However, due to some controversial issues and non-availability of required funding, it has not been possible to undertake construction of any of storage dam, despite urgent national need for additional storage capacity, especially on river Indus. The construction of additional dams would not only augment irrigation supplies, but also mitigate floods, besides increase in power generation, employment opportunities and economic boost.

These dams would store billions $m^3$ of water that presently is going unutilized into the sea during floods in summer months. Confidence building measures among provinces, to address the related controversial issues and to develop consensus for building new dams, is a real national need. Additional water will be needed to meet requirements of irrigated agriculture and anticipated improvements in the standard of living through greater economic activity. Increase in industrial activity will also require additional water. Our underground water resources are already being over exploited. Increase in sustainable water resources in the country can be achieved through a combination of exploring valuable groundwater aquifers and constructing potential large, medium and small surface water storages and through water conservation strategies. This combination will be critical in meeting the water needs of the 21st century in the country.

The 4th Assessment Report (AR4) by the three working groups of the Intergovernmental Panel on Climate Change (IPCC) has projected that average global surface temperature will increase by approximately over 4°C during the 21st century. The recent annual average temperatures for Northern and Southern Pakistan are about 19°C and 24°C respectively. The glaciers constitute about 2,700 $km^3$ of stored volume of ice (Roohi, 2005), equivalent to about 14 years of average Indus River System inflows. The thermal regime of HinduKush Himalayan (HKH) glaciating region has in general warmed up and the frequency of occurrence of moderate as well as severe heat waves have also increased significantly.

The climatic change in Pakistan has slowed down melting of glaciers and snow from mountains due to low temperatures in recent years in northern area, thus reducing flow of rivers resulting in severe water shortages during winter. However, temperature in the northern glaciating area is again rising since 2014-2015 (Figure 5). Based on the studies available, it looks likely that the HKH glaciers are receding under the influence of global warming and their melting will increase

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with increase in summer temperatures. Recent simulation modeling conducted by Global Change Impact Studies Center (GCISC), Geneva, Switzerland, on Indus flows for a scenario where temperature will rise by **4°C** and glaciers would shrink to half of their present size, indicates that overall annual river flows would reduce by about 15% and the monthly flow pattern would also change considerably, with more water coming in spring and early summer and less water in the later part of summer.

Due to anticipated major climate change related concerns, it is projected that it will increase variability of monsoon rains to enhance the frequency and severity of high floods as during 2010 and recent past droughts. Incorporating realistic scenarios of water resources availability is a fundamental element of sustainable development. Water resources and climate change not only affects agriculture, but these also affect urban centers, industry and human health. Management of water resources requires a balanced and careful review of current knowledge under a comprehensive framework for the national development purposes of all sectors of life including environmental requirements.

## 1. BACKGROUND

The recent dry years, particularly in the southern part of the country and the resulting water-crisis has raised awareness about the changing climatic conditions in the country and in the region. It has also shown our weakness in understanding and anticipating climatic changes. Our behavior, for the most part has been reactive rather than anticipatory. The crisis cannot be addressed fully in the long term without some understanding of the dynamics and relationship between our water resources, the local weather, the regional weather and the global climate change. In this paper we have tried to anticipate a perceptible climatic change and how it will affect water resources of the country. Pakistan's water resources are greatly influenced by climatic changes in its region and also in the neighboring regions.

In global-scale assessments, basins are defined as severely being water stressed if they have per capita water availability below 1,000 $m^3$ per year. A water volume of 1,700 $m^3$ per capita/year is considered as threshold value required for domestic, industrial and agricultural water uses.

The water scarcity indicator of Falken Mark ~~are~~ is shown below:

| **WATER SCARCITY INDICATORS (Falken Mark Indicator, 1992)** | | |
|---|---|---|
| > | 1700 $M^3$ / CAPITA | Water Scarcity Rare |
| < | 1700 $M^3$ / CAPITA | Country Faces Seasonal Or Regular Water Stressed Conditions |
| < | 1000 $M^3$ / CAPITA | Water Shortages Hamper The Health and Well Being Of The Human Beings |
| > | 500 $M^3$ / CAPITA | Shortages Are Severe Constraints To Human Life |

Such water-stressed basins are located in northern Africa, the Mediterranean region, the Middle East, the Near East (including Pakistan), southern Asia, northern China, Australia, the USA, Mexico, north-eastern Brazil and the west coast of South America (Figure 1). Water use, in particular for irrigation, generally increases with temperature and decreases with precipitation.

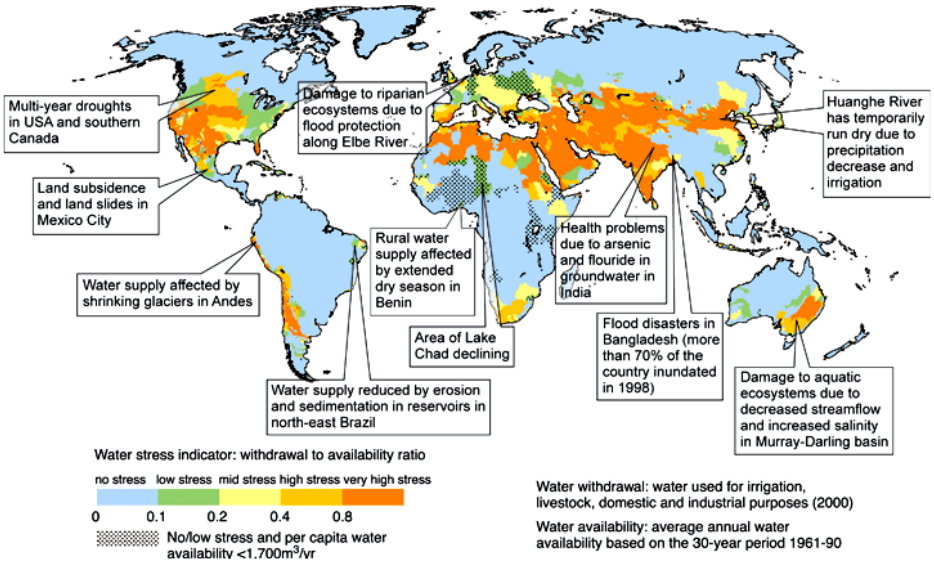



**Figure 1:** Examples of recent past vulnerabilities of freshwater resources, a water stress map relation to climate change (IPCC-WG/II).

The water resources of the region are also affected by climate changes in the world at large. There is a significant change in climate of Asia, in general, and Near East, in particular, to warrant global climate change concerns, such as follows (IPCC, WG-II):

**a)** **Lhasa, Tibet:** Warmest June1998 on record. Temperatures above77°F (25°C) for 23 days.

**b)** **Himalayas, India:** Glacial retreat at record pace. The Dokriani Barnak Glacier receded over 20 meter in 1998, despite a severe winter. The Gangorti Glacier is retreating about 30m per year. At this rate, scientists predict the loss of all central and eastern Himalayan glaciers by 2035.

**c)** **Tien Shan Mountains, China:** Glacial ice reduced by one quarter in the past 40 years.

**d)** **Middle East:** Severe drought has affected many parts, including Iran, Iraq and Jordan. Rainfall in parts of Iran decreased by 25% as compared to 1999. Eighteen out of the country's twenty-eight provinces are suffering in Iran.

**e)** **In Iraq the flow of Tigris and Euphrates** rivers have reduced by around 20% of their average flow, with serious consequences for irrigated farming. Jordan's agricultural production has been severely affected in recent past due to drought. Most countries in Middle East are suffering the consequences of prolonged water-shortages.

The evidences listed above show that in the recent past dry years in Pakistan were not isolate events; as the whole Near East region suffered as well.

The principal water resources of Pakistan are represented by surface flows of river Indus, its tributaries which are fed by snow and glacier melt from the Himalayan and Hindu Kush mountains and the rainfall in their catchments. The major surface water development in the country has been based on the water utilization of the rivers. About 15.4 Mha (38.1 Ma) land is irrigated by canals and tubewells. Water availability from canal withdrawals has progressively increased from 82.65 to 130.76 BCM (67 to 106

MAF) during the years 1947 and 1976 to meet water demand for irrigated agriculture and for human consumption. This increase was achieved with the construction of water storages at Chashma, Mangla and Tarbela.

The irrigation and river network of Pakistan is shown in Figure -2. It is one of the largest contiguous canal irrigation systems in the world. Apart from providing additional irrigation water, both Mangla and Tarbela Dams are also used for hydel power generation with installed capacity of 1,000 MW and 3,478 MW respectively. Ghazi Barotha Hydropower Project, commissioned in 2003-04, has a 1450 MW capacity, and 1320 MW is being added through Tarbela 4th Extension Project.

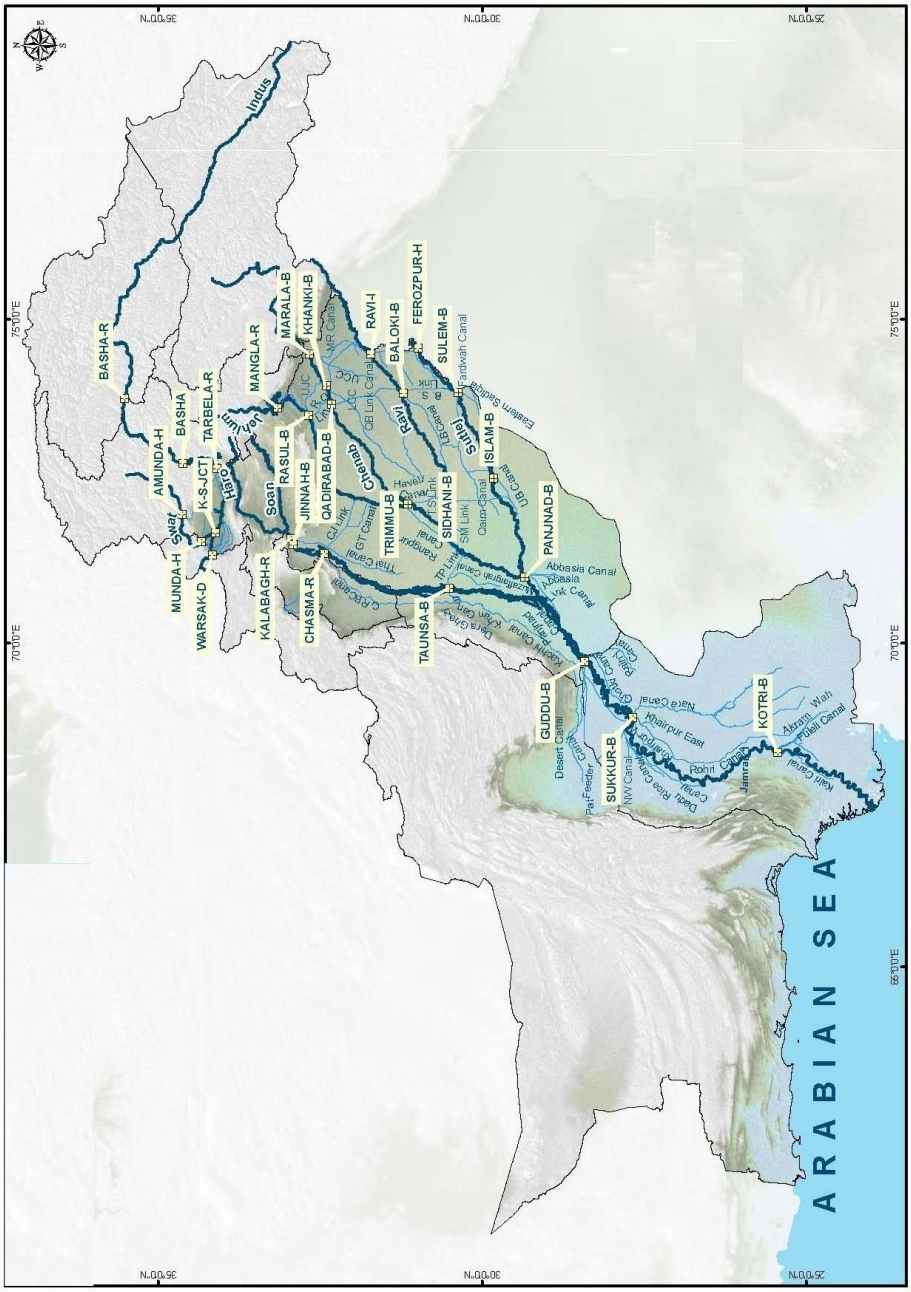



**Figure 2:** Irrigation and River Network of Pakistan

However, after completion of Tarbela Dam Project in 1976, there has not been any further increase in canal withdrawals although the population has continued to grow and have crossed 200 million in 2017. With increasing population and depleting water resources, Pakistan is heading towards a situation of sever water shortage and threat of famine (Figure 3). Per capita surface water availability was 5,260 m³ per year in 1951 which has reduced to 960 m³ in 2017. The position is worsening and with rapidly increasing population, water availability may further drop to about 860 m³ by 2025 representing "acute water short conditions". The threshold water requirement to avoid food and health constraints due to be a "water short country" is 1,700 m³ per capita per year. Pakistan has thus reached the stage of "acute water shortage", where, people fight for every drop of water.

In addition, the storage capacity of existing reservoirs is reducing rapidly due to sedimentation. However, it is worth mentioning that with Mangla Dam Raising the storage capacity of 3.6 BCM (2.9 MAF) has been added to the reservoirs capacity. However, it has not fully replenished the lost storage capacity till 2017. Further additional storages, therefore, have to be created by building major storage dams (Basha, Kalabagh and Akhori) to replenish the lost water storage capacity and reduce flood damages and to regulate stored water as per national needs.

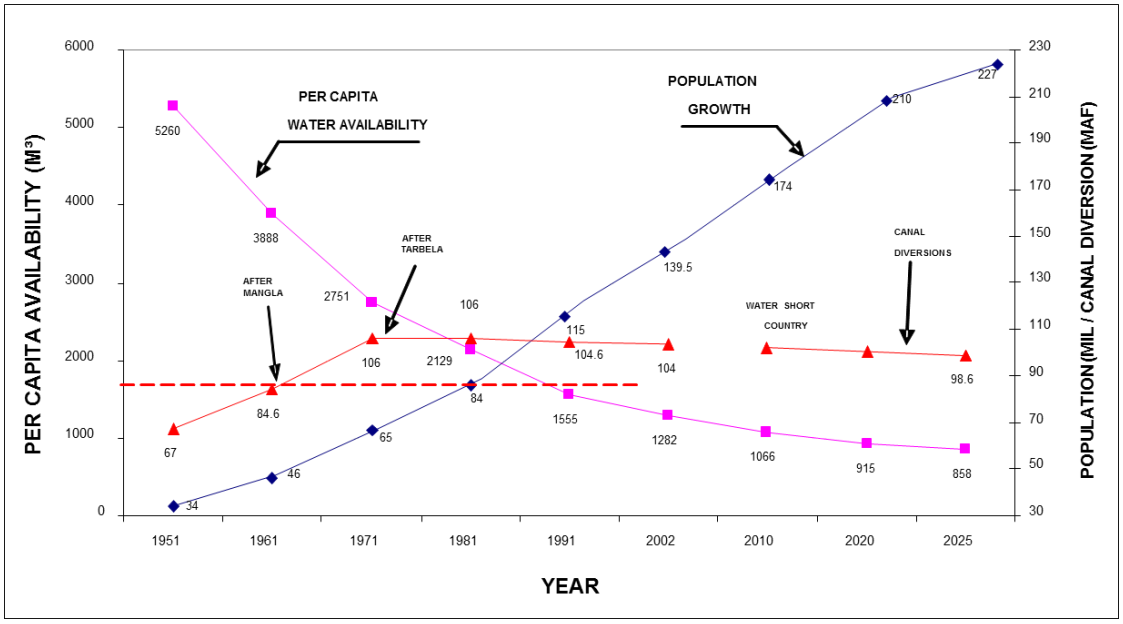



**Figure 3:** Water Availability and Canal Water Diversion v/s Population Growth.

Water Sector Investment Planning Study (WSIP, 1990), carried out by WAPDA, indicated that Pakistan has faced a deficit of 12 million tons in total grain production in the year 2012-13. This large scale deficit was bridged by improving farming practices, on-farm water management and recently some storage addition of Mangla Dam Raising. The irrigation supplies scenario in the year 2016-17 has reached a critical stage which will further deteriorate year-by-year due to increasing population and reduction in storage capacity, emphasizing the need for construction of more water storages.

## 2. GLOBAL CLIMATE CHANGE

**Climate Change Studies / Monitoring:** Climate Change phenomena is being studied by different agencies and organization i.e., Inter-governmental Panel on Climate Change

(IPCC), Geneva, Switzerland, World Monitoring Organization (WMO), United Nations Environment Protection (UNEP) Agency, USA University of Wisconsin, Climate Change ministries of various countries through their Environment Protection Agencies (EPAs) and Hydrological Studies Centers (HSCs).

**The Global Temperature Increase:** Globally, the 1990's was the warmest decade and 1998 the warmest year since 1861. New analyses of data for the Northern Hemisphere indicate that the increase in temperature in the 20th century is likely to be the largest of any century during the past 1,000 years. The global average surface temperature has increased by +0.15 ± 0.05°C per decade. The record shows a great deal of variability; for example, most of the warming occurred during the 20th century, during two periods, 1910 to 1945 and 1976 to 2000. The Earth had warmed by an average of more than 1.7$^{o}$C over the past 150 years, according to an analysis of the University of Wisconsin Study "freeze and- thaw" records for lakes and rivers in the Northern Hemisphere.

## 3. REASONS FOR GLOBAL TEMPERATURE INCREASE:

### 3.1 Variation of Solar output Effect

The Sun is the primary source of energy responsible for governing both the weather and the climate of Earth. For that reason, the amount and type of energy the earth receive from the Sun could alter weather and climate on the earth. There are variations in the energy that the Earth receives from the Sun, both in the form of total electromagnetic radiation and other types of energy, such as outflows of charged particles. The total radiant power received by the Earth from the sun, was about 0.036 percent higher in 1996 than in 1986. Meteorological and climatic data suggest that there are significant responses in the Earth's atmosphere and oceans in the form of drought cycles, variations in global sea-surface temperatures, etc.

### 3.2 Greenhouse Effect

The greenhouse effect is produced as certain gases allow incoming solar radiation to pass through the Earth's atmosphere, but prevent part of the outgoing infrared radiation from the earth's surface from escaping into outer space. Greenhouse gases include water vapour, carbon dioxide ($CO_2$), methane ($CH_4$), nitrous oxide ($N_2O$), halogenated fluorocarbons (HCFCs), ozone ($O_3$), perfluorinated carbons (PFCs), and hydrofluorocarbons (HFCs). Major sources of greenhouse gases include; burning of fossil fuels (gas, oil, and coal), cutting of natural forests, and agriculture straw in wheat, rice and sugarcane fields. The average concentration of $CO_2$ was about 290 ppm (parts per million) in pre-industrial times. In 1990 it was about 350 ppm and is increasing steadily from 0.3–0.4%/yr. Although these climate forecasts are fairly dependable, most scientists agree that much remains to be done to improve climate models, before we will be able to predict future climate changes in a credible manner for adopting mitigation measures.

### 3.3 El Niño and LA Niña

El Niño phenomenon is the result of persistent higher Sea Surface Temperatures (SSTs) in the central and eastern tropical Pacific; the magnitude of the warming determines the magnitude of El Niño. In 1997, Sea Surface Temperatures recorded in some of these tropical Pacific waters were more than 5$^{o}$C (9$^{o}$F) higher than their average value. Past climate data reveal that El Niños have been a recurring feature of the earth's climate for thousands of years; however they have now become more persistent.

El Niño is the warm phase of El Niño / Southern Oscillation phenomena (ENSO), and LA Niña is the cold phase. Both involve the tropical oceans and the atmosphere, and the exchange of energy between the ocean surface and the air above it. The ocean and the atmosphere are linked together, interactively, and each affects the other. Atmospheric winds push the ocean currents and help determine the patterns of sea surface temperature. But Sea Surface Temperatures, at the same time, help determine the force and direction of winds and atmospheric circulation, by adding heat to tropical land areas (10°N to 10°S). Rainfall has decreased over much of the Northern Hemisphere sub-tropical land areas (10°N to 30°N) during the 20th century by about 0.3% per decade. This is the region in which most of Pakistan is situated.

In contrast to the Northern Hemisphere, no comparable systematic changes have been detected in broad latitudinal averages over the Southern Hemisphere. There are insufficient data to establish trends in precipitation over the oceans. In the mid- and high latitudes of the Northern Hemisphere over the latter half of the 20th century, there has been a 2 to 4% increase in the frequency of events of heavy precipitation. Increases in heavy precipitation events can arise from a number of causes, e.g., changes in atmospheric moisture, thunderstorm activity and large-scale storm activity. In most areas, there is an observed decrease in daily temperature range. Since 1950, there has been a reduction in the frequency of extreme low temperatures, with a small increase in the frequency of extreme high temperatures.

### 3.4 Snow Cover And Ice Extent

Satellite data show that there have been decreases of about 10% in the extent of snow cover since the late 1960s. Ground-based observations show that there has been a reduction of about two weeks in the annual duration of lake and river ice cover in the mid- and high latitudes of the Northern Hemisphere in 20th century. There has been a widespread retreat of mountain glaciers in non-polar regions during the 20th century. Northern Hemisphere spring and summer sea-ice extent has decreased by about 10 to 15% since the 1950s. There has been a decline in Arctic sea-ice thickness of about 40% in recent decades and a considerably slower decline in winter sea-ice thickness. It may be remembered that increase and decrease in ice-cover does not depend on winter temperature. It is the summer temperature that determines the extent of ice cover. If the summer is warm, all the accumulated ice is melted and the ice cover decreases or remains constant. However, if the summer is mild, the previous season's ice is carried over to the next season and the ice cover increases.

### 3.5 Sea Level Rise and Ocean Heat Content

Tide-gauge data show that global average sea-level rose between 0.1 and 0.2 meters during the 20th century. Global Ocean heat content has increased since the late 1950s, the period for which adequate observations of sub-surface ocean temperatures have been available. The average temperature of Atlantic, Pacific and Indian Oceans has risen by 0.06°C during last 60 years.

## 4. EFFECTS OF GLOBAL WARMING IN THE ASIAN REGION

The Asian region that spans polar, temperate, and tropical climates and is home of over three billion people, is a particular concern as far as effects of global warming are concerned. As climate warms, many mountain glaciers may disappear and the northern forests may shift further north. A report by IPCC sponsored by United Nations Environment Protection Agency (UNEP) predicts that the major impacts in Asia under global climate change may cause large shifts of the natural forests, the disappearance of

significant portions of mountain glaciers and shortages of water supply. The most critical uncertainty in these estimates stems from the lack of credible projections of the hydrological cycle under global climate change scenarios. The effects of climate change on the Asian monsoon and the El Niño / Southern Oscillation (ENSO) phenomenon are among the major uncertainties in predicting the water picture in the region. The positive ENSO phase of the pattern (wet south and dry north) is promoted by warm Sea Surface Temperatures in the tropical Pacific and Indian Oceans. A rather abrupt change in the ENSO behavior occurred around 1976/77 and the new regime has persisted. There have been relatively more frequent El Niño episodes as compared to LA Niña.

This behavior is highly unusual in the last 140 years (the period of instrumental record). Although global temperature increase has occurred over the last century, clear signs of its effects were noted in the late 1970s, with the more frequent appearance of El Niños. The explanation is that, the global warming influence was, until that time insufficient to perturb the normal working of the climate system: its capacity to affect the overall behavior of the climate system was reached only after a certain threshold had passed. The major risk from the climate change in south Asia, is increased summer precipitation intensity in temperate regions. This may increase flash-flood prone areas. Out of all natural disasters, floods are the most destructive. In terms of human life and property they account for over 50 % fatalities (58% deaths were due to floods in the decade 1988-1997). In terms of economic loss, they account for one third of total losses due natural disasters. On the other hand, the arid and semi-arid regions would be drier in summer, which could lead to severe droughts. The Himalayas have a critical role in the provision of water to continental monsoon in Asia. Increased temperatures and increased seasonal variability in precipitation are expected to result in increased recession of glaciers and increasing danger from glacial lake outburst floods.

## 4.1 Effect of Global Warming on Water Resources

Global weather changes and water resources are deeply inter-related. The largest source of freshwater is rain. Global climatic changes will have major effects on precipitation and runoff. In the relatively arid and semi-arid regions, modest changes in precipitation can have proportionately large impacts on water supplies. In mountainous watersheds, higher temperatures will increase the ratio of rain to snow, accelerate the rate of spring snowmelt, and shorten the overall snowfall season, leading to more rapid, earlier, and greater spring runoff because the temperature projections of climate models are less speculative than the projections of precipitation. Temperatures induced shifts in the relative amounts of rain and snow and in the timing of snowmelt in mountainous areas are considered likely. Climate induced changes in hydrology will affect the magnitude, frequency, and costs of extreme events. Flooding could become more frequent and extreme. Recent reports of the IPCC suggest that a greenhouse warming is likely to increase the number of intense precipitation days as well as flood frequencies in northern latitudes and snowmelt driven basins. These reports also suggest that the frequency and severity of droughts could increase in some areas, as a result of a decrease in total rainfall and more frequent dry spells.

**A macro-scale hydrological model for river-flow suggests that the runoff of the Indus will decrease by 27% by the year 2050. This implies that the availability of fresh water in Pakistan is highly vulnerable to climate change. A reduction in average flow of snow-fed rivers, coupled with an increase in peak flows and sediment-yield would have major impacts on hydropower generation and agriculture. Availability of water from snow-fed rivers may increase in the short**

**term, but decrease in the long-run. Runoff from rain-fed rivers may also change in the future. A reduction in snowmelt water will put the dry-season flow of these rivers under more stress than is the case now, especially in Pakistan where one major snow-fed river, the Indus, accounts for as much as 80 % of the normal water flow. Increased population and increasing demand for the agricultural, industrial and hydropower sectors will put additional stress on water resources.**

## 5. WATER RESOURCES OF PAKISTAN

5.1 **Water Resources:** The water resources of the country are natural precipitation, surface water and groundwater. In the arid to the sub-tropical climate of the country, the natural precipitation is scanty. Half of the country area receives less than 200 mm of annual rainfall, and rainfall in excess of 400 mm occurs in about 20 percent of the northern areas. Apart from being scanty, the precipitation is distributed quite unevenly over the seasons and in a major part of the country this is concentrated in 3 to 4 months of summer monsoon. Despite its meager amount, rainfall is utilized for rain-fed agriculture and for meeting the drinking water needs of the people and livestock alongwith industrial and environmental requirements.

5.2 **Surface Water:** The sources of surface water available to Pakistan are its rivers. Most of these rivers, flow in the western half of the country which are ephemeral streams that have quite low flows for most of the year. Besides this the Indus River and its tributaries have perennial flows that constitute the main source of water supply. The Indus and its tributaries, have their sources in the Himalayan and the Hindu Kush mountains, with a total drainage area of 944,569 $km^2$, which extends beyond the country's territorial limits. The inflow to these rivers is mainly derived from snow and glacier melt and rainfall in the catchment areas. The tributaries of the Indus, originating in India but flowing into Pakistan are Jhelum, Chenab and Ravi and Sutlej with little flows in monsoon. Kabul river originating from Afghanistan, is another major tributary of Indus.

Under the Indus Waters Treaty 1960, the flows of the three Eastern Rivers, (Sutlej, Beas and Ravi) have been allocated to India, whereas, Pakistan is entitled to all the waters of the Western Rivers (Indus, Jhelum and Chenab). The average annual inflow of the Western Rivers at the rim stations (the most important gauging stations: Indus at Kalabagh, Jhelum at Mangla and Chenab at Marala), as these enter the Indus Plain is 172.70 BCM (140 MAF). This constitutes the main source of surface water supply for the country.

Although the surface flows of the Indus river and its tributaries available to Pakistan are quite significant, these are highly variable seasonally and yearly. Against the average annual inflow of 172.70 BCM (140 MAF), the long term historic data indicates a high river inflows ·in the order of 230.42 BCM (186.79 MAF) or 34% above the average inflows in 1959-60, and a low inflows of 120.57 BCM (97.74 MAF) or 30% below average in 1974-75.The inflows during the six months of summer cropping season of Kharif and during remaining six months of Rabi season are 84% and 16% of the average annual inflows respectively. However, there was a huge flood in 2010 with inflows of about 211.0 BCM (171.0 MAF) or 31% above average inflows with death toll of over 1980 people in the flooded area of 160,000 $Km^2$. The high flow variability indicates a compelling need for storing and regulating the high and surplus summer flows for use during low flow in winter season to meet irrigation water needs and to generate cheaper hydel power. The construction of additional water storages will also help to mitigate flood damages and meet enhanced irrigation, domestic and industrial needs.

Table-1 indicates historical flood events experienced during 1950 – 2016, and Figure-4 shows some of the flood conditions.

**Table1:** HISTORICAL FLOOD EVENTS EXPERIENCED IN PAKISTAN

| Sr. No. | Year | Direct losses (US$ million) @ 1US$= PKR 86 | Lost lives (No) | Affected villages (No) | Flooded area (Sq-km) |
|---|---|---|---|---|---|
| 1 | 1950 | 488 | 2,190 | 10,000 | 17,920 |
| 2 | 1955 | 378 | 679 | 6,945 | 20,480 |
| 3 | 1956 | 318 | 160 | 11,609 | 74,406 |
| 4 | 1957 | 301 | 83 | 4,498 | 16,003 |
| 5 | 1959 | 234 | 88 | 3,902 | 10,424 |
| 6 | 1973 | 5134 | 474 | 9,719 | 41,472 |
| 7 | 1975 | 684 | 126 | 8,628 | 34,931 |
| 8 | 1976 | 3485 | 425 | 18,390 | 81,920 |
| 9 | 1977 | 338 | 848 | 2,185 | 4,657 |
| 10 | 1978 | 2227 | 393 | 9,199 | 30,597 |
| 11 | 1981 | 299 | 82 | 2,071 | 4,191 |
| 12 | 1983 | 135 | 39 | 643 | 1,882 |
| 13 | 1984 | 75 | 42 | 251 | 1,093 |
| 14 | 1988 | 858 | 508 | 100 | 6,144 |
| 15 | 1992 | 3010 | 1,008 | 13,208 | 38,758 |
| 16 | 1994 | 843 | 431 | 1,622 | 5,568 |
| 17 | 1995 | 376 | 591 | 6,852 | 16,686 |
| 18 | 2010 | 10,000 @ 1US$= PKR 86 | 1,985 | 17,553 | 160,000 |
| 19 | 2011 | 3730* @ 1US$= PKR 94 | 516 | 38,700 | 27,581 |
| 20 | 2012 | 2640** @ 1US$= PKR 95 | 571 | 14,159 | 4,746 |
| 21 | 2013 | 2,000^ @ 1US$= PKR 98 | 333 | 8,297 | 4,483 |
| 22 | 2014 | 440^^ @ 1US$= PKR 100.89 | 367 | 4,065 | 9,779 |
| 23 | 2015 | 170# @ 1US$= PKR 105.00 | 238 | 4,634 | 2,877 |
| 24 | **2016** | **6#** @ 1US$= PKR 104.81 | **153** | **45** | - |
| **Total** | | **38,171** | **12,330** | **197,275** | **616,598** |

*Sources:* ** Economic Survey of Pakistan 2011-12*
*** NDMA (http://www.claimsjournal.com/news/international/2012/10/05/214891.htm)*
*^ Thomson Reuters Foundation (http://www.trust.org/item/20130909134725-rm708/)(Agriculture sector)*
*^^ Economic Survey of Pakistan (2014-15)*
*# Based on PIDs & FATA's interim reports related to irrigation, drainage & flood protection infrastructure only*



**Figure – 4:** Flood 2010 Conditions at Various Places

5.3 **Groundwater:** It is another important source of water supply. Investigations have established the existence of a vast aquifer with an aerial extent of 194,000 $km^2$ underlying the Indus Plain which has been recharged in geologic times from natural precipitation and river flows, and more recently by the seepage from the canal irrigation system and during field application. Although the quality of the groundwater in the Indus Basin is highly variable both aerially and depth, it is estimated that 67.85 BCM (55 MAF) of groundwater, representing the safe yield, could be withdrawn annually for beneficial uses. Over 61.68 BCM (50 MAF) is being pumped presently for irrigation, domestic and industrial uses.

5.4 **Water Demand:** Additional demand of water for the anticipated improvements in the standard of living and greater economic activities will have to be fulfilled. Our underground water resources are already being exploited to the maximum. Increase in sustainable water resources in the country can be achieved through increased exploitation of groundwater aquifers and building additional large, medium and small surface water storage dams. This combination will be critical to meeting the ever increasing water needs of the 21st century.

## 6. Sedimentation of Water Storage Reservoirs

Pakistan has three major reservoirs, with total storage capacity of 23.0 BCM (18.64 MAF) in 2012. The storage capacity of Tarbela, Mangla and Chashma reservoirs has depleted by 6.16 BCM (4.99 MAF) or **27%** by the year 2017 due to sedimentation. It is estimated that the gross storage capacity would be further reduced by 7.40 BCM (6.00 MAF) or 32% by the year 2025 as shown in Table-2.

**Table2:** Storage Loss Due To Sedimentation

| Reservoir (Commissioning Year) | Storage capacity | | | Storage loss | |
|---|---|---|---|---|---|
| | Original | Year 2017 | Year 2025 | Year 2017 | Year 2025 |
| | BCM / MAF | BCM/MAF (%) | BCM/MAF (%) | BCM/MAF (%) | BCM/MAF (%) |
| Tarbela (1976) | 11.94 / 9.68 | 7.48 / 6.06 (63%) | 6.63 / 5.37 (56%) | 4.47 / 3.62 (37%) | 5.31 / 4.3 (44%) |
| Mangla (1967) | 6.6 / 5.34 | 5.37 / 4.35 (82%) | 5.17 / 4.19 (79%) | 1.22 / 0.99 (18%) | 1.42 / 1.15 (21%) |
| Chashma (1971) | 0.88 / 0.72 | 0.47 / 0.38 (53%) | 0.33 / 0.27 (38%) | 0.42 / 0.34 (47%) | 0.55 / 0.45 (62%) |
| **Sub-Total** | **19.42 / 15.74** | **13.31 / 10.79 (69%)** | **12.13 / 9.83 (63%)** | **6.05 / 4.95 (31%)** | **7.29 / 5.91 (37%)** |
| Addition of Raising Mangla (2012) | 3.6 / 2.90 | 3.53 / 2.86 (99%) | 3.47 / 2.81 (97%) | 0.05 / 0.04 (1%) | 0.11 / 0.09 (3%) |
| **Total** | **23.00 / 18.64** | **16.84 / 13.65 (73%)** | **15.60 / 12.64 (68%)** | **6.16 / 4.99 (27%)** | **7.40 / 6.00 (32%)** |

Source: WRM Directorate, WAPDA

## 7. Water Requirements of Water use Sectors

It is estimated that Pakistan's population of 200 million in the year 2017 would rise to about 242 million by the year 2030. The anticipated population rise, rapid urbanization and· better socio-economic conditions would bring additional pressure on water resources. The existing and future water requirements since 2000 to 2030 for all sectors are given in Table – 3.

**Table 3:** Future Water Requirements at Farm Gate (MAF)

| Sector | Year | | | | | |
|---|---|---|---|---|---|---|
| | **2000*** | **2010**** | **2017**** | **2020**** | **2025*** | **2030**** |
| Agriculture | 99.0 | 104.0 | 109.0 | 114.0 | 119.0 | 124.0 |
| Water Supply & Sanitation | 4.5 | 6.0 | 7.5 | 9.0 | 10.5 | 12.0 |
| Industry | 3.5 | 3.8 | 4.1 | 4.5 | 4.8 | 5.1 |
| Environmental Protection | 1.3 | 1.4 | 1.5 | 1.6 | 1.7 | 1.8 |
| **Total** | **108.3** | **115.2** | **122.1** | **129.1** | **1.36.0** | **142.9** |

*Source: *For Year 2000 & 2025 National Water Policy Vol II, January 2003*
***For years 2010, 2017, 2020 & 2030 computed on proportional basis.*

The existing and future demand of water to meet water requirements for the period 2000 to 2030 for all sectors are described below:

**7.1 Agriculture Use:** The total area of the country is 79.61 Mha (196.72 Ma) of which 29.4 Mha (72.65 Ma) is designated as cultivatable area. About 15.4 Mha (38.1 Ma) cultivated land is served by canal irrigation system, 5.2 Mha (12.85 Ma) is tubelwell & rain fed, while the remaining 8.8 Mha (21.75 Ma) area is barren, needing additional irrigation water. Over 90 percent of available water is used to meet crop water demand. Increases in agricultural production to meet the needs of a rising population, will require additional water. Based on population growth projections, by 2030, an estimated additional 25.9 BCM (21 MAF) water as compared with 2017 will be required for agriculture, assuming a 50 percent increase in crop yields from non-water inputs.

**7.2 Municipal Use:** The current total water use for domestic and municipal purposes in both urban and rural areas is estimated at 5.6 BCM (4.5 MAF). By 2030 requirements for water supply, rural potable water and sanitation is estimated to be 14.81 BCM (12.0 MAF) indicating an increase of 9.26 BCM (7.5 MAF).

**7.3 Industrial Use:** There are over half a million large and small industrial units in the country, of which nearly 120,000 are engaged in textile, chemical, fertilizer, tanneries and other manufacturing and processing activities. The water use by all industries in year 2010 was estimated to be 4.7 BCM (3.8 MAF). This is expected to rise to 6.3 BCM (5.1 MAF) by 2030, i.e. an additional requirement of 1.6 BCM (1.3 MAF).

**7.4 Environmental Use:** It is estimated that in order to ensure adequate water throughout Pakistan for wetlands, environmental protection and increased irrigated forestry, about 2.2 BCM (1.8 MAF) water will be required by the year 2030 compared to 1.9 BCM (1.5 MAF) in the year 2017.

**7.5 Water Threshold Value:** Water availability in Pakistan is 960$m^3$/capita/year in 2017. This is well below 1,700 $m^3$ per capita/year threshold value for water stressed condition.

Thus Pakistan is already fast moving into a condition of severe ‘water scarcity'. This situation is likely to deteriorate in future as the gap between supply and demand widens. However, the gross additional water demand (at the farm gate) for all sectors over year 2010 was about 34.2 BCM (27.7 MAF i.e. 24.7 BCM (20 MAF) for agriculture and 9.5 BCM (7.7 MAF) for municipal water supply, rural potable and sanitation, industry and the environment). The corresponding requirement at the canal head (including provision for system losses where applicable) would be nearly 48.1 BCM (39 MAF). Water available for future development is about 38.3 BCM (31 MAF) including 26.7 BCM (21.6 MAF) of river flow, 7.9 BCM (6.4 MAF) from groundwater and 3.7 BCM (3 MAF) from rainfall

harvesting). This shortfall of about 48.1 BCM (39 MAF) of water by the year 2030 is to be met by building additional storage reservoirs and diverting 26.7 BCM (21.6 MAF) to canals which at present is flowing to sea after allowing 10.6 BCM (8.6 MAF) for environmental uses.

## 8. Water Escapages Below Kotri

Un-captured Water: Despite acute water shortage in the system, data shows that a substantial amount of water escapes below Kotri to the Arabian Sea. Post-construction of Tarbela (1976-20l7) average annual escapages below Kotri are 38.9 BCM (31.5 MAF), with a maximum of 113.32 BCM (91.83 MAF) in 1994-95 and minimum of 0.98 BCM (0.79 MAF) in 2000-01. Most of the flow to the sea occurs during Kharif season and very little during Rabi season. For better water management, storage capacity should be equivalent to at least 40 percent of total water availability, but Pakistan's live storage capacity is about 7 percent. Pakistan needs to create a minimum of 80.2 BCM (65 MAF) of storage for effective resource management and meeting national additional water demands.

## 9. CLIMATE CHANGE RISKS & IMPACTS

The climate change impact, on water availability due to global warming, is becoming a major concern for the researchers and the policy making personnel dealing with water resources. The main impacting factor is temperature that is causing increased melting of glaciers, snow melt hydrology and precipitation. Initially these factors would create an environment to enhance river inflows and resultantly more flood frequency and flood peaks. Later on river inflows would be causing more increasing water shortages to meet with crop water requirements, domestic, industrial and environmental needs, as discussed below:

### 9.1 Temperature

The 4th Assessment Report (AR4) by the three working groups of the IPCC has projected that average global surface temperature will increase by approximately over 4°C during 21st century. Following information is derived for Pakistan for two IPCC scenarios, A1B (very rapid economic growth, population growth peaking mid-century and balanced use of energy sources), and A2 (fragmented and slow economic growth and continuous population growth).

The temperature increase in Northern Pakistan is larger than that in Southern Pakistan, in line with the IPCC global scenarios, which show higher temperature increase over Central Asia than that over South Asia. The annual average temperatures for Northern and Southern Pakistan are about 19°C and 24°C respectively. The maximum temperatures (Tmax $^{o}$C) in northern area of Pakistan for the period 2004-2015 as recorded by the Ice & Snow Melt Hydrology Directorate of WAPDA at various sites as shown in Figure-5, are given in Table – 4. Average variation of Tmax $^{o}$C at these sites is shown in Figure-6. The temperature increase in Northern and Southern Pakistan in each scenario is higher than the corresponding globally averaged temperature increase (for A2 scenario, the projected temperature increases by 2080s in Northern and Southern Pakistan will be 4.67°C and 4.22°C, respectively as shown in Figure-7.

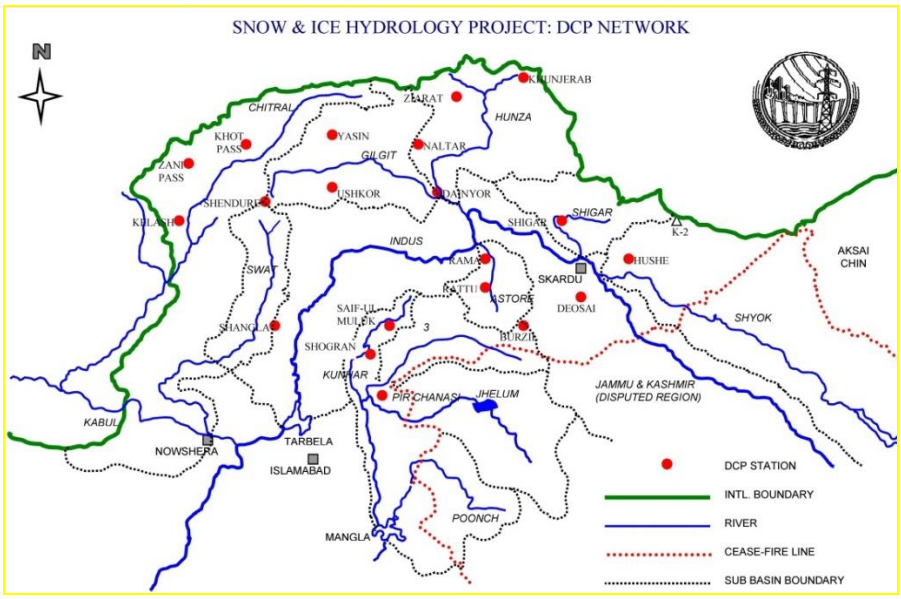



**Figure- 5:** Snow & Ice Hydrology Network of WAPDA

**Table 4:** Maximum Temperature (Tmax) at Various Sites in Northern Areas of Pakistan (°C)

| Location | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | 2015 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Khunjerab | -0.4 | -1.3 | 0.2 | -0.1 | -0.6 | -1.0 | 0.0 | -0.1 | -2.0 | -0.6 | -0.8 | -1.5 |
| Naltar | 11.0 | 10.2 | 11.7 | 11.7 | 11.2 | 10.2 | 10.1 | 11.2 | 8.8 | 9.6 | 9.4 | 8.1 |
| Yasin | 8.9 | 8.3 | 9.5 | 9.3 | 9.1 | 7.9 | 7.6 | 0.6 | 4.8 | 0.0 | 0.0 | 7.0 |
| Ziarat | 7.7 | 7.3 | 7.3 | 8.2 | 8.1 | 6.7 | 7.8 | 6.8 | | 5.2 | 7.2 | 5.8 |
| Rama | 10.0 | 9.0 | 10.1 | 10.4 | 9.7 | 8.7 | 2.0 | 9.7 | 2.3 | 1.2 | 0.8 | 6.7 |
| Deo | 1.8 | 1.5 | 3.1 | 2.5 | 1.8 | 1.0 | 1.6 | 1.8 | | 0.5 | 0.7 | 0.2 |
| **Average** | **6.50** | **5.83** | **6.98** | **7.00** | **6.55** | **5.58** | **5.82** | **5.00** | **3.48** | **3.18** | **3.46** | **4.38** |

**Source:** Ice & Snow Melt Hydrology Directorate, WAPDA

The Table-4 and Figure 6 show decreasing trend in average maximum temperatures ranging from 5.82 to 3.18°C for the period 2009-2015 as compared to the period 2004 – 2008 (7.00 to 5.83 °C). There was a temperature dip in recent past years; however, temperature in the northern glaciating area is rising since 2014 – 2015.

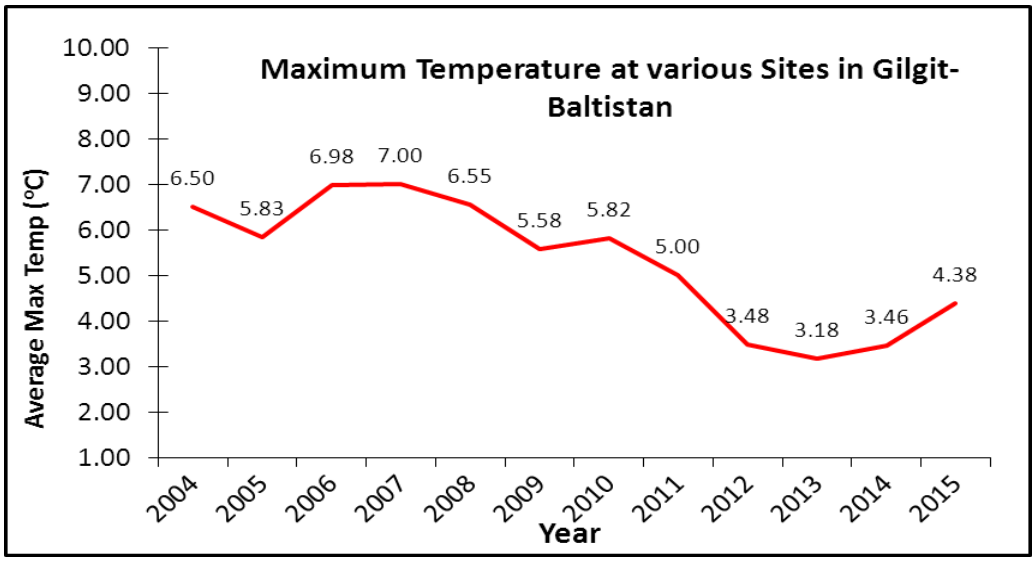



**Figure-6:** Indicating Average Tmax in ºC At Various Sites
(Ice & Snow Melt Hydrology WAPDA)

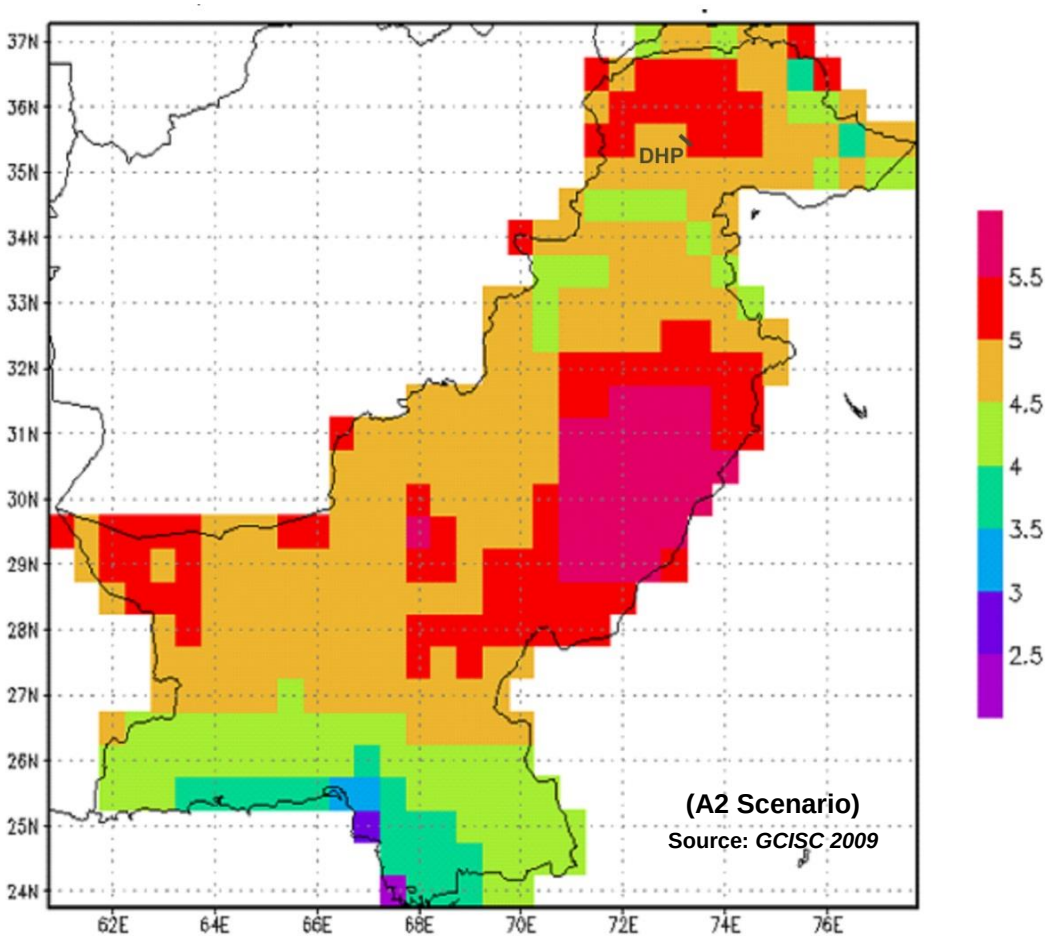



**Figure 7:** Projected Temperature Change (°C) for 2080s

## 9.2 Melting of Glaciers

About 50-80% of the average river inflows in the Indus River System (IRS) are fed by snow and glacier melt in the Hindu Kush-Karakoram (HK) part of the Hindu Kush-Karakoram-Himalayan (HKH) mountain ranges. After the Arctic/Greenland and Antarctic glaciers, the HKH represent the third largest ice mass on earth. These glaciers constitute about 2,700 $km^3$ of stored volume of ice (Roohi, 2005), equivalent to about 14 years of average IRS inflows.

Over the past century, glaciers worldwide have exhibited a receding trend; the HKH glaciers are receding faster than in any other part of the world. It is expected that if the present rate of recession continues, the HKH glaciers might be reduced to a significant extent by 2035 (Rees, G. and D.N. Collins, 2004). The thermal regime of HKH glaciated region has in general warmed up and the frequency of occurrence of moderate as well as severe heat waves have also increased significantly (Pakistan Meteorological Department - PMD, 2009). Preliminary analysis of the time series data on flows of the Indus and its tributaries has not indicated any large melting of glaciers so far (Global Change Impact Studies Center - GCISC, 2009). However, based on the studies available, it looks likely that the HKH glaciers are receding under the influence of global warming and that melting will further increase with increase in summer temperature.

The simulation modeling conducted by GCISC on river Indus flows for a scenario where temperature will rise by 4°C, the glaciers may shrink to half their present size. As a result the overall annual flow would reduce by about 15% and the monthly flow pattern would also change considerably, with more water coming in spring and early summer and less water in the later part of summer (Figure 8).

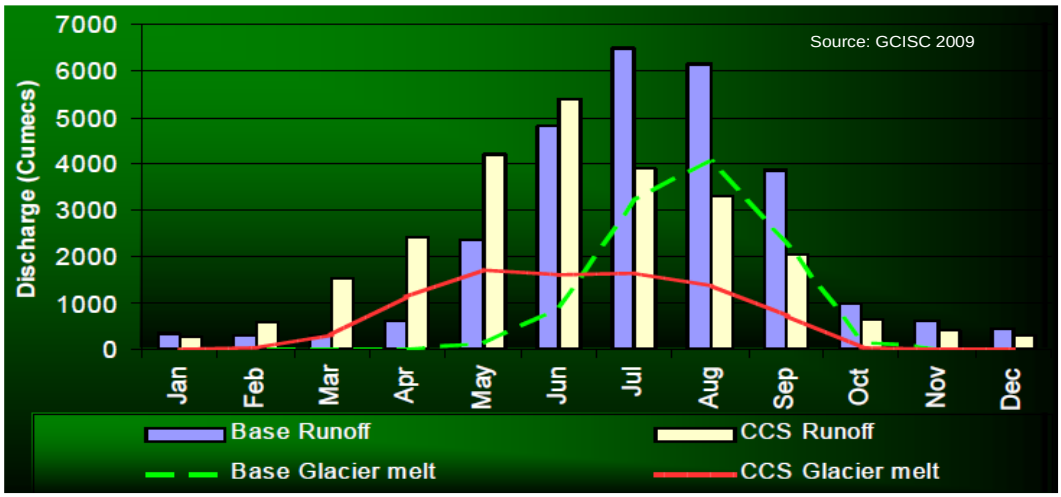



**Figure 8:** Base Flows and Future Flows in Indus at Besham Qila

## 9.3 Glacier Monitoring in Tarbela Catchment:

Climate Change has brought new challenges to the water resources of Pakistan which mainly lies in Upper Indus Basin (UIB) especially in Tarbela catchment. They are in the form of Glaciers and perennial snow which are annually nourished in winters from westerlies and supply waters to the Indus plains during summers. Their annual contribution to the River Indus is more than 70%. It is reported in the international literature that the glaciers of Himalayas are receding faster and the average annual water supply from these glaciers may reduce by 30 to 40 percent in coming 50 years. This is an alarming situation for Pakistan, which largely depends on the perennial flows

from UIB Rivers for its irrigated agriculture, which contributes 21% towards GDP and directly supports over 50% of its population. Realizing the importance of River Indus and a probable threat of Climate Change to its flows, it is imperative to take up a comprehensive study to ascertain the present status of our glaciers and possible risks & impacts of Climate Change in relation to future water availability from these.

Therefore, "Monitoring of UIB Glaciers for Water Resources Management and Climate Change" is essentially needed. For this purpose, an independent “Glaciers Monitoring and Research Centre" may be established.

A glacier monitoring study was commenced in July 2012 by WAPDA. It is providing forecasts on the long-term future water availability to Tarbela Reservoir of Climate Change. Climate Change is a slow acting long term phenomenon, hence requires long term commitment for its understanding.

## 9.4 Precipitation

An overall increase in rainfall of 50-150mm has been observed in the monsoon belt during the period 1931-90. However, a decrease in rainfall was seen in rest of the country. The decrease is 50-100 mm in western and northern mountains, whereas in the southeastern and central parts it is 25-50 mm. **These variations are in line with the predictions made on the expected effects of global warming.** Global warming is expected to cause drier south and wetter north. Changes in El.Nino/Southern Oscillation (ENSO) also affect Asia, especially the monsoon. Based on the monsoon rainfall pattern from 1901 to 90, a strong El Niño event causes poor or below normal rainfall in Pakistan. However, when a strong El Niño year is followed by another strong El Niño year, the rains return to normal or above average. When a strong El Niño is followed by a moderate El Niño in the succeeding year, the monsoon activity is reduced. Most of the floods in Pakistan have been recorded in the La Niña years. The predominance of El Niño in recent years may be one of the causes of the recent past dry years (Imtiaz, 2005).

The precipitation in northern area of Pakistan for the period (2004-2015) as recorded by the Ice & Snow Melt Hydrology Department of WAPDA at various sites are shown in Figure - 5 and are indicated in Table – 5. An average variation of precipitation on six stations (Khunjrab, Naltar, Yasin, Ziarat, Rama and Deo) is given in Figure – 9.

**Table 5:** Precipitation (mm) at Various Sites in Northern Areas of Pakistan

| Location | 2004 | 2005 | 2006 | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 | 2015 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Khunjerab | 173.1 | 178.3 | 215.7 | 154 | 165.2 | 159.6 | 334.6 | 329.7 | 406.6 | 183.1 | 198 | 244.3 |
| Naltar | 651.8 | 728.1 | 802.7 | 585.2 | 564.7 | 711.4 | 758.4 | 950.5 | 755 | 314.4 | 426.7 | 466.2 |
| Yasin | 351.3 | 305.2 | 308.3 | 297.5 | 284.3 | 569.8 | 638.8 | 34.7 | 413.5 | 0 | 0 | 304.3 |
| Ziarat | 577.5 | 158.2 | 110.5 | 156 | 130 | 196.2 | 501.9 | 395.1 | | 517.5 | 182.8 | 170.7 |
| Rama | 1048 | 810.9 | 540.6 | 464.8 | 844.1 | 820.1 | 801.4 | 774.8 | 296 | 76.7 | 59.4 | 668.7 |
| Deo | 272.2 | 298.3 | 404.2 | 278.3 | 348.4 | 298.8 | 501.8 | 544.6 | | 542.5 | 534 | 755.2 |
| **Average** | **512.35** | **413.17** | **397.00** | **322.63** | **389.45** | **459.32** | **589.48** | **504.90** | **467.78** | **326.84** | **280.18** | **434.90** |

**Source:** Ice & Snow Melt Hydrology Directorate, WAPDA

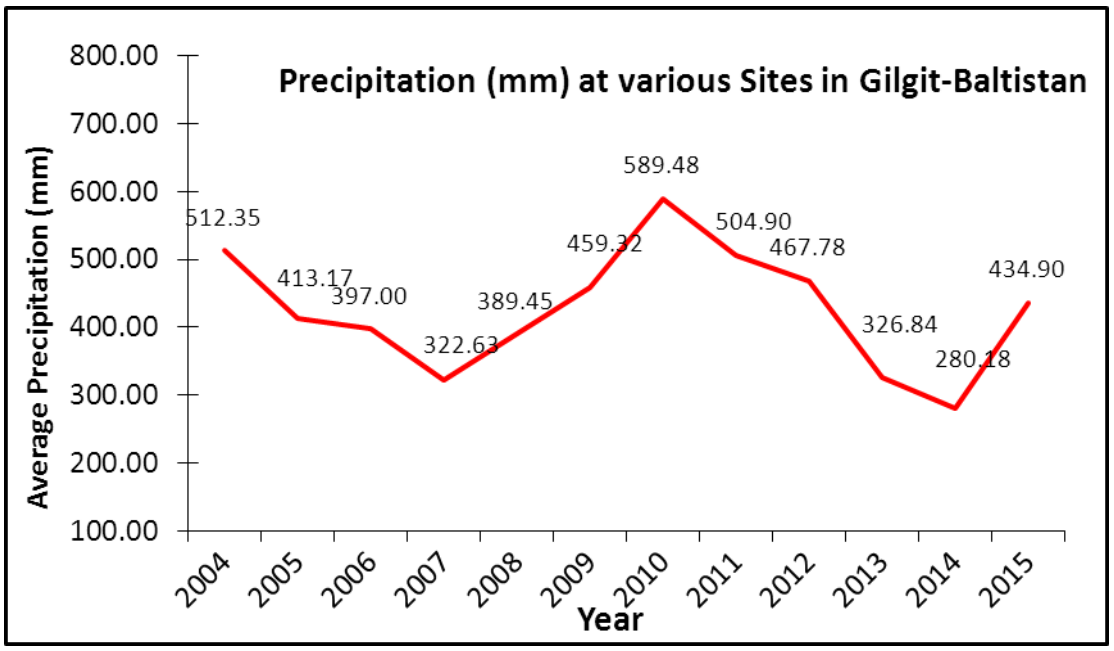



**Figure - 9:** Indicating Average Precipitation (mm) At Various Sites
(Ice & Snow Melt Hydrology Directorate, WAPDA)

The Table-5 indicates that there was high precipitation that is 589.48 mm in the year 2010 and consequently a severe flood occurred with massive economic loss and death toll over 1980 persons in the country.

## 10. WATER CRISIS IN PAKISTAN

a) The persistent low trend of rains in recent years has led to persistence of droughts in most of the country. Droughts in one country or region also affects neighboring regions that otherwise are supposed to make surplus water available for inter-basin transfers in the flood seasons. Increase in instances of drought requires a re-examination of our assumptions, operating norms and contingency measures for existing and planned water-management measures. Droughts increase migration of population.

b) Besides social disruptions, there is the cost of the degradation of the natural resources due to the over-exploitation. A United Nations report in 1997 determined each member country's ratio of water consumption to water-availability, in order to gauge the overall pressure on its water resources. The report predicts that moderate to high water-stress translates to consumption levels that exceed 20% of available supply. In Pakistan, this ratio has already exceeded 65 percent. Water-use in agriculture is projected to increase as our food-demand rises.

c) World-wide, agriculture accounts for about 70 percent of water-consumption. However, in Pakistan, its share is already over 90%. Better water management and earliest building of additional water storages is the key to mitigate water-scarcities. More efficient use of water could help to meet the shortages.

d) The major challenges being faced in Pakistan, with respect to water availability include:

- Low financial capacity to build additional water storages,
- Existing storage capacity reduction due to continuous sedimentation;

- Water Resources deficit in time & space in rural as well as urban areas;
- Mining of aquifers, by lowering of water tables;
- Below Kotri escapes to sea;
- Climate change effects;
- Over irrigation and wasteful agricultural practices;
- Water logging and salinity to affect soil fertility and productivity;
- Low water use efficiency in all sectors (Agriculture, Domestic, Industry and Environment) with inadequate awareness to the importance of fresh water.

Water policy that addresses these concerns is the need of the time and consequently, may help to save the situation. Under all scenarios, including the most optimistic, there is a dearth of fresh water. Climate change, as a result of global warming will further constrain our resources.

## 11. FUTURE STRATEGIC AREAS FOR WATER MANAGEMENT TO ENSURE SUSTAINABILITY

The following core areas require immediate attention while formulating contingency action plan and management/policy plans:

### 11.1 Water Demand Management

Water availability is diminishing with a growing population and increasing urbanization. The need for better water demand management is well established. The following represent some areas of immediate attention.

- Promoting efficient use of water by all sectors of life;
- Pricing water to an appropriate level; and
- Better optimizing cropping pattern.

### 11.2 Climate Change Impact on ·Water

Pakistan has been cited amongst the most vulnerable countries facing extreme weather changes in temperature and rainfall. To mitigate risks & impacts of climatic change, the following actions are required:

- Need for additional dams by adopting cascade approach (Basha, Kalabagh and Akhori)
- Confidence building among provinces needed for national harmony.

## 12. CONCLUSIONS AND RECOMMENDATIONS

### 12.1 Conclusions:

- It is generally thought that Climate change is a very slow process and the Earth has always been a temperate world. The Earth has been much hotter and much colder than it is now; actually the past eight thousand years have been a period of unusually stable climate. The IPCC has projected that average global surface temperature will increase by approximately 4°C during the 21st century. Presently, annual average temperatures for Northern and Southern Pakistan are about 19°C and 24°C respectively.

- The glaciers constitute about 2,700 $km^3$ of stored volume of ice, equivalent to about 14 years of average IRS inflows.
- The climate change related concerns have projected increase in variability of monsoon rains that may enhance the frequency and severity of extreme events of floods and droughts. The flood of 2010 occurred due to more precipitation of about 590 mm in northern areas of Pakistan.
- Based on the studies available, it looks likely that the Hindu Kush – Karakoram-Himalyan (HKH) glaciers are receding under the influence of global warming and glacier & snow melting will increase with increase in summer temperature in the northern glaciating areas.

## 12.2 Recommendations:

- It is highly emphasized and recommended that the construction of additional storage dams for generation of low cost hydropower and to meet the ever increasing requirements of irrigation, domestic, industrial and environmental sectors.
- Construction of additional storages would also help to mitigate flood damages besides increase in employment opportunities and economic boost. Moreover, these reservoirs would store billions $m^3$ of water, that is going waste to sea during floods.
- Confidence building measures among the provinces is real need of the time to address the concerns.

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**Paper No. 779**

# SUSTAINABLE DEVELOPMENT AND MANAGEMENT OF GROUNDWATER- LAHORE CITY

**Ghulam Zakir Hassan & Faiz Raza Hassan**

# SUSTAINABLE DEVELOPMENT AND MANAGEMENT OF GROUNDWATER- LAHORE CITY

By

**Ghulam Zakir Hassan[1] and Faiz Raza Hassan[2]**

## Abstract:

Groundwater consists of 96% of planet's unfrozen freshwater and globally 60% of it is used for agriculture, 20% for domestic and 20% for industry. Pakistan is 4th largest user of groundwater after India, USA and China where more than 90% drinking, almost 100% industrial and more than 40% irrigation water requirements are met from groundwater. Lahore is the 2nd largest city of the Pakistan and huge pumpage of groundwater is being carried out to meet the domestic and industrial demands of water supply. Lahore is the second largest populated city of Pakistan with estimated population of 10 million, with an area of 1014 km2. It is located on the alluvial plain of Indus Basin on an altitude ranging between 682 ft to 698 ft above mean sea level and is bounded by Ravi River in the North West and BRBD Canal on the east. Average annual rainfall recorded is about to 675 mm. Groundwater is the only source of domestic and industrial use in the city. The un-planned excessive pumpage of groundwater as 1645 cusec has threatened aquifer depletion along with other socio-economic issues. After creation of Pakistan, the groundwater level in Lahore city was as 15-16 feet which has now reached its depth 150 feet. With the increase in population and industry, extraction of groundwater is tremendously increasing and consequently groundwater reservoir is rapidly depleting. Ravi River, the major source of recharge, remains almost dry throughout the year except few monsoon months. This situation has seriously created a fear that in near future Lahore city can face water crisis and its citizen will suffer badly. In this paper findings of different research studies have been outlined to identify the critical areas and to suggest some management options for the replenishment of this fast depleting natural resource in Lahore. It has been found that in Lahore City annual average depletion rate of water level is 2.5 ft. Potential sites for artificial recharge of aquifer have been identified and possibility of aquifer recharging through rain water has been explored. Results have revealed that quality of groundwater is deteriorating, moving downstream from Ravi Syphon to Lahore and then again improves towards Mohlanwal. It has been further found that pollution in River Ravi is contributing in deterioration groundwater quality.

## 1. BACKGROUND

Lahore, the 2nd largest city of Pakistan is bounded by Ravi River in the North West and BRBD Canal on the East and is the hub of industrial activities being the provincial capital. According to the 1981 census, Lahore had a population of 2.7 million which increased to 6.4 million in1998 **(Faiza & Tabsum 2009).** Now the estimated population of Lahore is more than 10 million. Lahore's population is growing at a rate of 4% a year. The area under Water and Sanitation Agency (WASA) Lahore jurisdiction has a population of about 6.152 million, out of which 5.475 million (89%) are being served.

Issues of groundwater in the Lahore are multifarious and complex in nature. The groundwater level of Lahore after the creation of Pakistan was close to surface at the depth of 15-16 feet but as the city started expanding, it reached upto 70 feet in the period of thirty years (1959-1989)

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which show the decline of more than 50 ft in water-table in Lahore City **(Nazir & Akram1990, Alam 1996).** The un-planned pumpage of groundwater results salt-water intrusion into fresh groundwater due to which sweet groundwater resource is becoming scare in the aquifer underlying the Lahore city. The only source to recharge Lahore aquifer is Ravi River which remains nearly dry except during monsoon season. Currently, groundwater level has declined to more than 100 feet at many places. Natural recharging of groundwater aquifer is almost negligible due to construction activities and pavements of streets and roads. Untreated waste water from municipal and industrial units is being discharged into the Ravi River **(EPD 2008, Ejaz et al 2011).** Pollution in the Ravi River is contributing directly towards the deterioration of groundwater quality in the aquifer underlying the Lahore city **(Hassan et al 2013).**

It has been estimated that about 60-70% population of Pakistan depends directly or indirectly on groundwater for its livelihood **(Lashari et al. 2007).** Groundwater is contributing about 50% of total water being used for irrigation purposes in the Province, while almost 100% demands for drinking water are being met from groundwater.

Over exploitation of groundwater causes negative impacts on our infrastructural network in addition to other socio environmental concerns. Land subsidence is the lowering of the land-surface elevation from changes that take place underground. Common causes of land subsidence from human activity are pumping water, oil, and gas from underground reservoirs; dissolution of limestone aquifers (sinkholes); collapse of underground mines; drainage of organic soils; and initial wetting of dry soils (hydrocompaction). In areas where climate change results in less precipitation and reduced surface-water supplies, communities will pump more ground water. Also, increased demands on ground-water supplies cause more land subsidence in areas already subsiding and new subsidence in areas where subsidence has not yet occurred. In the future, however, increasing population may result in subsidence problems in metropolitan areas where damage from subsidence will be great. Land subsidence causes many problems including: (1) changes in elevation and slope of streams, canals, and drains; (2) damage to bridges, roads, railroads, storm drains, sanitary sewers, canals, and levees; (3) damage to private and public buildings; and (4) failure of well casings from forces generated by compaction of fine-grained materials in aquifer systems.

In developing countries including Pakistan where environmental legislation either does not exist or is not implemented to the desired standards, the groundwater situation is alarming. The main objective of this study is to investigate the potential environmental threats to groundwater and suggest some mitigation measures for groundwater management in the Lahore city area.

## 2 DESCRIPTION OF STUDY AREA

### 2.1 Geographical Location

The study area consists of the Lahore City and its surroundings as shown in Fig. 1. Lahore is located on the alluvial plain of Indus Basin and lies on the left bank of Ravi River. It is situated on an altitude ranging between 682 ft to 698 ft above mean sea level and is bounded on the east by India (international border) and on the south by Kasur district. It covers a total area of 1014 km² and lies between 31°15′ to 31°45′ N and 74°01′ to 74°39′ E. and considered to be the 24th largest city in the world.

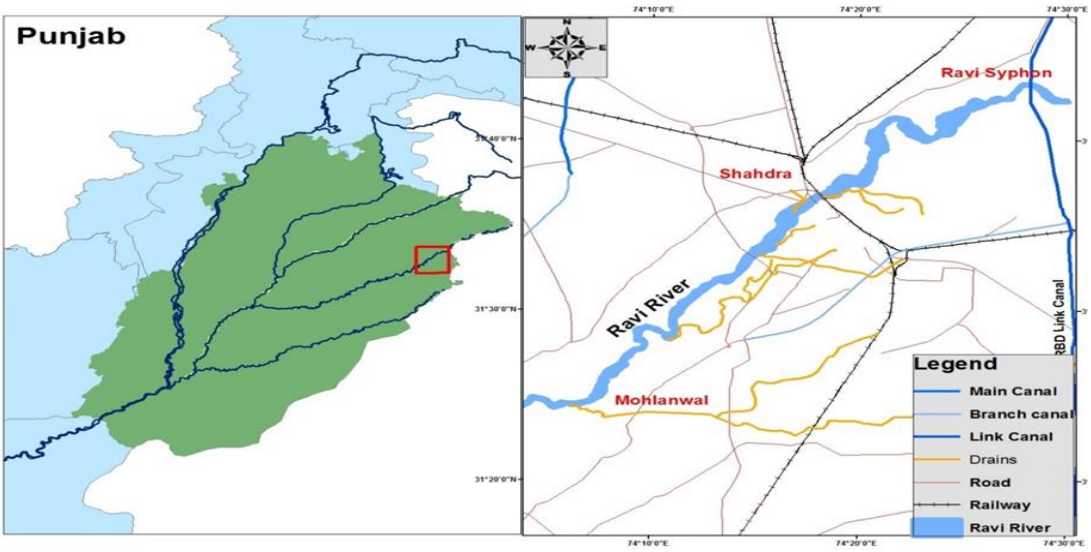



***Fig. 1:*** Map of Lahore City Area

## 2.2 Climatic Conditions

Climatic data have been collected from Pakistan Meteorological Department and analyzed which revealed that Lahore has a hot semi-arid steppe climate and is characterized by large seasonal variations in temperature and rainfall. Mean annual temperature is approximately 24°C ranging from 36°C in June to 12°C in January. The highest maximum temperature 48 C (118°F) was recorded on June 9, 2007 while the lowest temperature recorded in Lahore was –1°C recorded on 13 January 1967.The average annual rainfall recorded is close to 675mm, which can vary from 300 to 1200mm shown Figure 2. The highest rainfall 221 mm was recorded on 13 August 2008. Approximately seventy five percent of the annual total rainfall occurs in monsoon season in the months from June to September and contributes approximately 40mm to groundwater recharge in a normal year. The annual potential evapotranspiration rate is 1750mm which greatly exceeds the rainfall, making irrigation for agriculture essential to supplement rainfall **(Gabriel and Khan, 2010).**

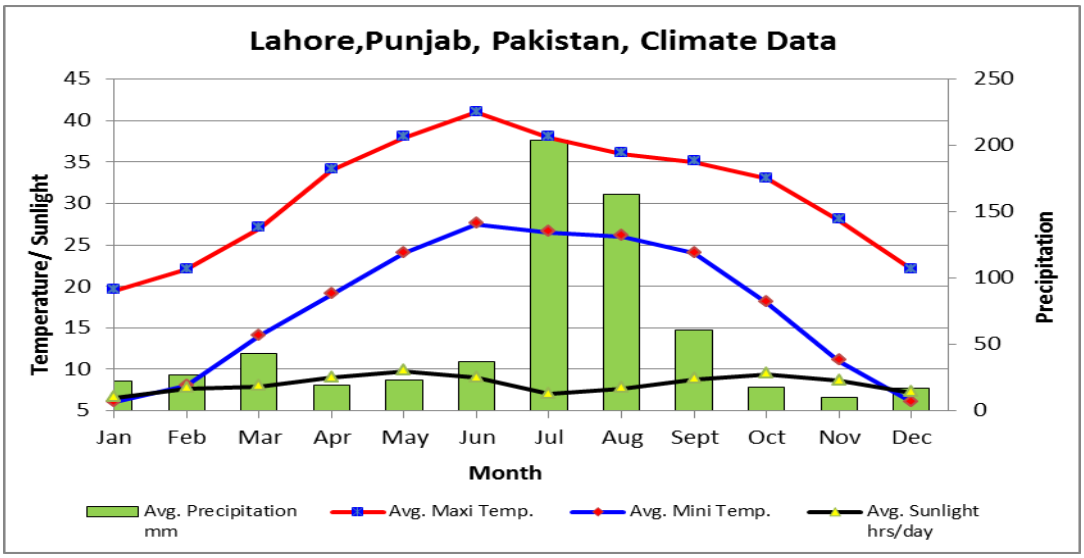



***Fig. 2:*** Climate Data of the Lahore City from (1995-2013)

## 2.3 Geological Conditions

Lahore area is underlain by a significant thickness of alluvial deposits, upto 300 m in depth as investigated by WASID during the period 1961-62 **(WAPDA, 1980**).The sedimentary complex has a thickness of more than 300 meters and is composed of unconsolidated alluvial sediments, consisting of sand, silt and clay in different proportions **(Ahmad et al 2002).**

## 3. ACTIVITIES PERFORMED AND DATA ANALYSIS

Different experiment/Observation has been carried out in the study area to collect the required data to assess the existing scenario of groundwater. Similarly, previous case studies, technical & research papers/reports have also been reviewed. Such activities have been briefly discussed as under:-

## 3.1 Installation of Piezometers

For successful management of groundwater resources it is imperative to have a monitoring network of piezometers. For this purpose nineteen piezometers have been installed at different location scattered in the study area as depicted in Fig. 3.

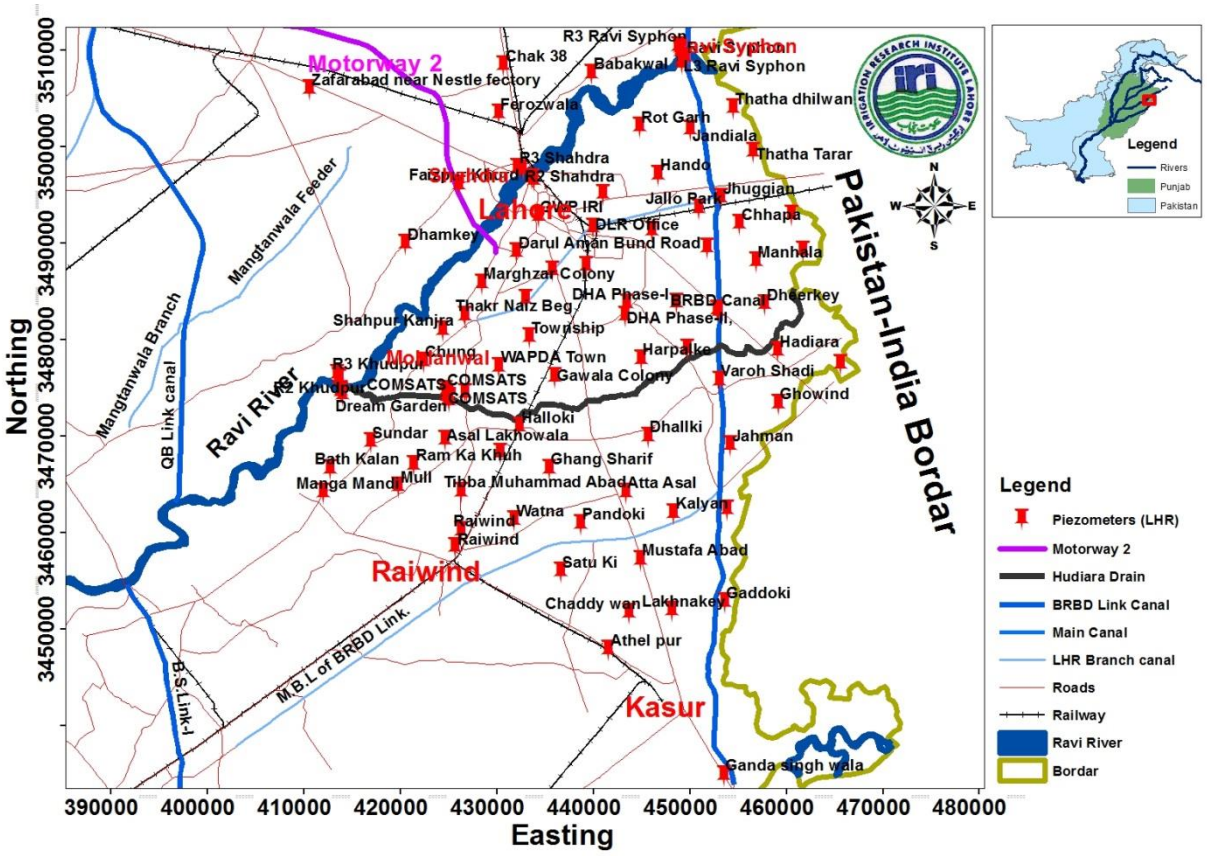



***Fig. 3:*** Location of Observation Wells in the Lahore City

## 3.2 Aquifer Behavior (Water Levels)

Groundwater levels have been measured by using water level indicator from the piezometers installed in the city area and along the river biannually (premonsoon and postmonsoon) soon 2009. The observe data have been analyzed to visualize the aquifer behavior in the study area.

It has been observed that as population of the Lahore city is expanding, groundwater abstraction is increasing for which more numbers of water supply tubewells are being installed

because 100% population of Lahore city depend upon groundwater for drinking purpose. This requirement is met by installing tubewells and water supply system.

Groundwater levels from 50 piezometers installed along Ravi River at three locations i.e. Ravi Syphon, Shahdra and Mohlanwal were observed. Water levels either from left or right side of river at Ravi Syphon, Shahdra and Mohlanwal are falling at most of the locations with the passage of time. The depth to water table observed in piezometers is more at Shahdra as compared to those at Ravi Syphon and Mohlanwal sites which indicate the excessive pumpage from Lahore aquifer.

Due to high rate of pumpage than that of recharge, depth to watertable is continuously increasing which results expanding of depression zone area. This indicates demographic pressure on Lahore aquifer. It has been observed that the depression zone area with water level below 38 m is also expanding continuously with the passage of time at an average rate of 24 $km^2$ per year. Depression zone area was 52 $km^2$ in 2007 and after the period of 5 years (2007-2012), it has reached 150 $km^2$ **(Mehmood et al, 2013)**.

Piezometers were also installed along Ravi River at three locations i.e. Ravi Syphon, Shahdra. Water levels either from left or right side of river at Ravi Syphon, Shahdra and Mohlanwal are falling at most of the locations with the passage of time. The depth to water table observed in piezometers is more at Shahdra as compared to those at Ravi Syphon and Mohlanwal sites **(Hassan et al 2013)** which indicate the excessive pumpage in the city.

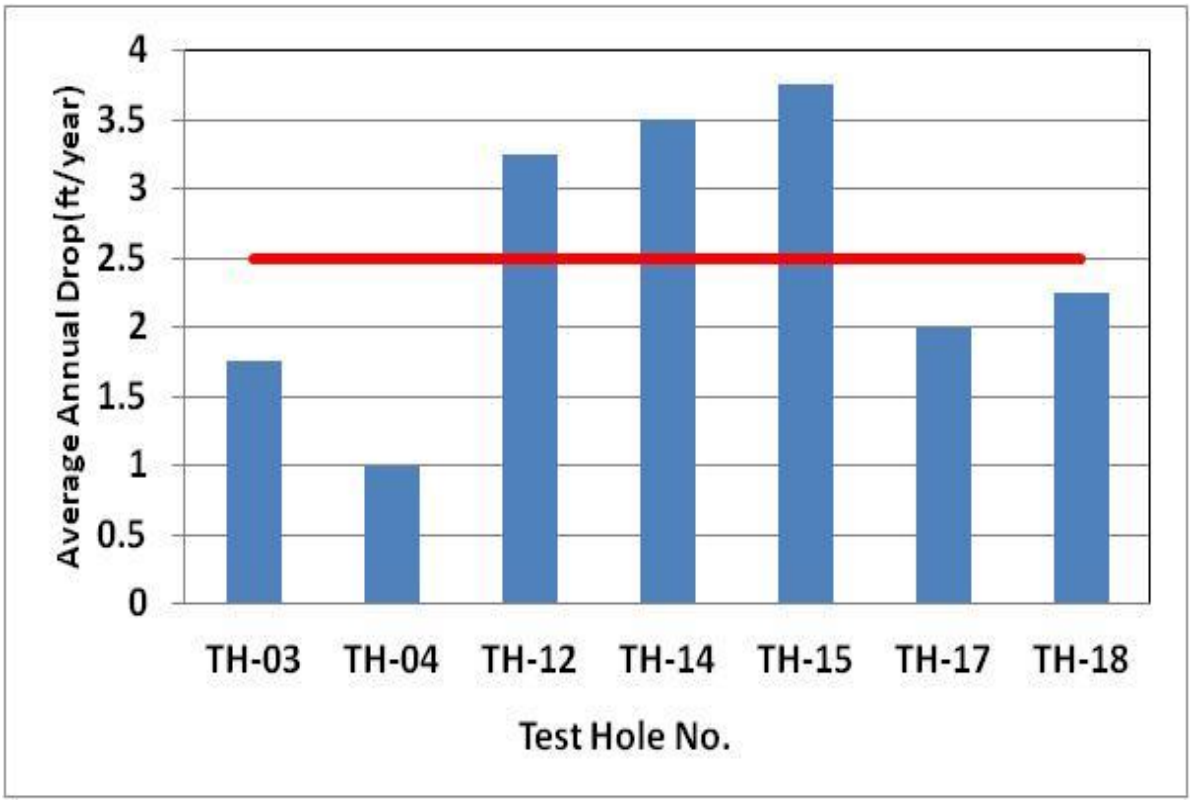



***Fig. 4:*** Average Depletion rates of groundwater in Lahore (2009-2015)

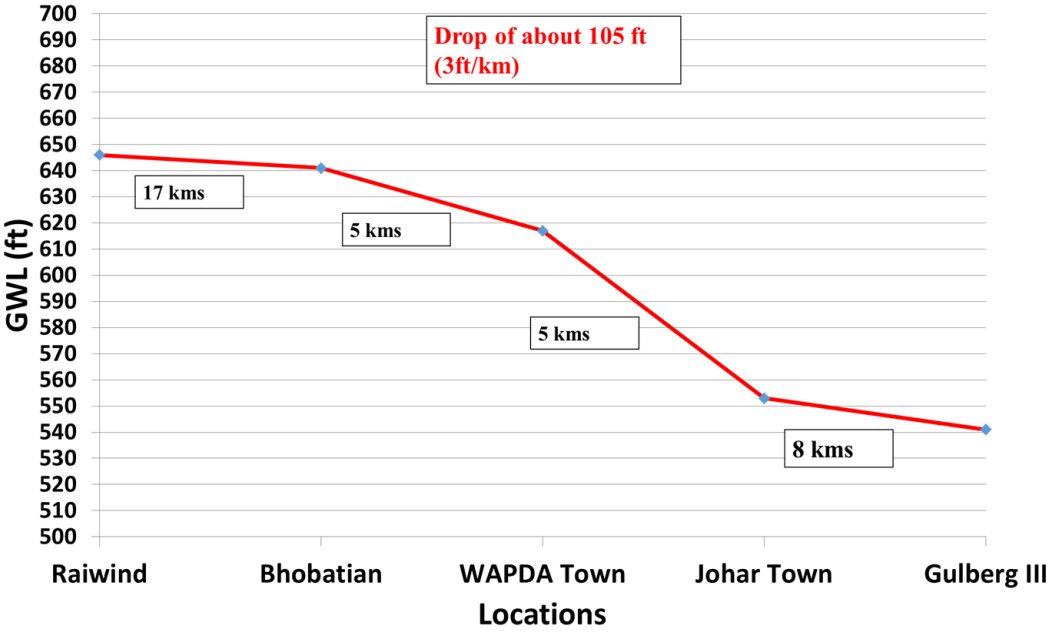



***Fig. 5:*** Longitudinal profile of Groundwater Levels from Raiwind to Dharampura, Lahore (2016)

## 3.3 Monitoring of Groundwater Quality

Groundwater quality has been observed from piezometers installed in the study area by a specially designed groundwater sampler from the piezometers installed in the city area and along the river biannually (premonsoon and postmonsoon) soon 2009.Groundwater samples from various sites have been collected in sterilized sample bottles and have been analyzed. pH and TDS parameters have been measured on site by using calibrated pH meter model 330I and Conductivity/TDS meter model Eutech Cyberscan CON 11.

Groundwater water samples from piezometers installed at different sites of Lahore area at different depths were collected and tested. Groundwater quality at downstream to Lahore city has deteriorated while the groundwater quality at Ravi Syphon upstream of Lahore is sweet because there is no entry of pollution at Ravi Syphon. It was observed that the color of groundwater of Lahore city aquifer has been varied from colorless to yellowish, odour varied from odourless to objectionable and turbidity varied from 2 to 4 NTU. Heavy metals have also founded in the groundwater and the concentration of lead (Pb), Nickel (Ni) and number of E. coli levels are exceeded the permissible limits of drinking water quality **(Ayesha 2010).** Municipal landfills are considered an other sources which have a serious threat to urban environments and a great source of pollution especially ground water **(Akhtar and Zhonghua 2014**).

The fluctuations in groundwater levels and quality to develop the link between River flows and groundwater in the underlying aquifer was done by water samples from the piezometers installed along the river, away from the river and at different depth. Groundwater quality has been analyzed in different directions like along the river, away from the river, vertically downward and with the passage of time to derive some conclusions. In addition to surface water (river and drains), groundwater samples from all 50 piezometers installed at different locations and depths along the river have been taken through a specially designed sampler to correlate

the quality of water in river with quality of groundwater. The impact of pollution in river water on the groundwater has been observed in four dimensions as mentioned already.

Groundwater water samples from piezometers installed at three sites of Ravi Syphon, Shahdra and Mohlanwal at different depths were collected and tested. Electrical Conductivity (EC) of groundwater at all sites has been graphically plotted as shown figures 7-9.

The results analysis of groundwater from piezometers installed at Ravi Syphon site indicate that groundwater quality downstream Ravi Syphon on both sides of the River at all depths (50 ft, 100 ft and 150 ft) is good and is not deteriorated. This indicates that groundwater quality perpendicular to the river from Left side or right side ($L_1$, $L_2$, L3 or $R_1$, $R_2$, $R_3$) is good and can be used as bench mark for comparison of groundwater quality while moving downward. The data analysis at Shahdra site reveals that EC values at 50 ft depth are more while the value at 150 ft depth is lesser on both sides of the River. Groundwater quality at 50 ft depth at $R_3P_3$ and $L_3P_3$ is deteriorating. At Mohlanwal site, EC values of piezometer installed at 50 ft depth are more as compared to those at 100 ft and 150 ft depth on left side while lesser on right side of the river.

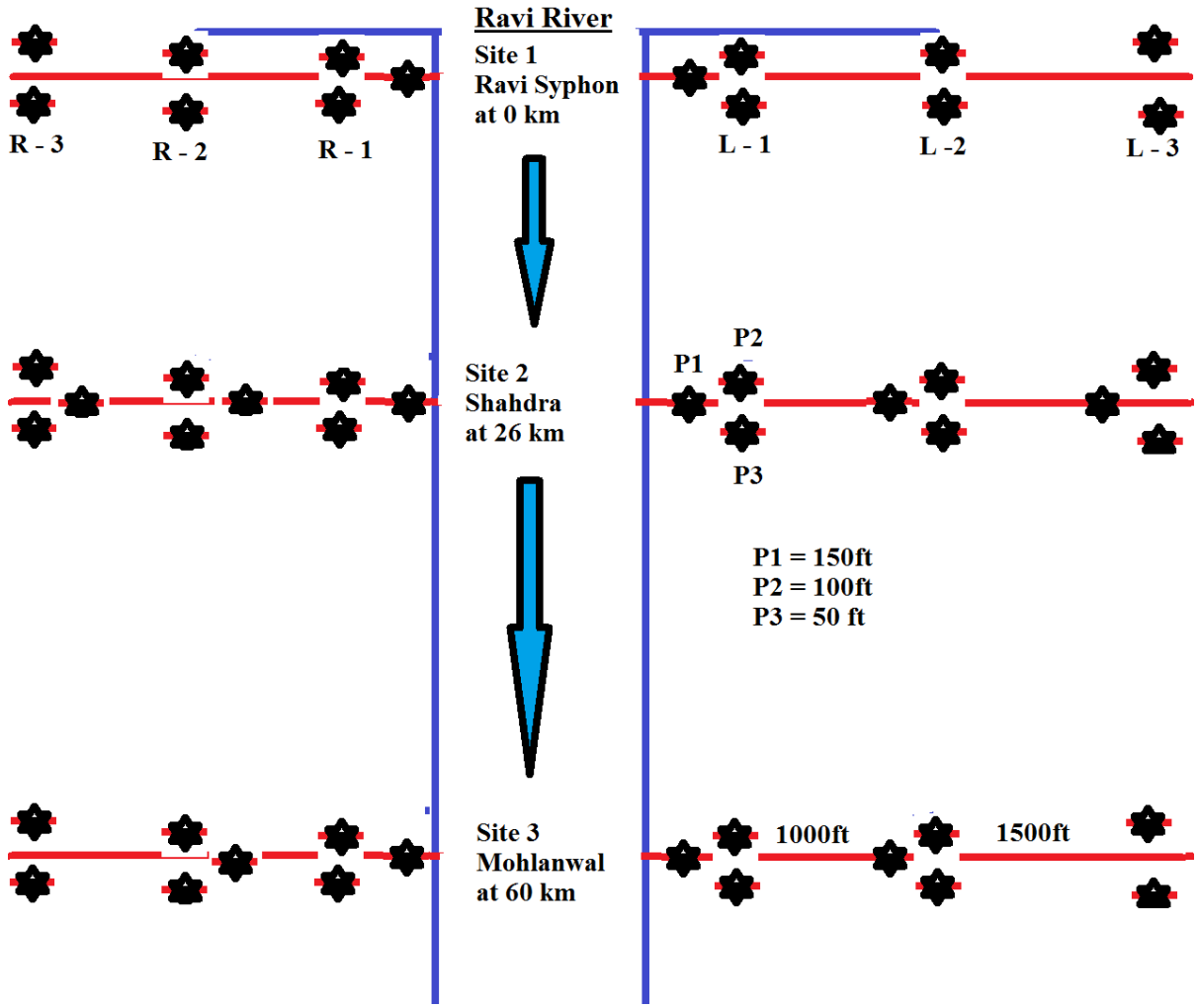



***Fig. 6:*** Schematic Diagram of Ravi River Showing Locations of Piezometers

It is clear from data that groundwater quality at $L_3P_3$, $R_1P_2$ and $R_3P_3$ (Shahdara sites) is deteriorating. During 2011 and 2012 the value of EC at $R_3P_3$, $L_3P_3$ (50 ft depth) has increased

rapidly. It indicates that quality of groundwater at 50 ft depth is deteriorating with the passage of time. Overall results of analysis of groundwater samples along Ravi River indicate that the quality of water is deteriorating, moving downstream from Ravi Syphon to Lahore.

EC values of groundwater on left side of the river at location (L3) of Shahdra site at various depth 50ft, 100ft and 150ft for the year 2012 plotted in figure 7 indicates that water quality of shallow groundwater (50ft) is deteriorating more as compared to 100 and 150ft.

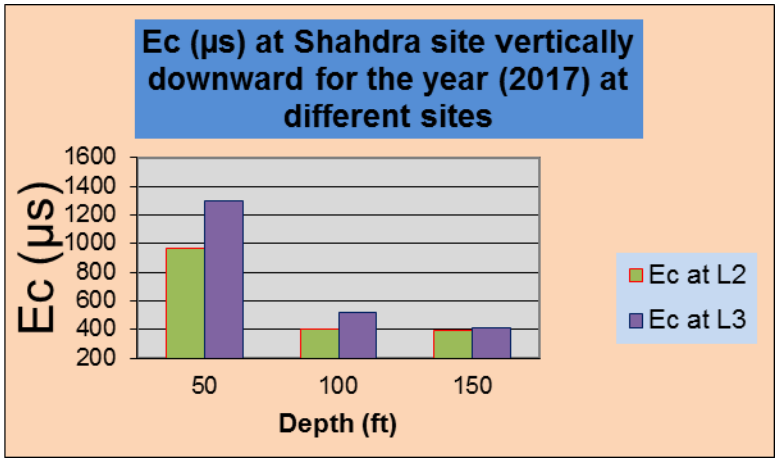



***Fig. 7:***

Water quality of shallow groundwater (50ft depth) along left side of the river at Location (L3) of Shahdra site for the years 2010 to 2017 were compared as indicated in fig. 8 The data indicates that with the passage of time water quality of shallow groundwater becomes poor. It is clear from figure 8. EC values of 50ft depth for the year 2016 are more as compared to that of 2010.

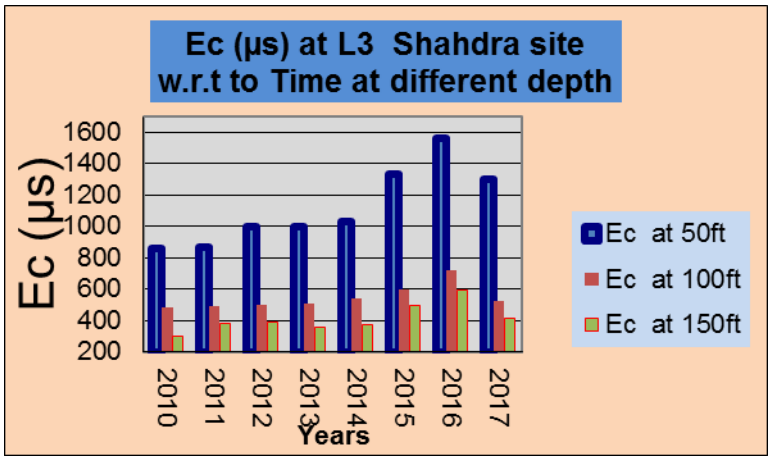



***Fig. 8:***

Shallow water quality data along the river from D/S of Ravi Syphon towards Mohlanwal site were also compared for the year 2010 to 2017. The fig. 9 indicates that as we go down from Ravi Syphon towards Lahore, water quality is deteriorating. The figure depicts that shallow water at Shahdra site is more poor and deteriorated and then from Shahdra to Mohlanwal it is improving due to dilation effect. By comparing three sites Ravi Syphon, Shahdra and Mohlanwal, water quality is becoming deteriorated/ poor at Shahdra site along the river.

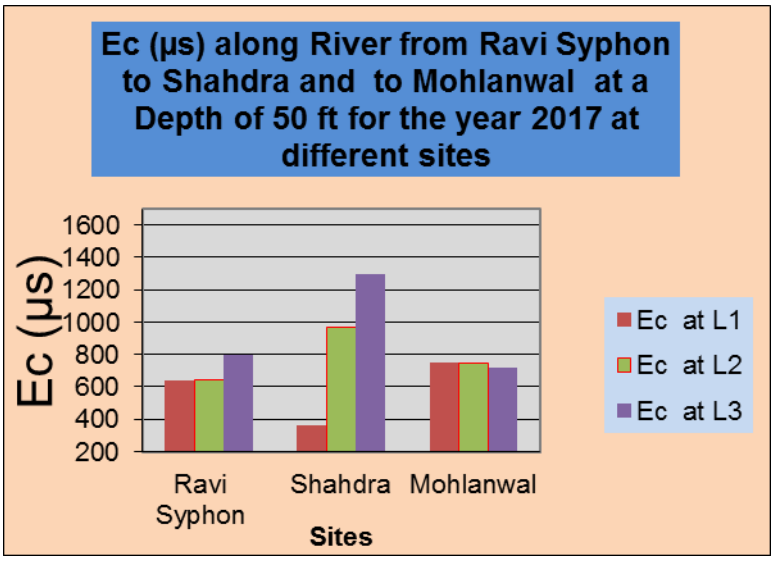



***Fig. 9:***

Overall result of analysis indicates that groundwater quality is deteriorating more at Shahdra (as compared to that of Ravi Syphon and Mohlanwal) due to entrance of effluents through different drains into the river. The data at Shahdra site along both sides of the river indicates that quality of shallow water at depth of 50ft at R3 and L3 is deteriorating more with the passage of time.

### 3.4 Sub Surface Lithology

Soil samples at different depths and locations were collected from all the piezometers installed in Lahore area and analyzed to determine the soil classification of sub-soil strata in the laboratory. The type of strata through which water leaches down plays an important role as sandy formation will support the higher flow rates and vice versa. Analysis of soil samples was carried out to obtain the subsurface lithology and grain size distribution below the soil surface up to a depth of 150 ft. Soil samples at different depths and locations were collected from all the piezometers installed in Lahore area and analyzed to determine the soil classification and permeability of sub-soil strata in the laboratory by constant head method.

## 4. POTENTIAL THREATS TO LAHORE AQUIFER

Being a thickly populated, centre of industrial activities and provincial capital Lahore has become a city of complex issues related to groundwater pollution. A wide range of pollutants generated by natural and human activated are contributing towards the degradation on natural resources of groundwater in the area. A list of such threats has been discussed as under:-

### 4.1 Over Pumpage

Groundwater is the only source of water supply for Lahore city. WASA, Lahore has installed 480 tube wells of different capacities at a depth of ranging from 150 to 200 m for supplying water to the citizens of Lahore. These tube wells are extracting about 1170 cusec of groundwater per day for drinking purpose. In addition to WASA tubewells, a large number of private tubewells installed in housing schemes are roughly pumping 100 cusec water daily. Water is also being pumped by industries at the rate of approximately 375 cusec **(Hussain & Sultan 2013).** In this way total extraction of groundwater in Lahore becomes 1645 cusecs. Over exploitation of groundwater causes many serious environmental concern s like salt water intrusion, increase in pumping cost, increase in installation cost of tubewells, land subsidence, land sliding etc.

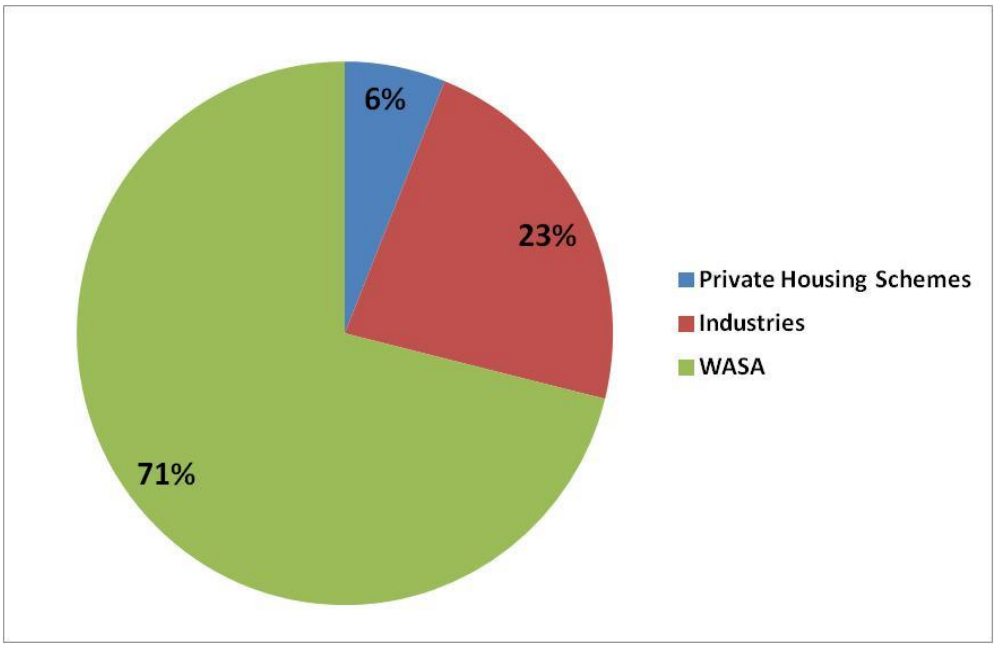



***Fig. 10:*** Major Groundwater Consumers in Lahore

## 4.2 Population Growth

Population growth has a direct impact on groundwater resources. Abstraction of groundwater increases as population grows. Such over exploitation of aquifer result in decline of groundwater level. Temporal trends of depth to watertable and population growth in Lahore as depicted in Fig. 11.

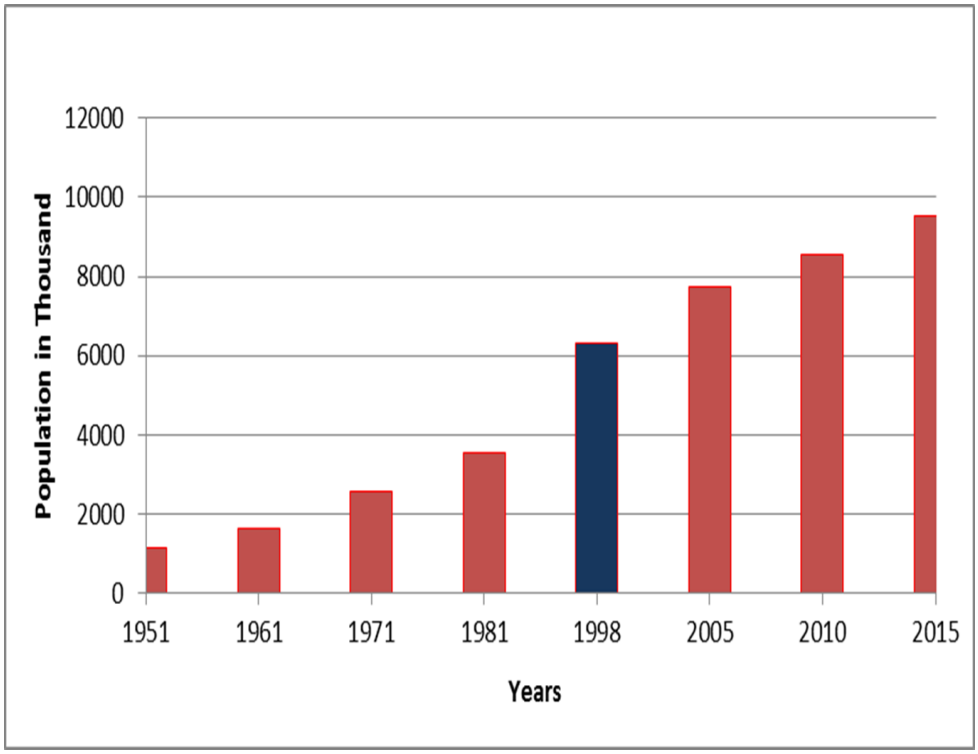



***Fig. 11:*** Impact of Increasing Population on Watertable Depth

## 4.3 Urbanization and Commercialization

Urban sprawl is the fastest growing threat to local environment and quality of life. Lahore city is adversely affected by uncontrolled and unplanned increasing urban industrialization and commercialization. As urban areas expand, environmental problems like losing green space, decreasing groundwater recharge area, degradation of natural ecosystem and deterioration of water resources are increased. The impact on groundwater quality from urban sprawl is attributed by the combined effect of population and land use change.

## 4.4 Low Flow in Ravi River

Ravi River is the smallest of five eastern rivers of the Indus River System (IRS). It enters in Pakistan at Jassar, about 120 km upstream of Lahore and joins the Chenab River near Kabirwala after flowing down about 520 km. The average annual flow of the Ravi River in Pakistan territory was 7 million acre feet (MAF) during the period 1922 to 1961 but due to Indus Water Treaty of 1960 between India and Pakistan, right to use the water of this river were allocated to India. The average annual flow from 1985 to 1995 was recorded as 5-MAF which was further decreased to 1.1 MAF in years 2000-2009 due to construction of hydropower projects/dams on Ravi River by India. It results in lowering in groundwater level in Lahore and it adjoining area. Ravi River seems to be the main source of recharge in the North-West of Lahore. For the last two decades, Ravi River remained almost dry except in monsoon, so the recharging through River has seriously decreased. Under these circumstances on one side recharge to the aquifer has decreased tremendously and on the other side the ecosystem in the river has suffered badly convert the river into a “sludge carrier”.

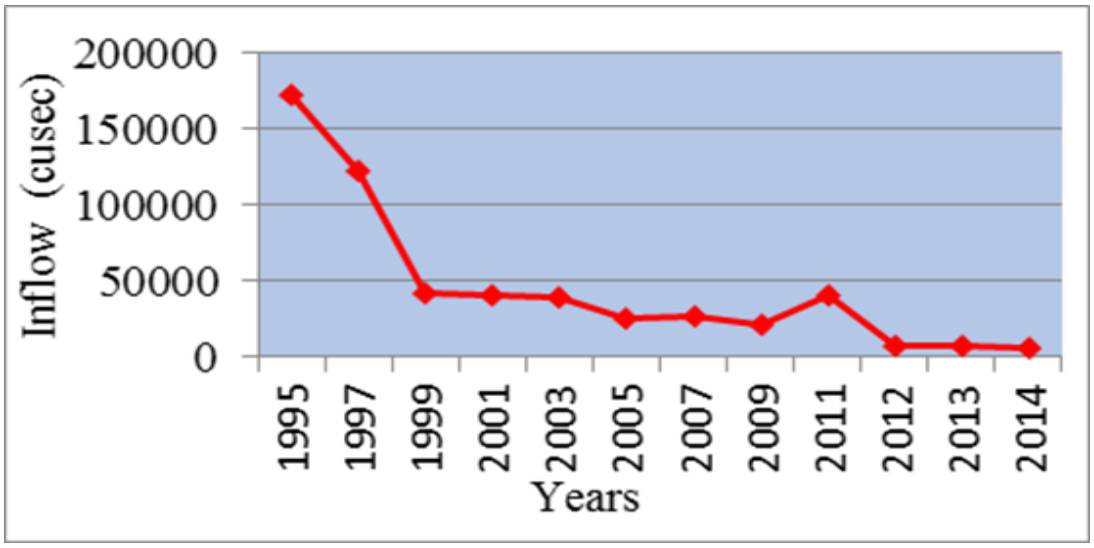



***Fig. 12:*** History of Water flow in Ravi Source

## 4.5 Sewage and Street runoff

Urban population in the Lahore is increasing at an alarming at 4% per year which is leading towards a continuous increase in domestic sewage. This sewage coupled with street runoff is a severe threat to groundwater as a part of it ultimately leaches down to groundwater. It was estimated that discharge of waste water of Lahore city into Ravi River was about 990 cusecs in year 2006 **(Saeed and Bahzad 2006)** and now has crossed to 3,304 cusecs through drains and various pumping stations without proper treatment (**Hussain & Sultan, 2013**) as depicted in Fig. 13.

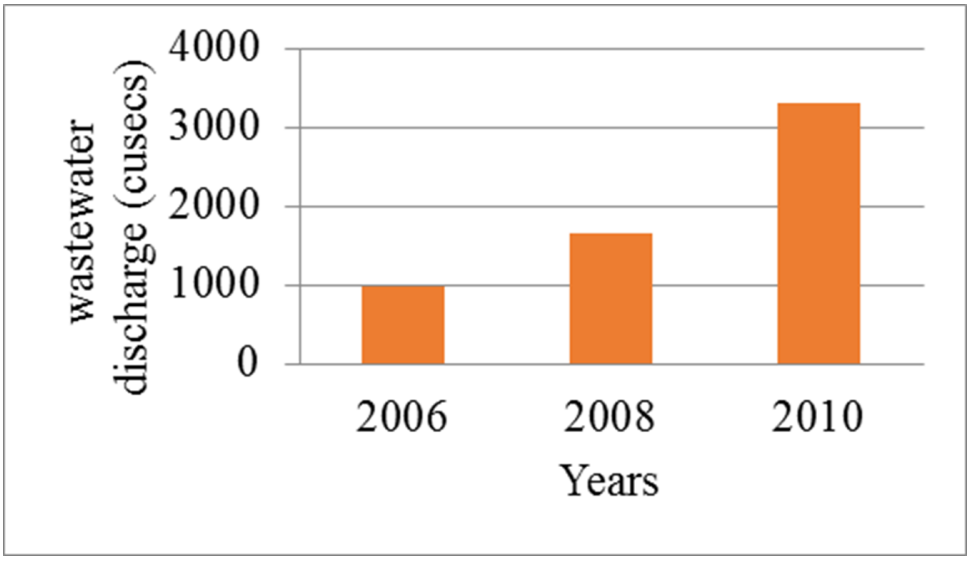



***Fig. 13:*** Effluents being thrown into the Ravi River

## 4.6 Surface Drainage Network

A network of surface drains in Lahore city is existed which carry wastewater from various sources and ultimately enters the Ravi River. These are earthen channels which causes the leaching of various pollutants directly to groundwater.

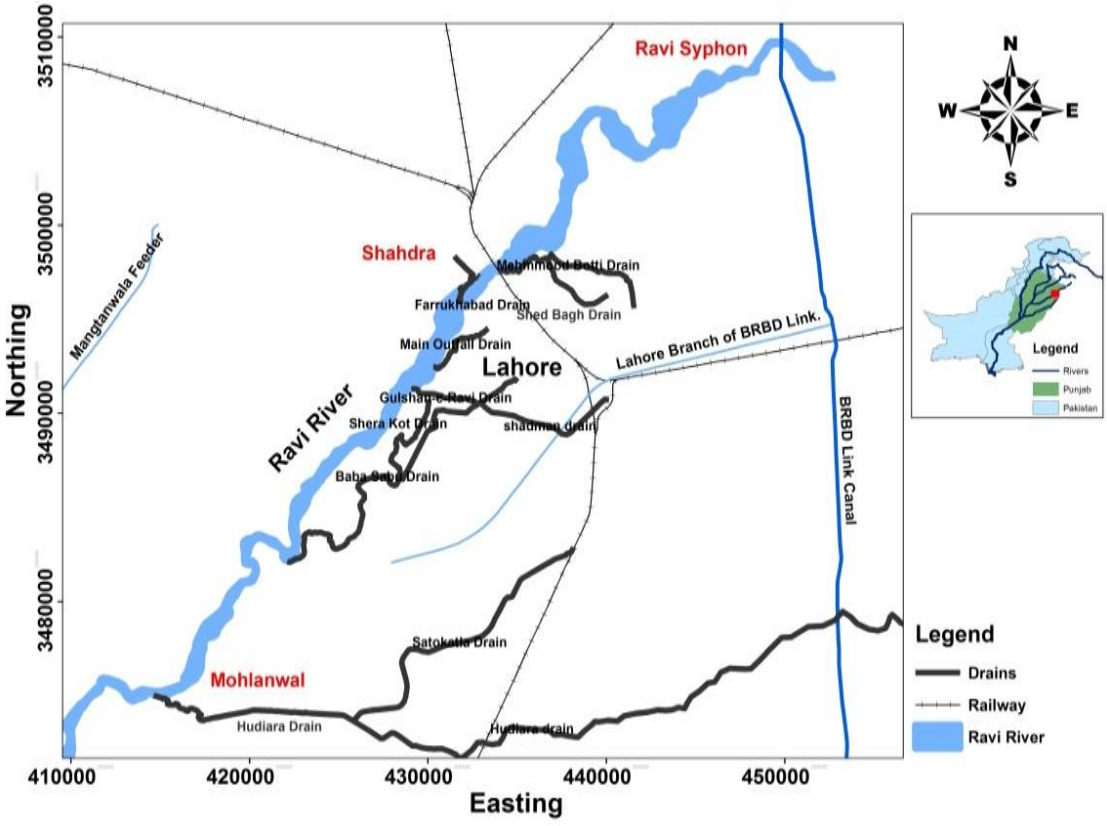



***Fig. 14:*** Location of Major Drains in Lahore area entering into the Ravi River

## 4.7 Industrialization

As mentioned already, Lahore has become hub of industry activities in the country. A large number of industry pertaining to textiles, chemicals, auto parts, electric appliances, machinery, food, restaurants, plastic and pvc retailers that are based in the beautiful Lahore city are polluting the environment. These Industries are located at Kala Shah Kaku, Lahore Sheikhupur road, along Lahore band road, Quaid-e-Azam Industrial estate at Kot Lakhpat, and Multan road in Lahore city and using huge amount of groundwater for processing raw material and finishing the products. Instead of these industrial estates, a large numbers of in house small industrial units are working within the Lahore city which are discharging wastewater into sewerage system without treatment.

Wastewater is being discharging into drains without treatment which finally fall into Ravi River and due to which the River is acting almost as a "sludge carrier" which is damaging the aquatic life, especially the fish **(Shakir & Qazi 2013)** and from the river wastewater reached to the groundwater after percolation. Domestic and industrial effluents contain organic and inorganic pollutants, which deeply percolate through the soil depending upon the soil nature and sooner or later deteriorate the groundwater quality. Flow in Ravi River during the winter is insufficient to wash off wastewater pollution. **(Hassan et al 2013, Ayesha 2010).** The environmental profile of Pakistan indicates that about 40% of deaths are related to waterborne diseases spread by water pollution, mainly due to the sewage and industrial wastewater contamination to drinking water distribution systems.

## 4.8 Dumping of Solid Waste

Typically Municipal Solid Waste (MSW) consists of household waste, commercial waste and institutional waste. Unscientific dumping of solid waste always poses serious environmental problems on groundwater. Leachate produced at landfill contains thousands of complex components and it becomes part of groundwater after infiltration. With reference to Lahore city, three sites were selected which are located at Mehmood Booti, Saggian and Baggrian for dumping of solid waste. Groundwater is suspected to be contaminated due to unscientific, unsafe, unplanned and traditional selection of these sites. It has been estimated that about 5,700 tons of solid waste is generated daily in Lahore city from different sources and this volume is equivalent to a generation rate of 0.84 kg/capita per day which is lower than world's average of 1.2 kg/capita per day **(Batool and Chuadhry 2009).** At least three-quarters of the total waste generated (3800 tons/day) in Lahore is dumped at these sites without proper treatment. According to a previously study, it was found that most of groundwater samples collected nearby these landfill sites contain pollutants and their concentration level in groundwater is higher than prescribed be Pakistan Standards and Quality Control Authority (PSQCA) and concentration of Arsenic in drinking water Is higher than WHO criteria**(Akhtar & Zhonghua 2013).** According to United Nations Environmental Programme (UNEP)'s data about 47% drinking water in Lahore city was contaminated due to presence of various hazardous toxic elements **(Manan, 2008).**

## 4.9 Agricultural Runoff

Excessive and uncontrolled use of chemical fertilizers and pesticides promotes contaminated agricultural run-off. This not only pollutes the surface drains but the water trickling down to lower layers of soil causes a severe contamination of the natural aquifer. Over abstraction of groundwater prompts recharge from the surface water drains, which themselves are severely contaminated. Different drains like Hudiara drain which collect surface runoff from agriculture field from India and Pakistan.

## 4.10 Air pollution

Vehicular and industrial emissions comedown with rainfall in the shape of acid rain which increasing the acidity of surface water body like lakes, rivers and drains due to which aquatic life is affected adversely. These toxic pollutants leach down from soil surface to groundwater. Acid rain dissolves all the useful minerals from the top soil like potassium, calcium, magnesium and leaches them down to the aquifer. Similarly aluminum is also activated by acid rain which causes the death of aquatic life and contaminates the groundwater reservoir. In addition to the threats mentioned above, lack of proper coordination between various stakeholders and awareness among the various groundwater users are also of prime concern and contribute significantly in degradation of the groundwater resources in the city.

# 5. MANAGEMENT OPTIONS

Groundwater is a precious gift of nature and is playing a vital role for the existence of life at the planet. This natural resource is being used for drinking, agricultural, industrial, livestock and other uses and is continuously under threat. Groundwater reservoir is a natural system which is balanced naturally by different sources of recharge including rainfall, dams/lakes, ponds, rivers, canals, water courses and irrigated fields etc. Different inflows and outflows from the system are balance automatically and system remain under equilibrium conditions. Human being is the major player who plays with the nature to meet with its different increasing needs of food and fiber. Human activities interrupt with the natural ecosystem and balanced is disturbed which then creates various multidimensional issues in the real world, which we call environmental threats. For example, when we talk about the Lahore aquifer, we have reduced/hindered all most all the sources of recharge of aquifer and on the same time extraction of groundwater is increasing tremendously. Such scenario leads to environment degradation of groundwater reservoir with respect to quantity and quality.

## 5.1 Artificial Recharge

To maintain the quantity sustainable, the recharge of aquifer is the only and only viable solution for replenishment of the fastly depleting aquifer no doubt we should pumpage as well. Recharge of aquifer can take place naturally, but if it is not possible we should intervene and should devise the artificial ways and means for recharging the aquifer. A variety of methods have been developed and applied to artificially recharge groundwater reservoirs in various parts of the world. Selection of the feasible methods is important and can be challenging. The methods may be generally classified as i) the direct surface recharging techniques, ii) indirect recharge techniques and iii) combination of surface and subsurface methods including subsurface drainage (collector wells), basins with pits, shafts, and wells etc. Possible options for artificial recharge in Lahore area are:

i. Rainfall harvesting at potential sites
ii. Lahore Branch Canal,
iii. BRBD canal
iv. Ravi River by diverting more water in it
v. By constructing a ponds/lake between Ravi River and BRBD canal.

## 5.2 Rainfall Harvesting

Rainfall is a natural source which can be harvested and diverted/injected to groundwater reservoir. It helps a lot in maintain groundwater potential and at the same time can we can avoid the street flooding and overflowing of drains/nalahs and choking of sewerage systems. Rainfall

water, after some time, is comparatively of good quality and can safely be used for recharge purpose by adopting a suitable design mechanism.

Field surveys and sub-surface investigations have been carried out by IRI to identify potential sites for artificial recharge through rainfall harvesting. A list of such sites is given in Table 1 and is shown in Fig. 15.

***Table 1:*** List of potential sites for artificial recharge of aquifer in Lahore by rainfall harvesting

| Sr. No. | Name of site/location |
|---|---|
| 1 | Jallo Park, Lahore. |
| 2 | Along BRBD Canal Right Bank, near Barki Village, Lahore |
| 3 | Padhana Village, Barki Road, Lahore<br>(Western Side of Lake). |
| 4 | Lahore International Airport Lahore<br>Post No. 4, Civil Aviation Authority, Southern end of Runway |
| 5 | Walton Airport, Ferozepur Road, Lahore. |
| 6 | Northern Plot of Badshahi Mosque, Lahore |
| 7 | Field Research Station, Niazbeg, Multan Road, Lahore. |

These sites have been identified after physical and topographical surveys which a reasonable volume of rainfall is collected naturally in depressions and can be recharged. After physical surveys, sub-surface lithology and soil characteristics have been determined to ascertain whether the sub-surface strata support recharge or otherwise. It has been found that all these locations are hydrological, geo-hydraulically and geo-chemically feasible for recharging the aquifer.

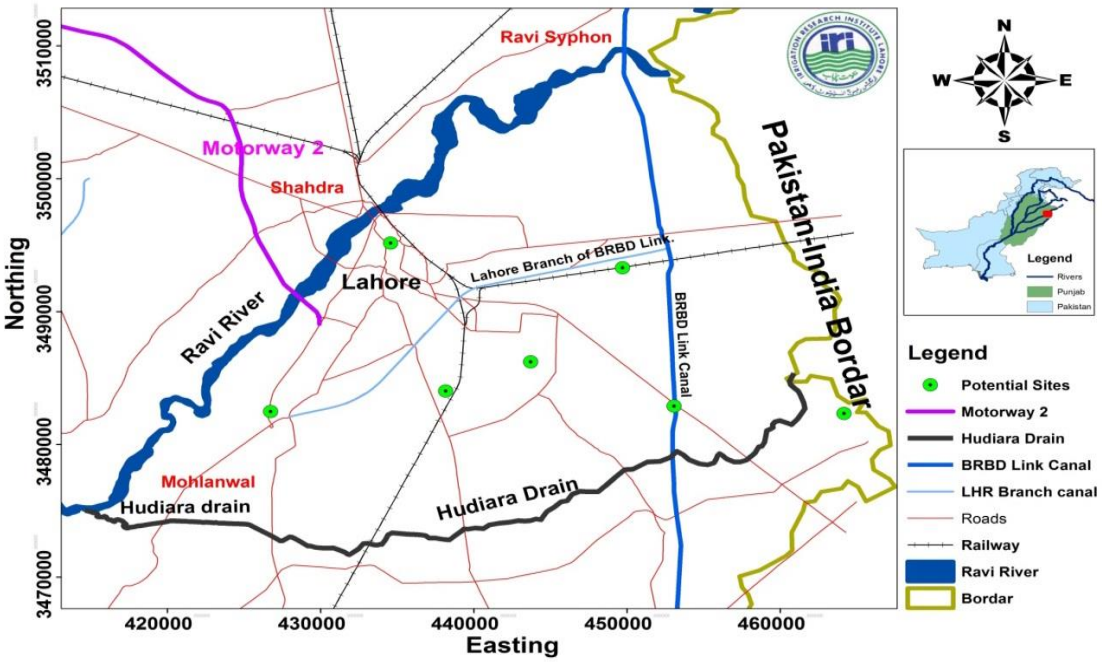



***Fig. 15:*** Potential Sites for Artificial Recharging



Outlet of Badshahi Mosque Lahore



Water Storage in Padhana Lake



Over view of Padhana Lake



Installation of Exploratory Boreholes in the Field






Rain-water flooding at Walton Air Port Lahore

***Fig. 16:*** Potential Sites for Artificial Recharging

## 5.3 REDRESSING THE SURFACE POLLUTANTS

The 2nd largest issue of groundwater is its continuously deteriorating quality which is of more concern as it deals directly with human health. Sources of groundwater pollution normally man made intervention on earth surface and in certain cases the salts in bed rocks. Most of the pollutants effluents like industrial, agricultural, municipal etc in liquid forms which leach down to groundwater. Some other are in solid form like solid waste heaps through which pollutant leach down to subsurface soil and then to groundwater. Some pollutants are in gaseous form like vehicular and industrial emissions, which return back to soil surface via acidic rains and percolate down to groundwater through unsaturated zone. Broadly speaking these are the surface pollutants.

As a first effort, their production at source should be reduced or minimized through scientific research and public awareness campaigns. For example, farmers must be educated to use less toxic and in limited/required quantity fertilizers, pesticides, weedicides etc. Similarly, industrialists should be provided with scientific solution to recycle or treat the effluents at source instead of throwing it into nearby water body or injecting directly to groundwater which off-course is not less than a crime. In the same way, sever steps are required to be taken for control/mitigation of gaseous emissions. Regarding solid waste management we can go for use of geo-synthetic materials at landfill sites to avoid leaching of pollutants. Solid waste can also be used to obtain energy. Regarding municipal effluents we should take care to avoid mixing of rainfall with such pollutants and should capture and harvest the rainfall separately and store it in surface or sub0surface storages. Treatment plants should be adopted to make the industrial, municipal, and agricultural waste waters useable some purposes like industries, irrigation etc keeping in view the cost and need options into considerations. Control on over-pumping can be helpful to avoid mixing of saline-fresh water in the aquifer

## 6. CONCLUSIONS

1. Potential identified threats to groundwater reservoir in Lahore are uncontrolled and un-planned over-pumping, untreated effluents, lack of recharge of aquifer, lack of coordination between various stakeholders
2. Groundwater levels in the city area are falling at an average rate of 2.5 ft per year and even more than 3 ft per year at certain locations due to excessive pumpage and less recharge.
3. Groundwater in some locations in the city area has fallen more than 100 ft below the natural surface level.
4. Ravi River has become a source of pollution for groundwater reservoir underlying the Lahore city (due to low flows and throwing of effluents in it)
5. Groundwater quality in the aquifer in deteriorating with the passage of time and sweet water in become rare and out of reach and improves with the depth below natural surface.
6. Groundwater quality deteriorates moving downward from Ravi Syphon to Mohlanwal and is the worst near Shahdra along the Ravi River.
7. Ravi River can contribute towards recharging the aquifer as groundwater levels fluctuate with the river gauge.
8. Sub-soil strata at most of the sites are generally sandy except thin lens of clay/silt in the upper layer of 50 ft. However, near Ravi Syphon thick silty/clay layers have and found below the depth of about 100 ft.

9. Quality of effluents in drains entering the River is deteriorating with the passage of time.
10. Slope of hydraulic gradient line has been observed from Raiwind/Kasur to lower and steeper slope has been observed as the line approaches Lahore.
11. There is lack of communication/coordination among different Government Departments/ agencies and the various stake holders/ consumers

## 7. RECOMMENDATIONS

### 7.1 General

1. A working group consisting of inter-departmental experts and the members of civil society/NGOs to sit together to bridge the gap of coordination/communications.
2. To strengthen the monitoring network for groundwater levels and quality (vertical and temporal)
3. Detailed groundwater investigations through modeling approach (future prediction of flow and solute transport under various stresses) to study the factors which are responsible for the worse groundwater quality and declines in levels.
4. Installation of treatment/recycling plants for sewerage and industrial effluents as well as for solid waste in the city at appropriate locations.
5. Pumping tubewells should be installed after assessment of aquifer potential and should be scattered uniformly and possibly near a recharge source.
6. Water saving campaign through public awareness and giving subsidy on water saving instead of water supply.
7. Artificial recharge / Rainfall harvesting through rainfall-runoff modeling at identified potential sites and to explore more feasible sites in parks, playgrounds and other natural depressions.
8. Regular monitoring and lining of surface drains in city area.
9. To maintain minimum flow in Ravi River for dilution of pollutants and enable it to act as source of recharge.
10. Construction of lake/ponds in the east-west of Lahore to recharge the aquifer after detailed investigations.
11. Explore possibility of Lahore Brach Canal and BRBD canal as recharge sources.
12. Concept of earthen green belts/dividers along the roads should be adopted from where rainfall water can leach down naturally or some injection wells can be installed as deemed appropriate. Trees can also be grown in some ditches/dividers.

## 8. ACKNOWLEDGEMENTS

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**Paper No. 780**

# COMPARISON OF TEMPORAL CHANGES IN GROUNDWATER DEPTH WITH RESPECT TO DROUGHT AND NORMAL YEARS IN IRRIGATED AREAS OF SINDH AND BALOCHISTAN

**Engr. Ijaz Javed, Engr. Dr. Muhammad Saeed, Engr. Muhammad Shoaib, Engr. Muhammad Jawad & Dilbar Hassan**

# COMPARISON OF TEMPORAL CHANGES IN GROUNDWATER DEPTH WITH RESPECT TO DROUGHT AND NORMAL YEARS IN IRRIGATED AREAS OF SINDH AND BALOCHISTAN

By:

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## Abstract

In the past, it has been observed that with the time there are fluctuations in watertable depth specially, in comparison of drought year (2000) to normal years afterwards (2010-14). This study relates to the comparison of temporal changes in groundwater depth with respect to drought and normal years in irrigated areas of Sindh and Balochistan.

In this study, the database of depth to watertable, was used as secondary data to assess spatial and temporal changes in groundwater depth conditions of pre and post-monsoon for the period (2000, 2010-2014). Depth to watertable contour maps have also been used for identification of waterlogged (disaster) areas. Area under different depth to watertable classes, percentage area of Sindh and Balochistan derived from the contour maps that were processed through GIS software. A spreadsheet has been developed for conducting analysis of depth to watertable data derived via contour maps presented in the form of bar charts.

The result shows that during pre-monsoons the waterlogged areas i.e. (<1.50 m DTW) which was 11.13% in 2000 increased to 53.72% in 2011 after which it gradually decreased in the preceding years i.e., 2012, 2013 and 2014. Whereas during post-monsoon the waterlogged areas i.e. (<1.50 m DTW) which was 11.13% in 2000 increased to 69.88% in 2014. On an average (2010-2014) about 60% (9.03 million acre) of the IBIS was waterlogged (disaster area), which is 40% more than the pre-monsoon. Overall trend of disaster area for post-monsoon (2010-2014) indicates an alarming situation in Sindh Province.

**Keywords:** Groundwater, Waterlogging, Depth to Watertable, Pre and Post Monsoon

## 1. Introduction

Indus Basin represents an extensive groundwater aquifer covering a gross command area of 19.33 million hectares. Watertable was well below the surface and aquifer was in a state of hydrological equilibrium before the development of canal irrigation system in the early 19th century. The recharge to aquifer from rivers and rainfall was balanced by outflow and crop evapotranspiration. The major part of the present day Indus basin irrigation system was completed around 1880. Since the introduction of the canal irrigation system, percolation to the aquifer was increased in irrigated areas of the Indus basin resulting in twin menace of waterlogging and salinity. Estimated recharge to the groundwater in the Indus Basin is around 56 billion m3, out of which 36 billion m3 occurs in areas of usable groundwater.

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Pakistan, once a water-surplus country, is now a water deficit country. The demand for water is increasing rapidly while the opportunities for further development of water resources or maintaining their use to existing levels are diminishing. The shortage of water particularly in Rabi season has further aggravated the ongoing water crisis. Pakistan in the category of a high water stress country and current per capita water availability is less than 1000 $m^3$/person, but due to lack of water management practices in the country and particularly in Sindh Province, the logging is increasing.

The high canal supplies and irrigation malfunctioning (seepage losses, tail-enders getting less or even no supplies in certain areas) are major explanations for extensive Waterlogging in Sindh Province. Average annual discharges in the canals of Sindh are 134110 cfs that is approximately 1032.5mm over CCA. In the year 1959 waterlogged area was 12.4% of the irrigated area, that steeply raised up to about 38.5% in 1999. Waterlogging can cause different problems like low cropping intensity, reduced farm yields and public health issues due to difficulty of developing rural sanitation facilities in waterlogged areas.

Waterlogging, salinity, pollution, and water scarcity are potential threats for sustainable use of land and water resources. Inadequate data and inappropriate analytical systems is a constraint for decision-making processes. To meet this perspective authentic and accurate spatial and non-spatial information about groundwater resources is needed. For management of the groundwater resources, traditional approach is to use computational process as decision support tools. With the introduction of Geographical Information System (GIS) traditional data analysis techniques to spatial data analysis have been diverted through GIS. The methodology for this study takes into account the existing GIS database and Remote Sensing capabilities of IWASRI to achieve the following objective.

### 1.1 Objective of Study

To compare the temporal changes in groundwater depth with respect to drought and normal years in irrigated areas of Sindh and Balochistan.

## 2. Methodology

### 2.1 Study Area

SCARPs Monitoring Organization (SMO) was engaged in monitoring the watertable depth in Sindh and Balochistan Provinces biannually i.e. pre-monsoon and post-monsoon through a network of observation wells. Mainly the study was conducted in irrigated areas of Sindh and Balochistan. Data received from SMO for the period of 2000, 2010 to 2014 was screened via removing errors prior to data analysis. The screening process involved checking raw data, identifying outliers and dealing with missing data. The screened data was used in preparation of depth to watertable contours maps using GIS technique. A spreadsheet has been developed for conducting analysis of depth to watertable data derived via contour maps presented in the form of bar charts.

## 3. Results and Discussion

### 3.1 Waterlogging in Sindh and Balochistan

In Sindh, the non-perennial canals are suffering the most from this twin menace of waterlogging and salinity. These non-perennial areas receive more than required supplies in the Kharif season, thus, the watertable rises significantly, which also acts as secondary menace for sowing of Rabi crops, particularly the wheat. Thus, many of the lands offer only one cropping pattern. Rice Canal is one of the prominent examples of such a phenomenon, where the watertable fluctuates between 0-1.5 meters during Kharif and Rabi. The command areas of

these canal commands are given in Table 3.1. This annual cycle of rise and fall of watertable brings the salts to the upper soil strata (Basharat *et al*., 2014).

***Table 3.1:*** Canal Command Areas in Sindh & Balochistan

| Sr. No | SINDH & BALOCHISTAN | Area (Mha) |
|---|---|---|
| 1 | B.S. Feeder | 1.232 |
| 2 | Dadu | 0.583 |
| 3 | Desert Canal | 0.465 |
| 4 | Fuleli | 1.049 |
| 5 | Ghotki | 0.984 |
| 6 | K.B Feeder | 0.718 |
| 7 | Khairpur East | 0.484 |
| 8 | Khairpur west | 0.308 |
| 9 | Lined Channel | 0.604 |
| 10 | Nara Canal | 2.431 |
| 11 | North West | 1.003 |
| 12 | Pat Feeder | 0.654 |
| 13 | Pinyari | 0.981 |
| 14 | Rice Canal | 0.602 |
| 15 | Rohri Canal | 2.928 |
| | Total | 15.026 |

### 3.1.1 Pre-monsoon

Table 3.2 and Figures 3.4 & 3.5 reveals that during pre-monsoon 2014, 2013, 2012, 2011, 2010 and 2000 waterlogged (disaster area) was 9.97% (1.49 million acre), 14.86% (2.23 million acre), 20.56% (3.08 million acre), 53.72% (8.07 million acre), 7.92% (1.19 million acre) and 11.13% (1.67 million acre) respectively. The result shows that during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 11.13% in 2000 increased to 53.72% in 2011 after which it gradually decreased in the preceding years i.e., 2012, 2013 and 2014. On an average (2010-2014) about 21% (3.21 million acre) was waterlogged (disaster area) in Sindh and Balochistan Provinces. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent.

### 3.1.2. Post-monsoon

Table 3.2 and Figures 3.4 & 3.5 shows that during post-monsoon 2014, 2013, 2012, 2011, 2010 and 2000 the waterlogged area (disaster area) was 56.41% (8.47 million acre), 48.92% (7.35 million acre), 69.88% (10.05 million acre), 69.66% (10.46 million acre), 56.41% (8.47 million acre), and 46.74% (1.67 million acre) respectively. The result shows that during post-monsoons that waterlogged areas (<1.50 m DTW) which was 11.13% in 2000 increased to 69.88% in 2014. On an average (2010-2014) about 60% (9.03 million acre) of the IBIS was waterlogged

(disaster area), which is 40% more than the pre-monsoon. Overall trend of disaster area for post-monsoon (2010-2014) indicates an alarming situation in Sindh Province.

***Table 3.2:*** Area under Different DTW Classes (cm) along with Percentage Area of Sindh for the Pre and Post-monsoon Period 2000 and from 2010 to 2014

| PRE-MONSOON | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0-150 | | 150-300 | | 300-600 | | 600-1200 | | >1200 | | TOTAL |
| | Area (acre) | % | Area (acre) | % | Area (acre) | % | Area (acre) | % | Area (acre) | % | Area (acre) |
| **2000** | 1672957 | 11.13 | 5538848 | 36.86 | 7310108 | 48.65 | 480521 | 3.2 | 23926 | 0.16 | 15026360 |
| **2010** | 1190399 | 7.92 | 9456520 | 62.93 | 3602044 | 23.97 | 757615 | 5.04 | 19782 | 0.13 | 15026361 |
| **2011** | 8072768 | 53.72 | 6485381 | 43.16 | 467674 | 3.11 | 536 | 0 | 0 | 0 | 15026360 |
| **2012** | 3089219 | 20.56 | 9749130 | 64.88 | 1805787 | 12.02 | 376812 | 2.51 | 5413 | 0.04 | 15026360 |
| **2013** | 2232572 | 14.86 | 10475035 | 69.71 | 1946712 | 12.96 | 366950 | 2.44 | 5091 | 0.03 | 15026360 |
| **2014** | 1498262 | 9.97 | 10209470 | 67.94 | 2863295 | 19.06 | 448595 | 2.99 | 6739 | 0.04 | 15026360 |

| POST-MONSOON | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0-150 | | 150-300 | | 300-600 | | 600-1200 | | >1200 | | TOTAL |
| | Area (acre) | % | Area (acre) | % | Area (acre) | % | Area (acre) | % | Area (acre) | % | Area (acre) |
| **2000** | 7023609.5 | 46.74 | 4507649.7 | 30 | 2356699.4 | 15.68 | 1138422.4 | 7.58 | 0 | 0 | 15026360 |
| **2010** | 8476144 | 56.41 | 4906442 | 32.65 | 1427591 | 9.5 | 213532 | 1.42 | 2651 | 0.02 | 15026360 |
| **2011** | 10467956 | 69.66 | 3186542 | 21.21 | 1068902 | 7.11 | 299898 | 2 | 3062 | 0.02 | 15026360 |
| **2012** | 10500507 | 69.88 | 3168791 | 21.09 | 1098967 | 7.31 | 256142 | 1.7 | 1953 | 0.01 | 15026360 |
| **2013** | 7350984 | 48.92 | 5989185 | 39.86 | 1324330 | 8.81 | 350024 | 2.33 | 11837 | 0.08 | 15026360 |
| **2014** | 8476144 | 56.41 | 4906442 | 32.65 | 1427591 | 9.5 | 213532 | 1.42 | 2651 | 0.02 | 15026360 |

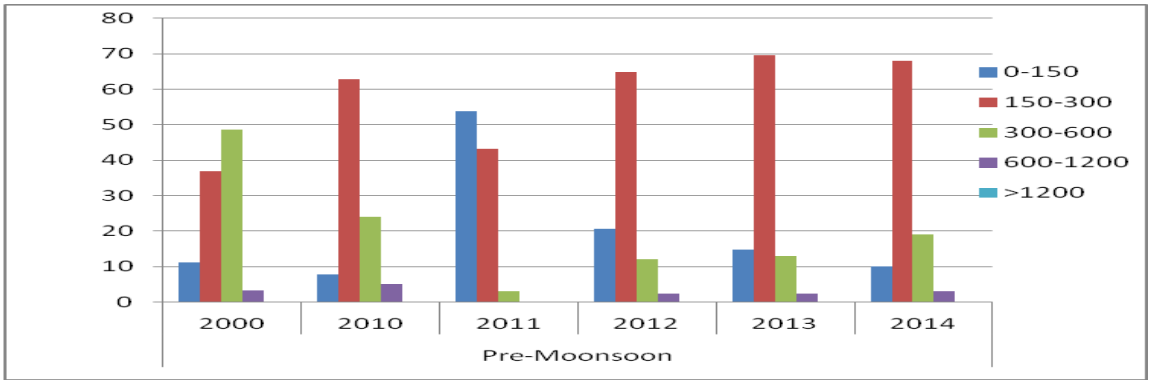



***Figure 3.4:*** Area (%) under Different DTW Classes (cm) of Sindh for the Pre-monsoon Period 2000 and from 2010 to 2014

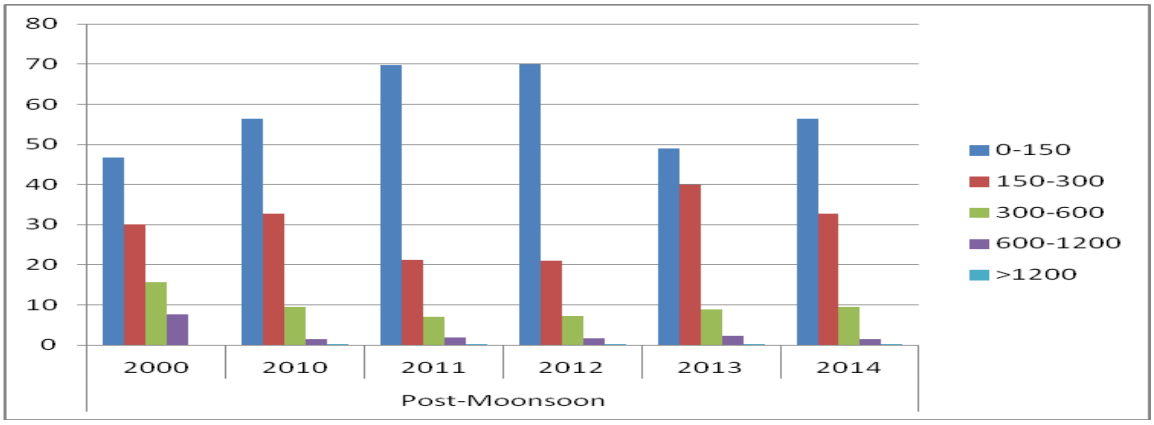



***Figure 3.5:*** Area (%) under Different DTW Classes (cm) of Sindh for the Post-monsoon Period 2000 and from 2010 to 2014

## 3.2 Spatial and Temporal Distribution of Depth to Groundwater Regions in Sindh and Balochistan Province

In Sindh and Balochistan province there are 15 canal commands that deliver to of 15.03 million acre area which is 32% of total IBIS. There are four canals i.e., B.S. Feeder, Fuleli, Nara and Rohri canal commands having one to three million acre area and other canals having canal command area less than one million acre.

### 3.2.1 B.S. Feeder

### (i) Waterlogging Condition

### Pre-Monsoon

Figures 3.6 reveals that during pre-monsoon 2000, 2010, 2011, 2012, 2013 and 2014 waterlogged area was less than 1% except 2011 when it was 31% due to heavy rainfall. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent except 2011 during pre-monsoon.

### Post-monsoon

Figures 3.6 exhibits that during post-monsoon 2000, 2010, 2011, 2012, 2013 and 2014 waterlogged (disaster area) of B.S. Feeder Canal Command was 95.23% (1.17 million acre), 75.56% (0.93 million acre), 47.03% (0.57 million acre), 97.95% (1.20 million acre), 73.75% (0.90 million acre), and 79.94% (0.98 million acre) respectively. The result shows during post-monsoons that waterlogged areas (<1.50 m DTW) which was 95.23% in 2000 decreased slightly to 79.94% in 2014. On an average (2010-2014) about 75% of B.S. Feeder canal command was waterlogged (disaster area), which is 75% more than pre-monsoon. The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly in B.S. Feeder Canal Command.

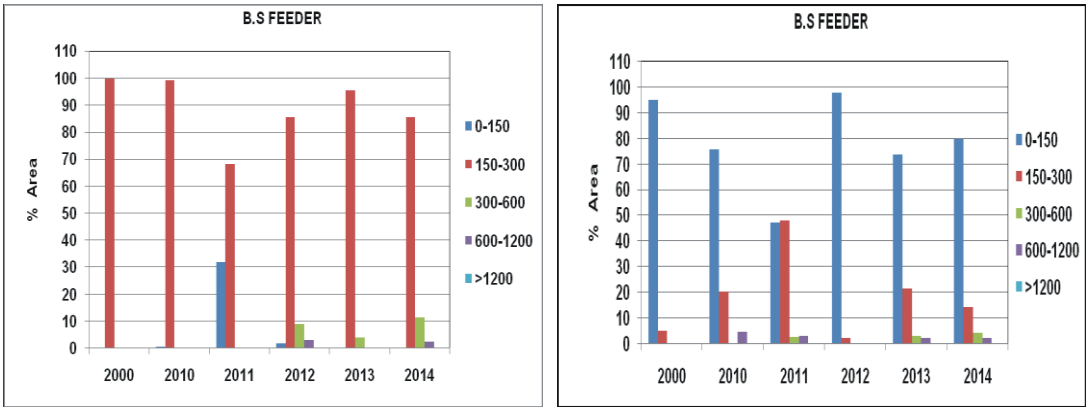



***Figure 3.6:*** Percentage Area of B.S. Feeder for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.2 Dadu Canal Command

### (i) Waterlogging Condition

### Pre-monsoon

Figures 3.7 shows that during post-monsoon 2000, 2010, 2011, 2012, 2013 and 2014 waterlogged (disaster area) of Dadu Canal Command was 60.22% (0.35 million acre), 44.51% (0.25 million acre), 41.38% (0.24 million acre), 41.38% (0.24 million acre), 41.38% (0.24 million acre), and 37.95% (0.22 million acre) respectively. The result shows during post-monsoons that waterlogged areas (<1.50 m DTW) which was 60.22% in 2000 decreased to 37.95% in 2014. On an average (2010-2014) about 41% of Dadu canal command was waterlogged (disaster area), which is 23% more than pre-monsoon.

### Post-monsoon

The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area decreased significantly in Dadu Canal Command. The result indicates that waterlogged (disaster area) of Dadu Canal Command is significant in Sindh and trend for post-monsoon 2010 to 2014 indicates alarming situation in Sindh Province. The waterlogged (disaster area) during post-monsoon is due to inequitable water allowance, no correlation between water allowance and crop-water requirement, over irrigation, flooding, seepage, improper cropping pattern and insufficient drainage system.

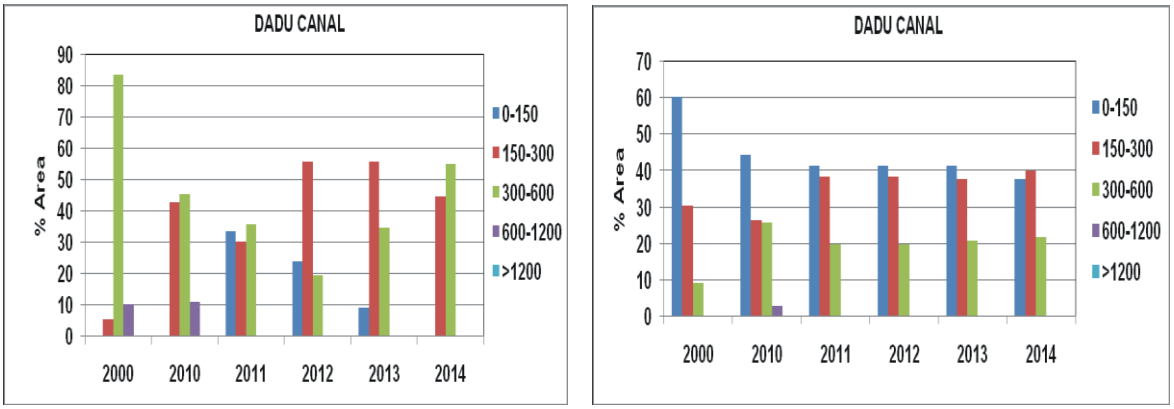



***Figure 3.7:*** Percentage Area of Dadu Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.3 Desert Canal Command

### (i) Waterlogging Condition

### Pre-monsoon

Figures 3.8 exhibit that during pre-monsoon 2000, 2010, 2011, 2012, and 2014 waterlogged (disaster area) of Desert Canal Command was 43.20% (0.19 million acre), 11.21% (0.052 million acre), 28.89% (0.13 million acre), 4.72% (0.021 million acre), and 9.52% (0.044 million acre) respectively. On an average (2010-2014) result showed that about 13% of the Desert Canal command area was depleted during pre-monsoon. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent.

### Post-monsoon

Figures 3.8 exhibit that during post-monsoon 2000, 2010, 2011, 2012, and 2014 waterlogged (disaster area) of Desert Canal Command was 67.98% (0.31 million acre), 97.82% (0.45 million acre), 94.57% (0.43 million acre), 4.72% (0.021 million acre) and 99.73% (0.46 million acre) respectively.

The result shows during post-monsoons that waterlogged areas (<1.50 m DTW) which was 67.98% in 2000 increased to 99.73% during 2014. On an average (2010-2014) about 74% of Desert canal command was waterlogged (disaster area), which is 61% more than pre-monsoon. The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly in Desert canal command. The waterlogged (disaster area) for post-monsoon 2010 to 2014 of Desert Canal Command result indicates alarming situation in Sindh. The waterlogged (disaster area) during post-monsoon is on account of inequitable water allowance, no correlation between water allowance & crop-water requirement, over irrigation, flooding, seepage, improper cropping pattern and insufficient drainage system.

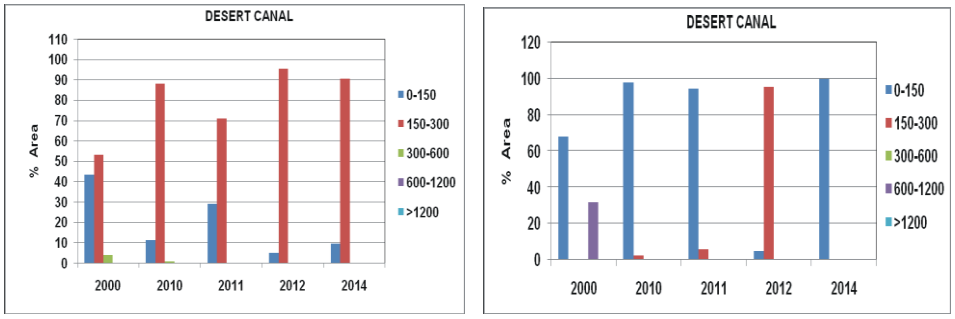



***Figure 3.8:*** Percentage Area of Desert Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.4 Fuleli Canal Command

### (i) Waterlogging Condition

**Pre-monsoon**

Figures 3.9 exhibit that during pre-monsoon 2000, 2010, 2011, 2012, 2013 and 2014 waterlogged (disaster area) of Fuleli Canal Command. The disaster area was not significant except 85.81% (0.89 million acre) and 3.46% (0.036 million acre) during 2011 and 2014 respectively. On an average (2010-2014) result showed that about 18% of the Fuleli Canal command area was depleted during pre-monsoon. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent.

**Post-monsoon**

Figures 3.9 exhibit that during post-monsoon 2000, 2010, 2011, 2012, 2013 and 2014 waterlogged (disaster area) of Fuleli Canal Command was 74.56% (0.78 million acre), 75.02% (0.78 million acre), 93, 94% (0.98 million acre), 93.95% (0.98 million acre), 59.16% (0.62 million acre) and 65.49% (0.68 million acre) respectively. The result shows that during post-monsoons (2010-2014) waterlogged (disaster area), (<1.50 m DTW) was about 77% of Fuleli canal command which is 60% more than pre-monsoon. The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly in Fuleli canal command.

The result indicates that waterlogged (disaster area) of Fuleli canal command is significant in Sindh and trend for post-monsoon 2010 to 2014 indicates alarming situation in Sindh Province. The waterlogged (disaster area) during post-monsoon is because of inequitable water allowance, no correlation between water allowance and crop-water requirement, over irrigation, flooding, seepage, improper cropping pattern and insufficient drainage system.

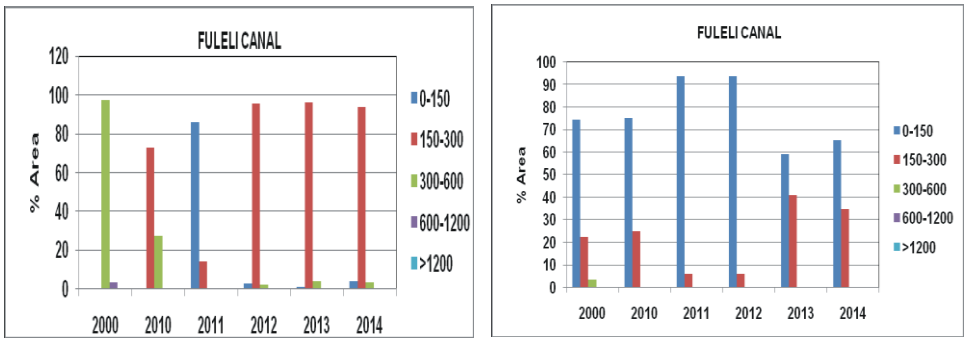



***Figure 3.9:*** Percentage Area of Fuleli Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.5 Ghotki Canal Command

#### (i) Waterlogging Condition

#### Pre-monsoon

Figures 3.10 exhibit that during pre-monsoon 2000, 2010, 2011, 2012, 2013 and 2014 waterlogged (disaster area) of Ghotki canal command. The disaster area was not significant except 60.56% during 2011 due to heavy rainfall and flooding. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent.

#### Post-monsoon

Figures 3.10 exhibit that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of Ghotki Canal Command was 52.89% (0.52 million acre), 82.63% (0.81 million acre), 80.85% (0.79 million acre), 80.85% (0.79 million acre), 78.82% (0.77 million acre) and 93.66% (0.92 million acre) respectively. The result shows that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50m DTW) was averagely about 83% of Ghotki canal command. The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly in Ghotki canal command.

The waterlogged (disaster area) of Ghotki canal command result indicates that trend for post-monsoon 2010 to 2014 is alarming in Sindh Province. The waterlogged (disaster area) during post-monsoon is due to inequitable water allowance, no correlation between water allowance and crop-water requirement, over irrigation, flooding, seepage, improper cropping pattern and insufficient drainage system.

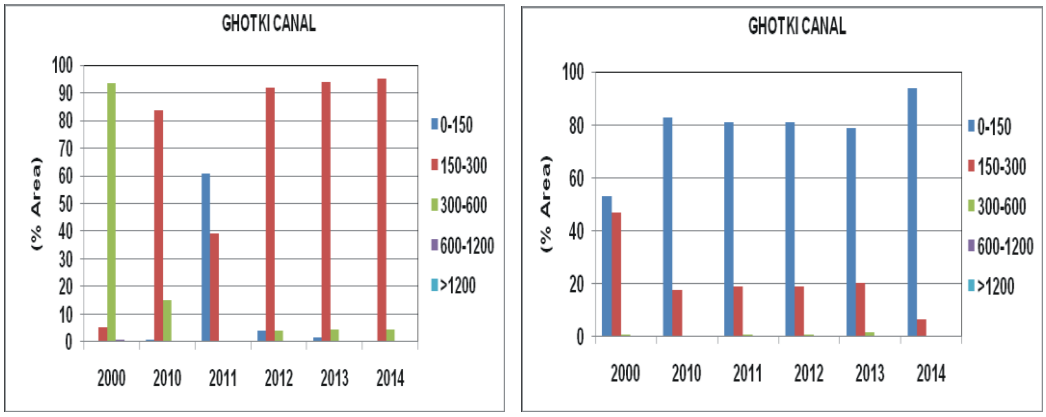



***Figure 3.10:*** Percentage area of Ghotki Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.6 K.B. Feeder Canal Command

#### (i) Waterlogging Condition

#### Pre-monsoon

Figures 3.11 reveals that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of K.B. Feeder canal command was 2.02% (0.014 million acre), 4.61% (0.033 million acre), 60.65% (0.43 million acre), 17.41% (0.12 million acre), 20.76% (0.14 million acre) and 25.70% (0.18 million acre) respectively. The result shows during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 2% in 2000 increased to 25.70% in 2014. On an average (2010-2014) about 25% of K.B. Feeder canal command was waterlogged (disaster area), having DTW less than 1.5m. The comparison of pre-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

### Post-monsoon

Figures 3.11 exhibit that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of K.B. Feeder canal command was 79.84% (0.57 million acre), 55.18% (0.39 million acre), 85.57% (0.61 million acre), 85.57% (0.61 million acre), 53.64% (0.38 million acre) and 45.64% (0.32 million acre) respectively. The results shows during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 79.84% in 2000 reduced to 45.64% in 2014. The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 65% of K.B. Feeder canal command which is 40% more than pre-monsoon. The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly from pre-monsoon in K.B. Feeder canal command.

The waterlogged (disaster area) of K.B. Feeder canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province..

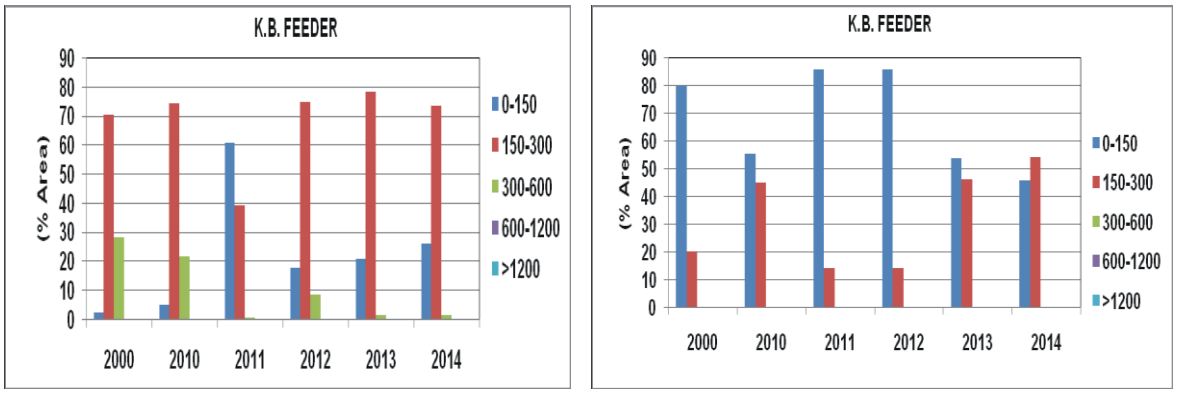



***Figure 3.11:*** Percentage Area of K.B. Feeder Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.7 Khairpur East Canal Command

### (i) Waterlogging Condition

### Pre-monsoon

Figures 3.12 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Khairpur East canal command was 0.00%, 6.27% (0.033 million acre), 43.93% (0.21 million acre), 18.45% (0.089 million acre), 23.63% (0.11 million acre) and 19.56% (0.094 million acre) respectively.

The result shows during pre-monsoon that waterlogged areas (<1.50 m DTW) which was 6.27% in 2010 increased to 19.56% in 2014. On an average (2010-2014) about 22% of in Khairpur East canal command was waterlogged, having DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

### Post-monsoon

Figures 3.12 demonstrates that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Khairpur East canal command was 20.18% (0.097 million acre), 52.06% (0.25 million acre), 66.59% (0.32 million acre), 49.32% (0.23 million acre), 33.13% (0.16 million acre) and 63.16% (0.30 million acre) respectively. The result shows during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 20.18% in 2000 increased to 63.16% in 2014.

The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 53% of in Khairpur East canal command which is 31% more than pre-monsoon. The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly in Khairpur East canal command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained consistent during post-monsoon. The waterlogged (disaster area) of Khairpur East canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

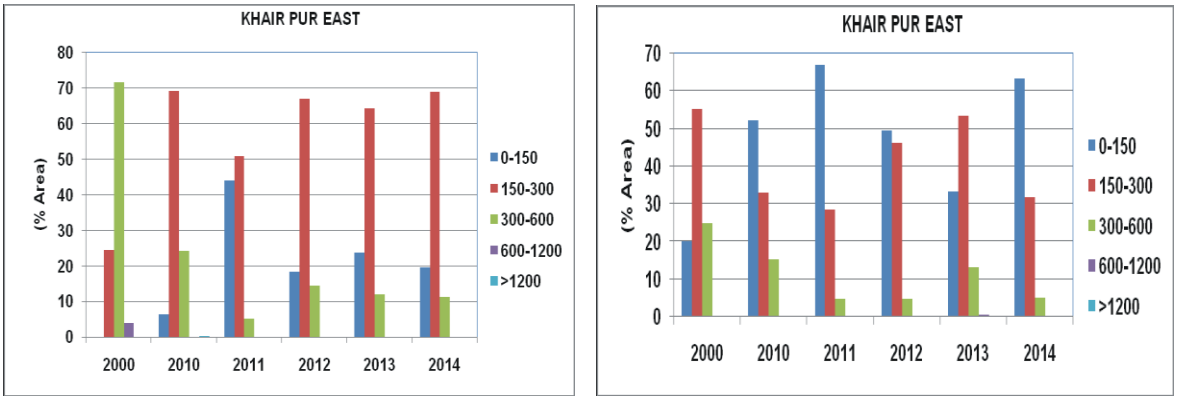



***Figure 3.12:*** Percentage Area of Khairpur East Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.8 Khairpur West Canal Command

### (i) Waterlogging Condition

#### Pre-monsoon

Figures 3.13 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Khairpur West Canal Command was 4.28% (0.013 million acre), 3.11% (0.009 million acre), 59.80% (0.18 million acre), 16.22% (0.050 million acre), 26.04% (0.080 million acre) and 24.22% (0.074 million acre) respectively. The result shows during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 4.28% in 2000 increased to 24.22% in 2014. On an average (2010-2014) about 25% of in Khairpur West Canal command was waterlogged (disaster area), having DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

#### Post-monsoon

Figures 3.13 reveals that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Khairpur West Canal Command was 16.75% (0.051 million acre), 46.92% (0.14 million acre), 36.75% (0.11 million acre), 36.75% (0.11 million acre), 37.07% (0.11 million acre) and 36.43% (0.11 million acre) respectively. The result shows during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 16.75% in 2000 increased to 36.43% in 2014.

The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 39% of in Khairpur West canal command which is 14% more than pre-monsoon. The comparison of post-monsoon for the period 2010 to 2014 with 2000 shows that percentage disaster area increased significantly in Khairpur East canal command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained consistent during post-monsoon. The waterlogged (disaster area) of Khairpur West canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

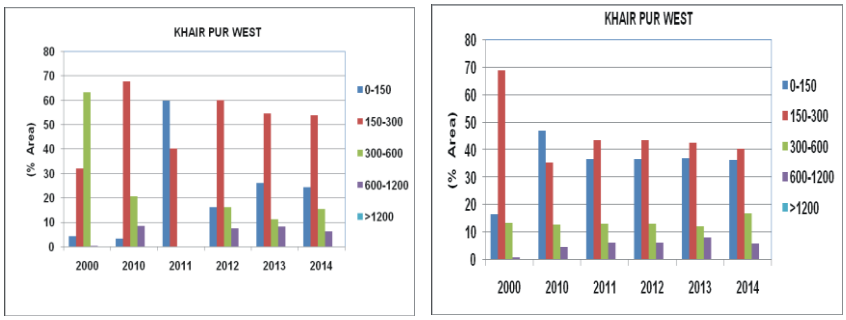



***Figure 3.13:*** Percentage Area of Khairpur West Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.9 Lined Channel Canal Command

### (i) Waterlogging Condition

#### Pre-monsoon

Figures 3.14 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Lined Channel Canal Command was 2.75% (0.016 million acre), 65.78% (0.39 million acre), 65.78% (0.39 million acre), 7.96% (0.048 million acre), 3.18% (0.019 million acre) and 2.30% (0.013 million acre) respectively. On an average (2010-2014) about 29% of in Lined Channel canal command was waterlogged (disaster area), having DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained consistent except 2011-12 during pre-monsoon.

#### Post-monsoon

Figures 3.14 demonstrates that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Lined Channel Canal Command was 74.14% (0.44 million acre), 65.78% (0.39 million acre), 98.66% (0.59 million acre), 49.07% (0.29 million acre), 55.46% (0.33 million acre) and 63.42% (0.38 million acre) respectively. The result shows during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 74.14% in 2000 decreased slightly to 63.42% in 2014.

The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 66% of in Lined Channel Canal Command which is 35% more than pre-monsoon. The results of post-monsoon for the period 2010 to 2014 with 2000 reveals that percentage disaster area was significantly increase in Lined Channel canal command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained consistent during post-monsoon. The waterlogged (disaster area) of Lined Channel canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

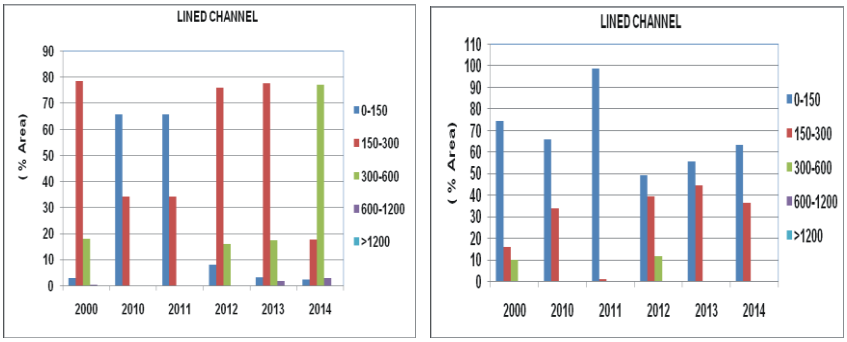



***Figure 3.14:*** Percentage Area of Lined Channel Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.10. Nara Canal Command

### (i) Waterlogging Condition

#### Pre-monsoon

Figures 3.15 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Nara canal command was 0.75% (0.018 million acre), 10.10% (0.24 million acre), 18.40% (0.44 million acre), 24.59% (0.59 million acre), 30.77% (0.74 million acre) and 25.60% (0.037 million acre) respectively. On an average (2010-2014) about 20% of in Nara canal command was waterlogged (disaster area), having DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

#### Post-monsoon

Figures 3.15 exhibits that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Nara canal command was 5.32% (0.12 million acre), 55.23% (1.34 million acre), 75.32% (1.83 million acre), 75.32% (1.83 million acre), 24.07% (0.59 million acre) and 44.07% (1.07 million acre) respectively. The result shows during pre-monsoons that waterlogged areas (<1.50 m DTW) which was 5.32% in 2000 increased significantly to 44.07% in 2014. The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 55% of in Nara canal command which is 35% more than pre-monsoon. The results of post-monsoon for the period 2010 to 2014 with 2000 reveals that percentage disaster area was significantly increase in Nara canal command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained inconsistent during post-monsoon. The waterlogged (disaster area) of Nara canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

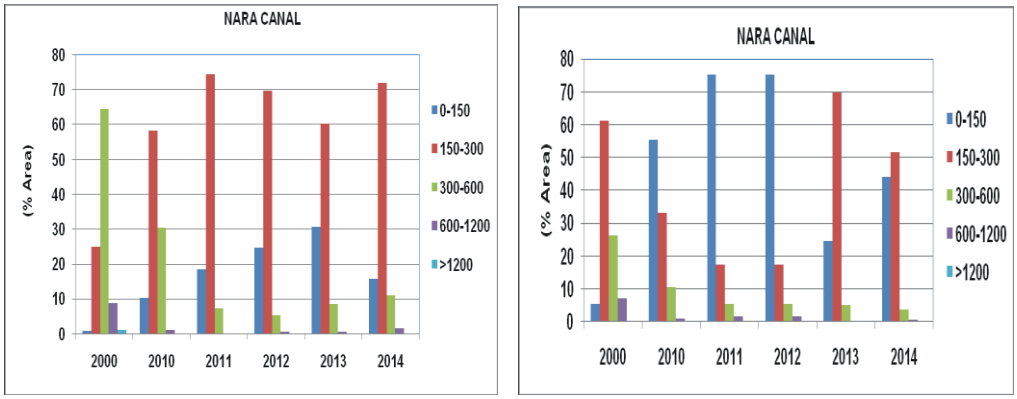



***Figure 3.15:*** Percentage Area of Nara Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.11. North-West Canal Command

### (i) Waterlogging Condition

#### Pre-Monsoon

Figures 3.16 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in North West canal command was 0.00%, 0.81%, 85.58% (0.85 million acre), 81.21% (0.81 million acre), 20% (0.20 million acre) and 7.07% (0.070 million acre) respectively. On an average (2010-2014) about 38% of in North West canal command was waterlogged (disaster area), with DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

### Post-monsoon

Figures 3.16 reveals that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in North West Canal Command was 98.96% (0.99 million acre), 99.01% (0.99 million acre), 96.67% (0.96 million acre), 96.67% (0.96 million acre), 83.72% (0.83 million acre) and 78.60% (0.78 million acre) respectively. The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 91% of in North West canal command which is 53% more than pre-monsoon. The results of post-monsoon for the period 2010 to 2014 with 2000 reveals that percentage disaster area was significantly increase in North West canal command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained consistent during post-monsoon. The waterlogged (disaster area) of North West canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

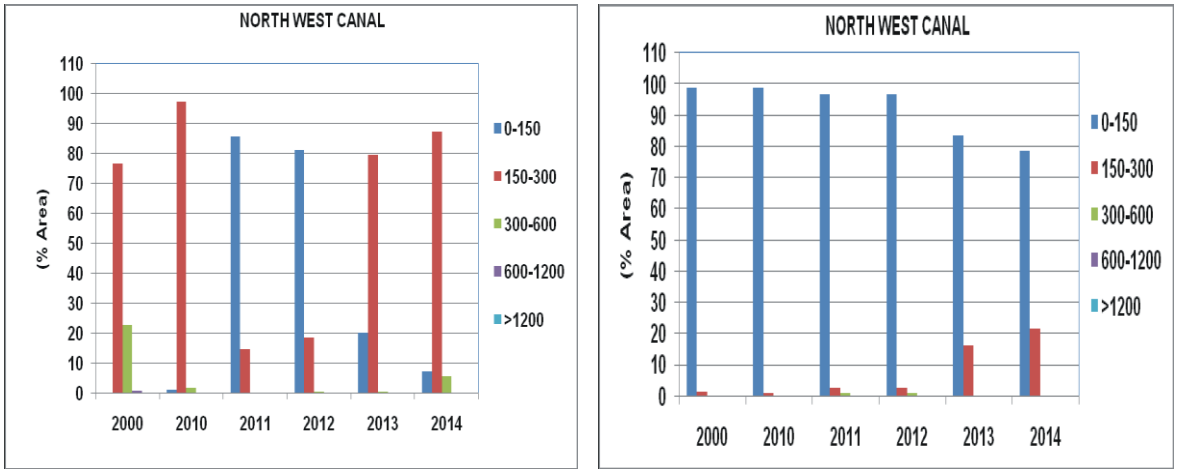



***Figure 3.16:*** Percentage Area of North West Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.12. Pat Feeder Canal Command

### (i) Waterlogging Condition

### Pre-monsoon

Figures 3.17 exhibit that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Pat Feeder canal command was not significant except during 2000 and 2011. On an average (2010-2014) about 2% of in Pat Feeder was waterlogged (disaster area), having DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

### Post-monsoon

Figures 3.17 exhibit that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Pat Feeder canal command was 7.34% (0.047 million acre), 100% (0.65 million acre), 5.64% (0.036 million acre), 5.64% (0.036 million acre), 5.18% (0.033 million acre) and 10.36% (0.067 million acre) respectively.

The result reveals that during post-monsoons (2011-2014) waterlogged (disaster area) (<1.50m DTW) was averagely about 6% of in Pat Feeder Canal Command which is 4% more than pre-monsoon, whereas during 2010 results was unexpected. Overall trend of disaster area for post-monsoon 2011 to 2014 remained consistent during post-monsoon.

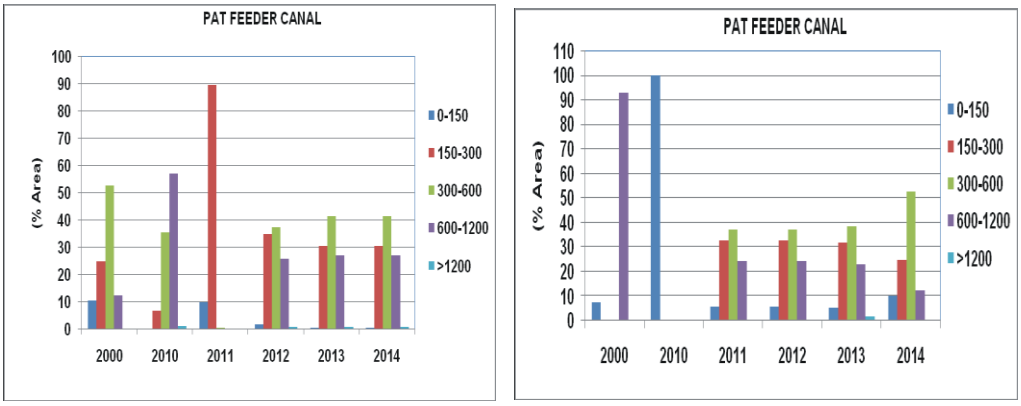



***Figure 3.17:*** Percentage Area of Pat Feeder for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.13. Pinyari Canal Command

### (i) Waterlogging Condition

### Pre-monsoon

Figures 3.18 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Pinyari canal command was not significant except 2011 when there was heavy rainfall. On an average (2010-2014) about 6% of in Pinyari canal command was waterlogged (disaster area), having DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

### Post-monsoon

Figures 3.18 demonstrates that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Pinyari Canal Command was 78.58% (0.77 million acre), 45.46% (0.44 million acre), 86.74% (0.85 million acre), 86.74% (0.85 million acre), 45.86% (0.44 million acre) and 47.40% (0.46 million acre) respectively. The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 62% of in Pinyari canal command which is 56% more than pre-monsoon. The results of post-monsoon for the period 2010 to 2014 with 2000 reveals that percentage disaster area was significantly increase in Pinyari canal command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained inconsistent during post-monsoon. The waterlogged (disaster area) of Pinyari canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

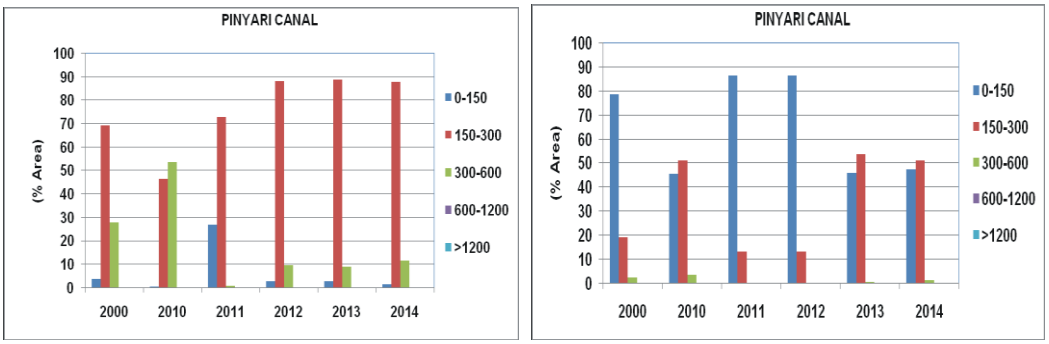



***Figure 3.18:*** Percentage area of Pinyari Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.14. Rice Canal Command

### (i) Waterlogging Condition

#### Pre-monsoon

Figures 3.19 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Rice canal command was 0.00%, 0.45%, 96.10% (0.57 million acre), 58.21% (0.35 million acre), 14.84% (0.089 million acre) and 7.147% (0.042 million acre) respectively. On an average (2010-2014) about 35% of in Rice canal command was waterlogged (disaster area), having DTW less than 1.5m. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent during pre-monsoon.

#### Post-monsoon

Figures 3.19 reveals that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Rice Canal Command was 98.53% (0.59 million acre), 82.12% (0.49 million acre), 83.27% (0.50 million acre), 83.27% (0.50 million acre), 99.72% (0.60 million acre) and 100% (0.60 million acre) respectively. The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 89% of in Rice canal command which is 54% more than pre-monsoon. The result of post-monsoon for the period 2010 to 2014 with 2000 reveals that percentage disaster area was significantly increase in Rice Canal Command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained consistent during post-monsoon. The waterlogged (disaster area) of Rice canal command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

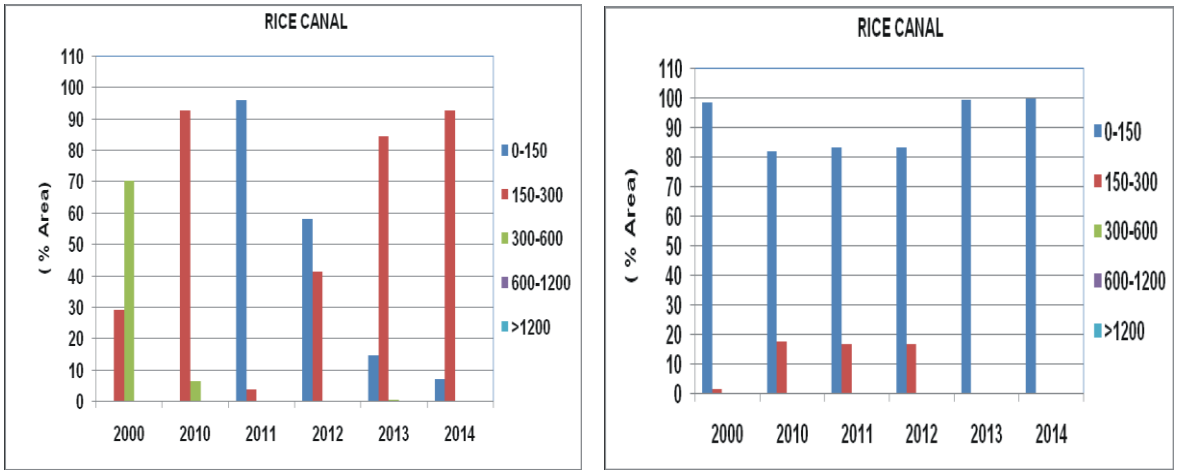



***Figure 3.19:*** Percentage Area of Rice Canal for the Pre-Monsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

### 3.2.15 Rohri Canal Command

### (i) Waterlogging Condition

#### Pre-monsoon

Figures 3.20 exhibits that during pre-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Rohri Canal Command was 44.66% (1.3 million acre), 13.39% (0.39 million acre), 82.47% (2.41 million acre), 24.79% (0.72 million acre), 23.24% (0.68 million acre) and 18.34% (0.53 million acre) respectively. On an average (2010-2014) about 32% of in Rohri canal command was waterlogged (disaster area), with DTW less than 1.5 m. The result of post-monsoon for the period 2010 to 2014 with 2000 reveals that percentage disaster area was

decreased in Rohri canal command over time. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained consistent except 2011 because of heavy rainfall during pre-monsoon.

### Post-monsoon

Figures 3.20 shows that during post-monsoon 2000, 2010, 2011, 2012, 2013, and 2014 waterlogged (disaster area) of in Rohri Canal Command was 6.05% (0.17 million acre), 39.28% (1.15 million acre), 54.34% 1.59 million acre), 61.37% (1.79 million acre), 28.18% (0.82 million acre) and 36.71% (1.07 million acre) respectively.

The result reveals that during post-monsoons (2010-2014) waterlogged (disaster area) (<1.50 m DTW) was averagely about 44% of in Rohri Canal Command which is 12% more than pre-monsoon. The result of post-monsoon for the period 2010 to 2014 with 2000 reveals that percentage disaster area was significantly increase in Rohri canal command. Overall trend of disaster area for post-monsoon 2010 to 2014 remained inconsistent during post-monsoon. The waterlogged (disaster area) of Rohri Canal Command indicates alarming situation for post-monsoon 2010 to 2014 in Sindh Province.

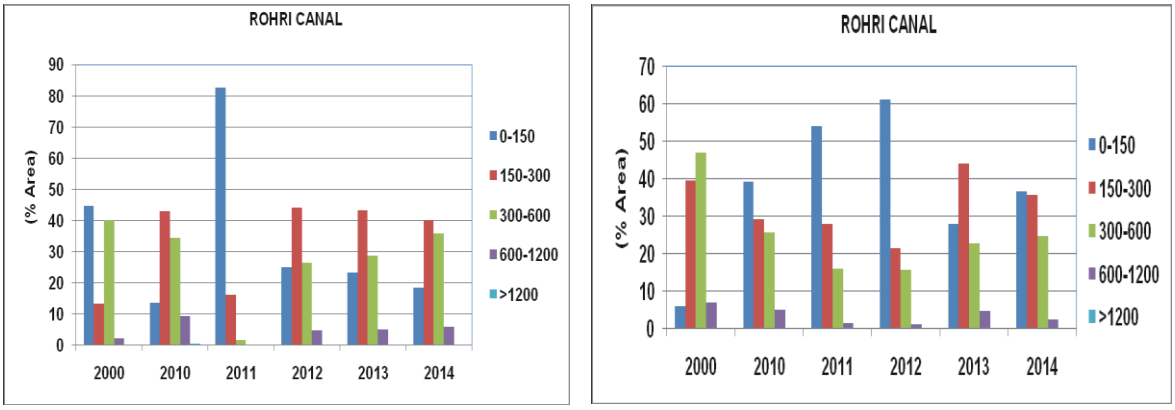



***Figure 3.20:*** Percentage Area of Rohri Canal for the Pre-Moonsoon (L) and Post-Monsoon (R) Period 2000, 2010 to 2014

## 3.3. Groundwater Depletion in Sindh and Balochistan

The results indicate that during pre and post-monsoon 2010-2014 the most depleted area (>12 m DTW) was less than 1%, showing stability.

## 4. Conclusions and Recommendations

## 4.1 Waterlogging Condition in Sindh and Balochistan Provinces

- The result demonstrates that during pre-monsoons the waterlogged areas (<1.50 m DTW) which was 11% in 2000 increased to 53% in 2011 after which it gradually decreased in the preceding years i.e., 2012, 2013 and 2014. On an average (2010-2014) about 21% (3.21 million acre) was waterlogged (disaster area) in Sindh and Balochistan Provinces. Overall trend of disaster area for pre-monsoon 2010 to 2014 remained inconsistent.
- The result illustrates that during post-monsoons that waterlogged areas (<1.50 m DTW) which was 11% in 2000 increased to 70% in 2014. On an average (2010-2014) about 60% (9.03 million acre) of the IBIS was waterlogged (disaster area), which is 40% more than the pre-monsoon. Overall trend of disaster area for post-monsoon (2010-2014) indicates an alarming situation in Sindh Province.

## 4.2 Waterlogging Conditions in Canal Commands of Sindh and Balochistan

### (i) B.S. Feeder, Pinyari, Ghotki and Pat Feeder Canal Command

- The result reveals that waterlogged area of B.S. Feeder, Pinyari, Ghotki and Pat Feeder canal command was less than 5% during pre-monsoon (2000, 2010-2014) except 2011 when it was significant due to heavy rainfall.
- The result demonstrates that during the post-monsoons (2010-2014) waterlogged (disaster area) was averagely in the range of 62% to 83% of Pinyari, B.S. Feeder, Ghotki canal command. The comparison of pre and post-monsoon for the period 2010 to 2014 proves that percentage disaster area increased significantly and indicates frightening situation in Sindh Province.

### (ii) Dadu, Desert, Fuleli, K.B. Feeder, Khairpur East, Khairpur West, Lined Channel, North West, Nara, Rice and Rohri Canal Commands

- The waterlogged area of Dadu, Desert, Fuleli, K.B. Feeder, Khairpur East, Khairpur West, Lined Channel and Nara canal command was in the range of 13 to 25% for the period of pre-monsoon (2000, 2010-2014). Whereas, Rohri, Rice and North West canal command showed averagely 35% disaster area during pre-monsoons.
- The result exhibits that during the post-monsoons (2010-2014) waterlogged (disaster area) was averagely in the range of about 74% to 91% of Desert, Fuleli, North West and Rice canal command. The results of post-monsoon (2010-2014) provide evidences that the disaster area increased significantly and indicates frightening situation in Sindh Province.

## 4.3 Recommendations

- Groundwater and surface water constitute a closely inter-related hydrologic system which should be managed as a single entity. A long-term water management plans are required in order to prevent Waterlogging and uncontrolled pollution.
- The disaster area during post monsoon is indicating an alarming situation in Sindh Province. The increasing trend of waterlogged (disaster area) is due to inequitable water allowance, over irrigation, flooding, seepage, inappropriate cropping pattern and inadequate drainage system which need to be managed on priority basis to get rid of waterlogging and salinity problems.
- Rationalized water allocations are required, as some of the areas are over-allocated while others are under-allocated.

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SMO. 2011. SCARPs Monitoring Organization (SMO) Depth to Watertable (DTW) data sheets for 2011.

SMO. 2012. SCARPs Monitoring Organization (SMO) Depth to Watertable (DTW) data sheets for 2012.

SMO. 2013. SCARPs Monitoring Organization (SMO) Depth to Watertable (DTW) data sheets for 2013.

SMO. 2014. SCARPs Monitoring Organization (SMO) Depth to Watertable (DTW) data sheets for 2014.

**Paper No. 781**

# PCRWR IRRIGATION ADVISORY SERVICES FOR DIFFERENT AGRO-CLIMATIC ZONES OF PAKISTAN – AN IMPACT ANALYSIS

**Muhammad Ashraf, Ashfaq Ahmad Sheikh, Ahmad Zeeshan Bhatti & Adil Altaf**

# PCRWR IRRIGATION ADVISORY SERVICES FOR DIFFERENT AGRO-CLIMATIC ZONES OF PAKISTAN – AN IMPACT ANALYSIS

By

**Muhammad Ashraf[1], Ashfaq Ahmed Sheikh[1], Ahmad Zeeshan Bhatti[1], Adil Altaf[1]**

## Abstract

In Pakistan, more than 80% water resources of the country are used by agriculture; nevertheless, it is the major limiting factor for agricultural sustainability and growth. Indus Basin Irrigation System (IBIS) is one of the largest in the world, but one of the least efficient. The overall IBIS losses are about 60 percent, which partially contribute to groundwater recharge. Groundwater is contributing more than 60% of the irrigation water requirements, but at a cost of huge chunk of energy. The present irrigation water deficit in the country is about 15%, which is predicted to be above 30% in 2025.

A major portion of irrigation water loss takes place in the fields (about 25%) where flood irrigation is practiced. Over irrigation is a common practice due to classical mindset that more water would produce more yields. Ultimately, irrigated agriculture is becoming less profitable business owing to high irrigation cost, particularly in groundwater-irrigated areas. Resultantly, food is becoming more expensive depriving a substantial portion of the society.

Quantification of crop water requirements is pre-requisite for optimizing water use efficiency. Water requirements depend mainly on the type and stage of crop growth and climatic conditions. A simplified approach to determine crop water requirements ($ET_c$) is linking it with reference evapotranspiration ($ET_o$) by a crop coefficient ($K_c$). Remote sensing techniques have made it possible to estimate the real time reference ET of the near past. The techniques are used worldwide to estimate crop water requirements of different crops and the irrigation advisories are broadcasted accordingly.

PCRWR cell broadcasts weekly irrigation requirements of different crops in 21 districts of Pakistan (10,000 farmers). The service is in collaboration with University of Washington, USA, which provides real time reference ET for entire Pakistan, whereas PCRWR has determined $K_c$ values of all major crops through lysimeteric studies. The message updates the farmers that how much water its crop (in inch) has used during the last week excluding effective rainfall in order to replenish the same through irrigation. The rainfall forecast for the upcoming week is also included in the message. A survey reveals that farmers who were blindly applying irrigation saved at least 15-20% water due to this messaging service.

**Keywords:** Million cubic meter, Billion cubic meter, Million acre feet, Irrigation advisory service, Indus Basin Irrigation System, Potential evapotranspiration, Evapotranspiration, Crop coefficient, Crop water requirement.

## Introduction

Water resources are becoming increasingly scarce over time, limiting sustainable agriculture and domestic water supplies in Pakistan. The renewable water resources are 246 BCM/year (199 MAF). The per capita water availability becomes around 1,333 $m^3$/person/annum ranking

[1] Pakistan Council of Research in Water Resources

142nd globally. The water availability in the neighboring countries as Bangladesh, Nepal, Sri Lanka and India are 7835, 7562 and 2482, 1526 $m^3$, respectively (FAO, 2014). According to Falkenmark et al (1992), a country is termed as water scarce when water shortage hampers health and well-being of the people for which the threshold limit is 1000 $m^3$/capita/annum.

Pakistan has the fourth largest irrigated area of the world (19.49 Mha) after India (60.85 Mha), China (57.78 Mha) and USA (22.39 Mha) and the largest contiguous irrigation system. The Indus Basin Irrigation System (IBIS) is though the largest yet least efficient with 40% overall system efficiency. The overall water use efficiency in Pakistan is 0.13 kg/$m^3$ vis-à-vis 0.39 in India. The canal supplies are highly variable and undependable due to which dependence on groundwater has increased to 60% of the irrigation water requirements. Extensive groundwater use is causing secondary salinization, deterioration of soil structure and groundwater mining besides consuming huge energy chunk. Most of the problems in irrigation sector have arisen from mismanagement of water at various levels. Over irrigation is a common practice, as farmers believe that more water would produce more yields, which is wrong. Every crop has a specific water requirement in a certain soil and climatic conditions. Lack of knowledge about crop water requirements and no irrigation scheduling are the major factors behind low water use efficiency in Pakistan.

Crop water requirement is defined as the depth of water needed to meet the water losses through evapotranspiration (ET) of a disease free crop, growing in large fields under non-restricting soil conditions achieving full production potential under the given growing environment. Accurate quantification of crop water requirements is pre-requisite for optimizing water use efficiency. Water requirements depend mainly on the type and stage of crop growth and climatic conditions (Allen et al, 1998). A simplified approach to determine crop water requirements ($ET_c$) is linking it with reference evapotranspiration ($ET_o$) by a crop coefficient ($K_c$). The crop coefficients are determined through lysimeteric studies using the prevalent soils of the region, which changes with the region, type and stage of the growth.

Remote sensing techniques have made it possible to estimate the real time reference ET of the near past. The techniques are used worldwide to estimate crop water requirements of different crops and the irrigation advisories are broadcasted accordingly (Fortes et al, 2005). PCRWR took up the initiative to provide such irrigation advisories to selected trainee farmers (700 Nos.) in Sargodha, Hyderabad and Peshawar districts. The farmers were trained under an ICARDA sponsored project named, “Watershed Rehabilitation and Irrigation Improvement in Pakistan”. In this regard, international collaboration was sought with the University of Washington, USA and NASA. The utility of instant service is limited to 700 farmers that too in bits and pieces as the messages are sent through the cell phones instead of cell broadcast being offered by the cellular phone companies. Therefore, the service suffers from disruption due to difficulty in handling the messaging through application.

The affectivity of the service therefore remained limited as it became difficult for the farmer to keep record due to disruption. The sustainability of the ongoing services could only be ensured if based on cell broadcast service. Most importantly, the respondent farmers should be much more in order to truly evaluate efficacy of the service. It was therefore deemed essential to expand the horizon the service to 10,000 farmers and adopting cell broadcast service with PCRWR mask.

Every crop has its specific water requirement, which changes over its growth period and also from one climatic zone to another. Due to lack of knowledge about crop water requirements, irrigated agriculture is becoming less profitable business, particularly for small farmers (80%) owing to high irrigation cost, especially in groundwater irrigated areas.

For proper irrigation scheduling, the knowledge of crop water requirement (CWR) is necessary. A simplified approach to determine crop water requirements ($ET_c$) is linking it with reference evapotranspiration ($ET_o$) by a crop coefficient ($K_c$). PCRWR has determined $K_c$ values of all the major crops through lysimertic studies in the upper and lower Indus basin of Pakistan through decade's long studies. Determining the CWR using historical data is laborious and most of the times gives unrealistic results as the rainfalls and $ET_o$ during the last week can significantly alter the CWR (Steppe et al, 2008).

Irrigation is considered one of the most complex operations that a farmer can carry out due to multiple factors that affect irrigation management (Knox et al, 2011). Most of the irrigation practices are based upon experience and long term averages of the past. As a result, in many cases, it leads to wrong irrigation management, resulting in low water-use efficiency and adverse effects on the environment. That is why the real-time Irrigation Advisory Services (IAS) is becoming increasingly popular in the world for improving irrigation water management. In a number of developed countries, real-time irrigation advisories are issued to the farmers for rationalizing the irrigation water use. In addition, at a number of places advisories are being issued on design and installation of irrigation systems, water quality control, and certain agricultural advisory services e.g. pest management, marketing, new alternative crops, etc. This work is envisaged for extending real time irrigation advisory for 10,000 farmers of different agro-climatic zones and its improvement over time based the farmers' feedback.

## Materials and Methods

Irrigation scheduling is the key to optimizing water use efficiency. Knowledge of the net crop water consumption excluding rainfall on weekly basis could play a significant role in rationalizing irrigation by the farmers. Moreover, if a farmer is also told the rainfall forecast during the upcoming week, he can further optimize irrigation decisions. The UW provided real time reference ET data through its web portal, which is downloaded on weekly basis. PCRWR maintains a directory of $K_c$ values over the growth period of different crops in different agro-climatic zones of Pakistan. The irrigation advisories were issued considering the past weekly reference ET and respective $K_c$, as well as rainfall forecast for the upcoming week.

## PCRWR Lysimetric Studies

PCRWR has done decade's long lysimeteric studies at its Research Stations at Lahore (Punjab) and Tando Jam (Sindh). A lysimeter may be regarded as a container in which the crops are grown under controlled natural conditions. The studies are particularly focused on determination of crop water requirements in a particular region. The native soils are used and same crop is grown in the nearby fields to minimize oasis effect. The climatic conditions under which crop is grown are monitored, water-table depths are maintained and all water related factors like rainfall, irrigation applied, groundwater contribution, drainage effluent are measured. The output of the study is determination of crop coefficient ($K_c$) values, which correlates the temporal variation of crop water requirements with reference evapotranspiration ($ET_o$). PCRWR has therefore managed to develop a directory of $K_c$ values for the major crops of the regions.

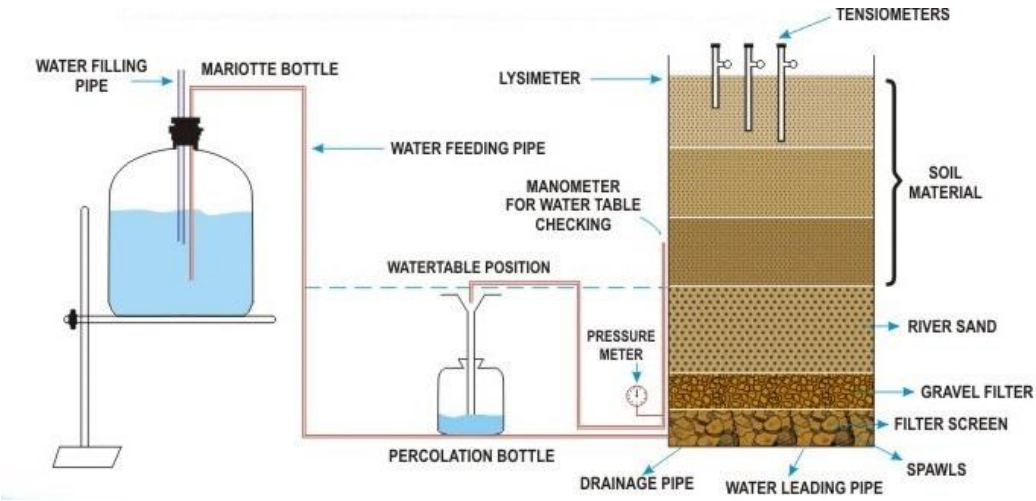



***Figure 1:*** A schematic diagram of the lysimeter

## Reference ET – Historical and Latest Approaches

The Reference ET for a particular region and season are established based on long-term historical data. It becomes practical to determine the average crop water requirements on weekly or fortnightly basis and develop irrigation schedule thereof. Worldwide trends of irrigation advisory are based upon the real time data instead of historical (Smith and Muñoz, 2002). University of Washington, USA is providing such real-time $ET_o$ data on daily basis.

## University of Washington, USA Collaboration

University of Washington's SASWE research group has developed the system. The $ET_o$ is calculated on daily basis for the entire Pakistani region at 10 km grids as per PCRWR specifications. This system visualizes the dynamic crop water requirement for easy interpretation at the web portal http://depts.washington.edu/saswe . The ETo is estimated using FAO56 report which is essentially a modification of the well-known Penman-Monteith equation (Richard et al, 1998). The input parameters of the equation are temperature (max and min), humidity, wind speed and solar radiation as inputs. These now cast inputs were obtained from global Numerical Weather Prediction (NWP) modeling system called the Global Forecast System (https://www.ncdc.noaa.gov/data-access/model-data/model-datasets/global-forcast-system-gfs). Independent validation by PCRWR against lysimeter data revealed acceptable agreement. The web portal is open access and therefore can be seen anywhere in Pakistan (http://depts.washington.edu/saswe/). Similarly, the portal also provides calibrated rainfall in different parts of Pakistan; the database is updated on daily basis. The ASCII data of each day is downloaded in Global Mapper as shown in Figure 2:

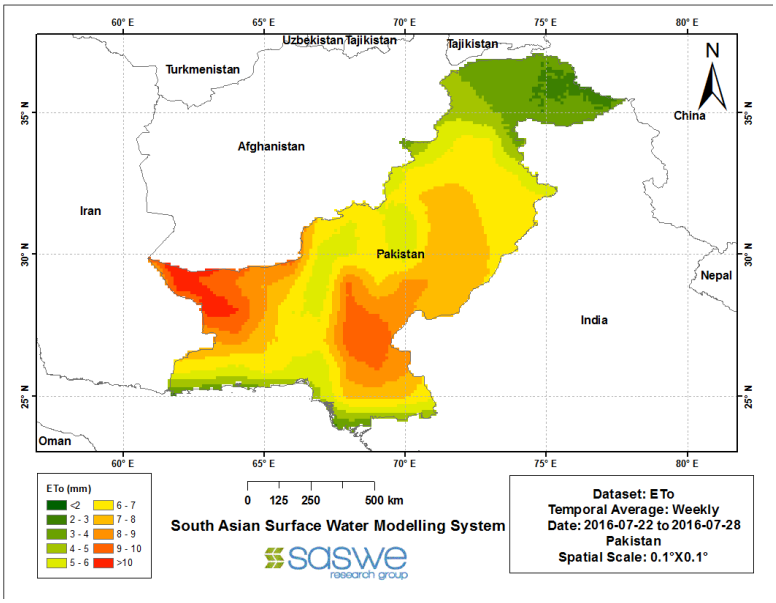



***Figure 2:*** A sample $ET_o$ information provided by UW

## PCRWR IAS Procedure

The major crops of the Pakistan; wheat, maize, sugarcane, cotton, rice and banana are particularly targeted in the current service. The calculations of the advisory service proceeds as follows:

1. The ETo values of the preceding week were downloaded and arranged for different regions at an over-estimation factor of 1.5 (as determined by PCRWR during the validation).
2. The effective rainfall (75% of the total rainfall) was downloaded and cumulated.
3. The net weekly crop consumption was calculated by cumulating the weekly ETo subtracting the effective precipitation multiplication with the respective Kc value of the crop and dividing by an application efficiency of 75%.
4. PCRWR is using cell broad cast service of Telenor Pakistan for delivery of weekly messages to 100,00 farmers. A sample message is included in the Figure 3:




***Figure 3:*** A sample message received by the beneficiary

5. The beneficiary farmers are being interviewed on phone and the feedback is compiled for further improving the message. The inventory of the farmers is developed using relevant information sources (University of Agriculture, Faisalabad, Agriculture Extension Department and ICARDA). Districts are selected depending upon different agro-climatic zones of the country for better distribution.
6. The broadcasted messages of the last four weeks for different districts are uploaded on PCRWR website for ready reference.

The logical flow sheet of the message formulation and delivery is given in Figure 4:

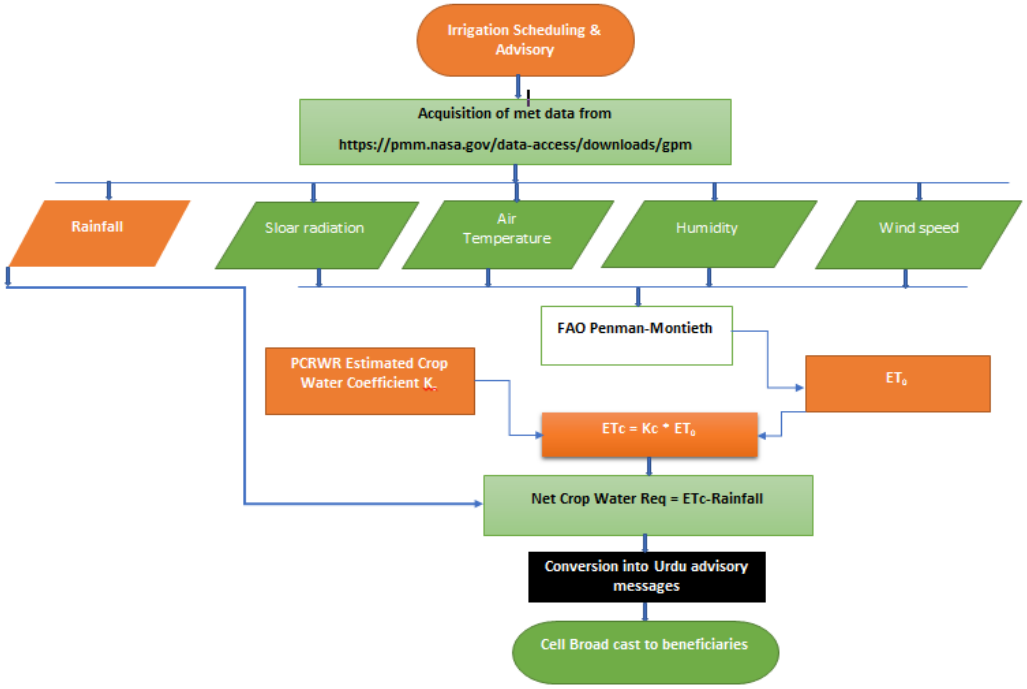



***Figure 4:*** Logical flow diagram of the message formulation and broadcast

## Results and Discussion

The beneficiaries are provided call back number for further assistance and feedback in the message. The farmers were also called randomly to get their feedback, however, a specific analysis was carried out during August, 2017 by calling the beneficiaries of different agro-climatic zones to assess the impact of the services on water conservation and yield improvement (Martin et al, 2003). It was targeted to get feedback of 25 beneficiaries from all 4 agro-climatic zones; rice-wheat, cotton-wheat, mixed cropping and orchard-crops.

A questionnaire carrying the following queries was developed: (i) Name (along with the district and contact number) (ii) cultivated area (iii) crops grown (iv) irrigation water sources (v) weekly message delivery/receipt (vi) message understanding (vii) message utilization (viii) any issues in using message (ix) reduction in water use (x) yield improvement and (xi) willingness for further interview.

The results of the survey are quite encouraging as more than 70% farmers confirmed that they are not only receiving the messages on weekly basis but making use of it in terms of rationalizing their irrigation. This healthy response may be attributed to the selection of the IAS beneficiary farmers by PCRWR. The IAS beneficiary farmers have had got some kind of training

by some governmental or non-governmental organizations for similar advisories about fertilizer and pesticides.

When the farmers were asked that how the messaged information helped them in terms of rationalizing their irrigation, they told that either we delay the irrigation until the total water consumption excluding the rainfall reaches 3 inches or we apply it as water becomes available but up to the depleted depth as communicated by PCRWR. When enquired, how you measure the depleted depth (inches)? Most of the farmers told that we use our fingers for water depth measurement. A finger has 3 portions, each approximately equals 1 inch, and "So we shut off the irrigation as soon as the water depth reaches the depleted depth as we continuously measure it with our fingers in different parts of the field".

Moreover, sometimes we skip the irrigation (groundwater irrigated areas) in the wake of expected rainfall in the upcoming week as predicted by PCRWR. In most of the cases (80%) the rainfall prediction told by the PCRWR was right, the respondents told. The rainfall component in particular helps us in planning our irrigation. IAS particularly helped for groundwater irrigated areas where farmers have control on irrigation water supplies.

In response to the questions that how much IAS helped in terms of water saving or yield improvement, the farmers mostly told that they are unable to quantify the results. Water saving is estimates are possible if we could provide some gadgets of water measurements simultaneously to installed at our farms as well as on the adjoining non-beneficiary farmers' fields. However, some of the farmers quantified it to be in the range of 15-20%.

The findings of the impact analysis survey have been bifurcated zone wise and are presented below:

### (i) Rice - Wheat Zone

The zone lies in the north-eastern part of Punjab and is famous for producing the finest quality rice of the world called *Basmati*. The average annual rainfall in the area is 800 mm, most of which is during the monsoon i.e. the season when the rice nursery is transplanted in the region. The surveyed farmers belonged to the districts Lahore, Gujranwala, Sheikhupura and Sialkot. Although the zone is classified as rice-wheat, but it was found that vegetables, maize, grass, fodder and even moringa is cultivated on commercial basis.

Almost 35-40 farmers were randomly selected and called for the feedback as per the questionnaire, out of which 25 farmers responded while the rest didn't attend the phone. More number farmers were entirely dependent upon groundwater (56%) while the others were having canal supplies as well. The landholding of the farmers greatly varied ranging from 2 acres to 100 acres. It was quite encouraging to know that only 4 farmers out of the 25 interviewed were not making use of the messaging service. The reasons of not using the service was being uneducated or unreliability. The farmers submitted that a number of messages are received from different entities; therefore they don't take it seriously. But from now onward they will try making use of the message as per the guidelines.

Twenty one out of the 25 farmers told that they are receiving messages regularly on weekly basis. They understand the message and are making use of it for irrigation scheduling. Particularly the rainfall prediction component of the message was appreciated by the farmers. Due the impacts of climate change, this season the monsoon started too early and caused torrential rains in the zone during the mid-June instead of July. As the rainfall forecast has informed the farmers about the upcoming rainfalls they planned their transplantation of rice nursery accordingly.

The farmers were particularly asked to tell about any increase in yield or reduction in water use because of using the messaging service. All the 21 respondents recorded that they have got better yields with much less water; however they couldn't quantify it. It is due to the reason that a proper record of the applied water is not kept in the rural economies. When the farmers were asked how we can further improve the service, they demanded that moringa and vegetables should also be included. However, it is not deemed appropriate to include these as the service was envisaged to cater the mega crops of the region. A suggestion was to extend rainfall forecast for 15 days in order to be comfortable for planning rice transplantation.

### (ii) Cotton - Wheat Zone

The zone mostly makes up the southern Punjab and Upper Sindh regions. Pakistani cotton was once famous in the world and produced in the zone. The survey was randomly conducted in the districts of Rahim Yar Khan, Multan, Bahawalpur, Bahawalnagar and Rajanpur. There is less rainfall in the region ranging from 250-300 mm in a year. Groundwater is marginal and mostly the crops are dependent upon canal supplies.

Almost 45-50 farmers were called out of which 25 submitted their proper feedback, while the rest either didn't pick the phone or were busy to not share their experience. Four farmers were entirely dependent upon groundwater (16%), while the rest either have canal water or both canal and tube well water. The area is known to have larger landholding although the beneficiary farmers owned land ranging from 1 to 400 acres but the average landholding came out to be 36 acre/farmer.

Only 3 farmers (12%) out of the sampled 25 were not making use of the message, while the rest were not only receiving the messages on weekly basis but also making use of them. In answer to the query that did you face any difficulty or issue in understanding or making use of the message, the farmers replied it is not something complicated that we cannot understand. The underlying reason of this response is that beneficiary farmers have got training of using similar messages for other services like fertilizers and pesticides. The 3 farmers who didn't made use of the message were those, who were uneducated or didn't take it seriously due to ongoing fake messaging services for the promotion of different items.

The respondents submitted their feedback that the canal supplies are only for 6 months (monsoon period), so they are forced to use marginal quality groundwater during the canal closure period, which have detrimental effects on their crops and soil health. The service is particularly useful during the canal closure period, which helps us in rationalizing the irrigation, and helping them save 15-20% water and 30-40% reduction in their fuel costs.

The zone lies in the low rainfall areas of Pakistan; therefore rainfall prediction in the message was of particular interest for the farmers. The respondents submitted that during the monsoon months, they traded off their canal water supplies considering the rainfall forecast. Particularly, during the month of June, when the farmers of the region were told that rainfall is expected it was quite pleasing for them as the crops' water requirements were on peak with no or limited canal supplies.

In response to the queries that how the messaging service can be improved, the farmers told that fertilizer and pesticide application advisory may be included, as maximum use of pesticides is for cotton. It was further submitted that though they are making use of the service according to their understanding, however further training can improve its

utility. Moreover, it has become common to get windstorms in the region so if it is also informed that a windstorm is coming, we will be better prepared for that.

### (iii) Mixed Zone

The survey was conducted in the districts Faisalabad, T.T. Singh and Sargodha mostly. The zone lies in central Punjab, where both canal and groundwater of relatively good quality is available for irrigation. The zone has diverse crops like sugarcane, rice, orchards, wheat and vegetables. The area is particularly renowned for producing vegetables and fruits like oranges, guava and lemon etc. The average rainfall in the area is 300-500 mm/year.

A total of 32 farmers were called to submit their feedback on the IAS, out of which 25 properly responded. Most of the farmers were having canal and tubewell water supplies, while only 4 (16%) were entirely dependent upon groundwater. The average landholding of the respondents came out to be 17.5 acres, though a lot of variations were there. Only 4 farmers were not using the service being uneducated or believing it to be unreliable.

The 21 farmers (84%) who submitted to make use of the message told that they continuously receive the message in every week. In response to the question how they make use the message in irrigation. The response was different for exclusive groundwater and groundwater-canal water areas. The groundwater dependent farmers submitted that they apply irrigation on weekly basis, therefore they have tuned up to apply irrigation on Saturday morning (46%) after receiving PCRWR message. It is quite simple to shut of the tubewell as soon the fields get the requisite depth of irrigation as guided by PCRWR.

For canal water-groundwater areas, the farmers said that canal water was supposedly making only a fraction of their irrigation requirements as per their perceptions. But now they have come to know that it is almost equal to the crop water requirements so their dependency on groundwater has substantially reduced. A few times it is needed to operate the tubewell to augment the irrigation supplies. We have observed improvement in crops yield by less applying the groundwater, which is of marginal quality as we know.

Rainfall forecast component of the message did impacted farmers of this zone also, although its effect was pronounced for the rice-wheat zone. The farmers (70% users) submitted that they have skipped some irrigation in the wake of rainfall forecast, which not only helped them in saving irrigations costs but protected their crops from lodging. Since the beneficiaries are getting advisories of the major crops, while the vegetables and fruit orchards were making a major component of their farm productivity so they requested to include the advisory for fruit plants in particular. Some farmers (40%) expressed their desire to get more knowhow of the drip irrigation system for their orchards.

### (iv) Crop - Orchard Zone

The northern and southern parts of the country come under this zone. As orchards are becoming an important component of their maize-wheat rotation in Peshawar (KP province) and banana for the cotton-wheat rotation in Hyderabad (Sindh province). The surveyed farmers belonged to the same districts. The rainfall in Peshawar is high but the Hyderabad area get too little rainfall like 200 mm.

Almost 45-50 farmers were contacted out of which 25 responded. Only 3 farmers (12%) were not using the service. The rest (22 Nos.) submitted that they are not only using the service but also making use of it. Three farmers were dependent upon rainfall and spring

water, so they were using the rainfall forecast component only. Four farmers were exclusively dependent upon groundwater so they submitted that message is very useful for them being in control of cutting the irrigation supplies on replenishment of the depleted moisture.

The major portion of the agriculture income in this zone comes from orchards, plum, peach, pear, tomato, and apricot etc. in Peshawar and banana in Hyderabad. Particularly, the farmers of the Peshawar area asked to include the advisory for the orchard crops. Almost 70% farmers of Peshawar are emphasized that messages should be in Pashto (regional language in KP province).

## IAS Impact Analysis – Distinct Stories

- Mr. Hussain Abad Khan, Jalalpur, District Multan, a progressive farmer owning 400 acres of land shared that he read the story of PCRWR IAS published on VoA online. I was not a beneficiary of the service, but when I came to know, how much it is beneficial for the others, I found out the PCRWR number from web and called them to include my number in the beneficiary list of the Multan district since 11th August, 2017). Being a learned farmer I knew the importance of the information provided by PCRWR as it can substantially reduce my irrigation costs, which makes up a huge chunk, almost 60%, of the total input costs of my crops.

- Mr. Nazir Ahmad Choudhary, District Faisalabad, a progressive farmer of 14 acres is also an Ex-Extension officer. He told that he is a beneficiary of the PCRWR IAS and a member of the local *Kisan Ittehad* (farmers' union group). Convinced on the efficacy of the message in water saving and yield improvement, he is not only using the messages for rationalizing his own irrigation but also explains it to the other beneficiary farmers during the group meetings.

- Muhammad Ashraf, Hayatabad, Sargodha, recorded his feedback on 11th May 2017 as, "*I had grown wheat on my 12 acres land this season and continuously received irrigation advisory messages from PCRWR system," he said, speaking in his native Urdu. "Keeping in view the advised water consumption and rainfall forecast, I only applied three irrigations, whereas my neighboring farmers applied six to seven irrigations. I have recently harvested my crop and got 48 maunds/acre [4,742 kg/ha] yields, whereas my neighbors could get 42 maunds/acre [4,149 kg/ha]. "I am thankful to PCRWR for their advice, which not only let me get better yields but the irrigation cost was substantially reduced*."

- Mr. Sajjad Hussain, District Gujranwala, an agronomist and owner of 90 acres farm. He shared that he is properly using the IAS message and have got convinced that he is saving at least 40% water. A grower of crops like sugarcane, wheat, rice and fodder, he shared that he cultivates the said crops mostly beds or ridges. He offered his services to arrange a field day at his farm where he would demonstrate to the farmers that how he is using the message information for application of water to the crops.

- Haji Nazeer Ahmad Qazi, Makhdoom, District Sargodha recorded feedback on 28th March, 2017 as, "*I had grown wheat on 10 acres. I received PCRWR message that 0.5 inch rainfall is expected in the upcoming week. Keeping in view of the expected rainfall and last week water consumption I skipped my last irrigation for wheat and used the same for leguminous crop. The rainfall forecast proved right, which not only saved me irrigation but protected wheat from lodging as his nearby farmers suffered*". Mohammad Tariq, a beneficiary from Faisalabad says, "*I think the information they send is quite useful for us as by conserving water, our profit margins will be greater*".

Some of the success stories shared by the beneficiaries during their random feedback were recorded by PCRWR and subsequently published in some leading magazines of the world like EoS of the American Geophysical Union, Voice of America, thirdpole.net and Dawn News.

Like there farmers there are hundreds of farmers, who are making use of the service. The service is helping them to determine rational irrigation needs of their crops, which allow them to plan and apply irrigation accordingly. It is estimated that the service would be saving 250 MCM of irrigation water in a year.

This work is in fact beneficiaries oriented, as the service is directly facilitating the farming community. However, the utility of the service is being improved by more follow up calls and feedback, as how actually the service is benefiting the farmers and how further it can be improved.

## Conclusions and Recommendations

- This work is an end user oriented and is facilitating the farming community in rationalizing irrigation practices.
- Rainfall forecast component of the message is most popular among the farmers.
- Water saving recorded by the farmers after using IAS is 15-20%.
- After rational irrigation the farmers' yields are improving to the tune of 10-15%.
- Proper training of the farmers may increase utility of the service substantially.
- The service may be extended to 100,000 farmers, with some targeted refinements.
- If extended to larger number of beneficiaries, the service can save substantial amount of water.

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**Paper No. 782**

# FARMER MANAGED IRRIGATION SYSTEM AND ITS IMPACT ON WATER SERVICE DELIVERY: A CASE OF SINDH-PAKISTAN

**Nazir Ahmed Memon & Masroor Ahmed Shahwani**

# FARMER MANAGED IRRIGATION SYSTEM AND ITS IMPACT ON WATER SERVICE DELIVERY: A CASE OF SINDH-PAKISTAN

By:

**Nazir Ahmed Memon[1], Masroor Ahmed Shahwani[2]**

## Abstract

Irrigation system of Pakistan is the largest integrated irrigation network in the world. The system is fed by the waters of Indus River and its tributaries. The Sindh province has 13 million acres of irrigated lands in its three barrage command areas, built between 1932 and 1962. In 1997 the Government of Sindh started the reform of the management of the entire irrigation system. The Sindh Assembly approved an Act, shifting the responsibilities for the management of the irrigation and drainage infrastructure from the centralised Provincial Irrigation Department (PID) to the farmer managed water institutions known as Sindh Irrigation and Drainage Authority (SIDA), Area Water Boards (AWB) and Farmers' Organisations (FOs). The institutional structure proposed under SIDA Act was restructured through Sindh Water management Ordinance in 2002 with an objective to make the institutions more participatory and autonomous in its decision making and service delivery.

The key process of Institutional Reforms has been carried out by implementing Participatory Irrigation Management (PIM) which has evolved and become generally accepted as a necessary aspect of productive and sustainable irrigation. The concern of this paper is to review the reform process and assess its role, impact especially in improving water service delivery under command of farmer managed system. The paper will also focus on the current reform, reform process, working of SIDA, FOs and AWBs and their strategy to address the issues and problems of farmers with respect to water distribution and governance.

**Keywords:** Irrigation, water service, equity, reform, governance

## Introduction

Agriculture sector in Pakistan contributes 21% to GDP, employs 45 % of the labor force and provide 60% of the total exports. Due to predominantly arid and semi-arid climate, 90% of the cultivated area in the country is irrigated. The major source of irrigation is the Indus Basin Irrigation System- the world's largest integrated irrigation system. Pakistan's major surface water resources are mainly derived from the rivers of the Indus basin.

Sindh has three major barrages on the Indus River that divert approximately 45 MAF to 48 MAF of water annually to the 14 main canal commands in Sindh. These canal systems have an aggregate length of 11,916 miles (19,066 km) of canals, which serve a gross command area (GCA) of 14.391 million acres. There are about 42,000 watercourses (tertiary channels), which have an aggregate length of about 75,000 miles (120,000 km).

Majority of the population of Sindh province is directly and indirectly relay on agricultural and entire province is heavily dependent on public services with little role for private sector. But the service delivery of the line departments was continuously on the downward slide mainly due to institutional weaknesses. The major institutional issues are centralized character of government institutions, lack of community and stakeholder participation, lack of corporate skills and infrastructure deficiencies in the delivery network, such as low irrigation delivery and application

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efficiencies, water distribution inequities, waste full use of water logging and salinity along with other factors like poor access to credit, skewed land holding, low social capital and disadvantageous role for rural women.

The Govt. of Sindh established Sindh Irrigation and Drainage Authority (SIDA) under SIDA Act 1997 for promotion of irrigation reforms, participatory irrigation management and improvement of irrigation and drainage network in the province. Currently, SIDA is working under Sindh Water Management Ordinance (SWMO)-2002. Sindh water sector improvement project was initiated with the assistance of world bank to augment the efforts of government in improved water services and rehabilitation of channels through participation of farmers and their management. Accordingly base line was also developed at the initial stage of the project.

So far 360 Farmers Organizations (FOs) have been formed in the command of Nara Canal Area Water Board (NCAWB), Ghotki Feeder Canal Area Water Board (GFCAWB), Left Bank Canal Area Water Board (LBCAWB) and outside Area Water Boards (AWBs) in Sindh Pakistan. The management of these channels have been transferred to FOs under Irrigation and Drainage Management Transfer (IDMT) agreement enabling farmers to manage the system at their own through structured manner of farmer organizations and water users associations. To a large extent, the case study pointed to the fact that the farmer managed irrigation system or participatory irrigation management (PIM) programme had led to better water management, improved service delivery, and consequent direct and indirect benefits to the farm households.

The objective of the present paper is to provide an overview of impact of farmers managed system on irrigation delivery performance.

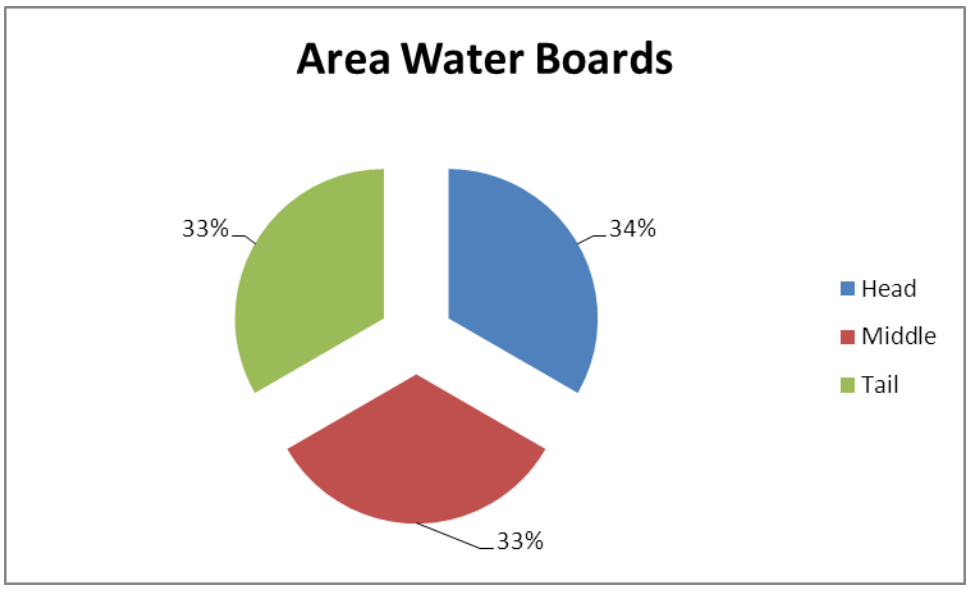
## Material & Method

This study is based mainly on the primary and secondary data collection, analysis and selection of sample FOs from each Area Water Board. Most of the data including baseline has been taken from the reports generated under Sindh water sector improvement project. The sample FOs has been from the FOs numbering more than 300. To give better understanding and comparison sample was also taken from controlled area (without project) to address the key parameters i.e. efficiency, reliability, equity, sustainability and user's satisfaction. To select adequate and representative sample, a multi-stage, stratified and quota sampling technique was adopted. The methodology for selection included a mix of surveys of suggested sites, consultations with different stakeholders. Adequate care was taken to ensure that the selected FOs/WUAs represented the head and tail reaches. The primary data has been collected through structured questionnaire from Board of Management members of farmers managed channels.

The sample size for FOs/irrigation channel was taken as 30 for the impact assessment to improve its AWB and location wise representation and its distribution is shown in the following Table-1 Meanwhile, 6 irrigation channels were taken for control purpose i.e. to be able to attribute the improvements to the project interventions "Without Project".

***Table-1:*** Sample FOs/Irrigation Channel

| Area | Location | | | Overall |
|---|---|---|---|---|
| | Head | Middle | Tail | |
| Area Water Boards | 10 | 10 | 10 | 30 |
| Control | 2 | 01 | 03 | 03 |
| Total | 12 | 11 | 13 | 36 |




## Result & Discussion

Though the participatory irrigation management in Sindh province of Pakistan has been introduced not very recently, but its contribution in improved water management is positive. There have been slow growth but results are encouraging on the parameters as indicated below:

***Efficiency:*** The conveyance efficiency considers efficiency of canal, distributary, minor and watercourse. During baseline the efficiency of irrigation system was estimated through water user's interviews that figured out efficiency of distributaries/minors 83%, watercourses 80% and farms 78%. This resulted in an overall conveyance efficiency of 66% for canals, distributaries, minors and watercourses. Later, the conveyance efficiencies were estimated through physical losses measurements of canals, distributaries, minors and watercourses. The comparison revealed for conveyance efficiency of the rehabilitated channels revealed improvement of 5 % as it increased to 71 % at current from the baseline value of 66 % and met the anticipated target.

***Reliability:*** The reliability of irrigation supplies were estimated as 39 % from the farmer's perception at baseline and it was further calculated from the canal discharges which revealed that the adequate irrigation water was supplied for 42 % of the days as per indent. Now, the post-project evaluation of canal discharges revealed that adequate irrigation water was supplied for 53 % days which figured out substantial increase of 11 % days.

***Equity of water deliveries:*** The delivery performance ratio (DPRs) estimated through physical measurements of Watercourse discharges prior to rehabilitation that ranked 57 % watercourses in poor equity category. The post-project DPRs analysis has revealed that the watercourses in poor equity have reduced to 45 % and a remarkable reduction of 13 % occurred due to the project implementation.

***Sustainability:*** Sustainability depends on the improvement in collection of water service fee (*abiana*) and available budget for operation and maintenance (O&M) of the irrigation and drainage system. For the sustainability of whole system financial viability of FOs is imperative. Presently, the water resources bodies like SIDA, AWBs, and FOs are not that much effective to guarantee overall sustainability of the improved I&D system. The water service fee is very low

and the FOs cannot meet the O&M expenses of the irrigation system. The change management process takes time to completely shift from centralized management to farmers and customer oriented system. In Sindh its taking place with slow pace.

***User's Satisfaction:*** Users' satisfaction level is one of the key indicators of service and some improvements have been observed in users' satisfaction level in comparison to the pre-project conditions. Under present impact assessment, the users' satisfaction level has been measured through using 7 indicators in both project and non-project areas. These indicators are: (i) supply of water on time, (ii) supply of sufficient water, (iii) equitable distribution of water, (iv) condition of channels, (v) maintenance of channels, (vi) abiana collection, and (vii) equal treatment to all farmers. At the end, an index that may be called as satisfaction index has been developed to measure the users' satisfaction level. The sample farmers were asked to mark each indicator from 1 (poor) to 5 (good). Finally, by taking average of scores the users' satisfaction level was evaluated. The average total satisfaction level of users was 2.89 out of 5, this score increased to 3.44 as by the end of 2014. In the control group which did not get benefit from the rehabilitation works, this score was calculated as 2.65. The scores of the project area are above 3.50 in all individual indicators except for "abiana assessment and collection practices" that is the main and the most problematic issue in the project area. Except for this indicator, some improvements have been realized for other items in comparison to the base line conditions in terms of user satisfaction level.

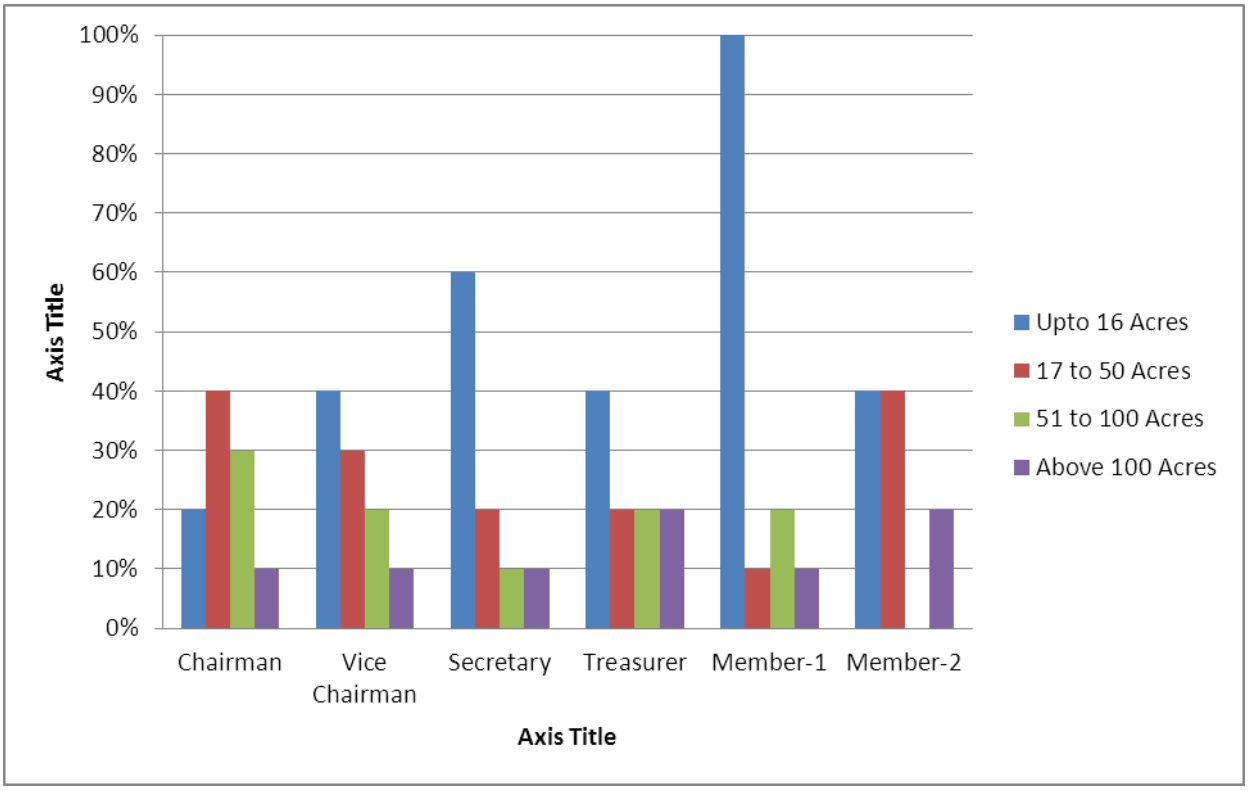
The awareness level of the respondents was also checked through face to face interviews regarding on-going institutional reforms in the irrigation sector. In the project area the awareness level seems to be increased from 70 % to 78 % in comparison to the baseline conditions as a result of social mobilization process during the project implementation. The impacts of social mobilization conducted during the project implementation period have also been observed when awareness levels about FOs in project area and control group in which no rehabilitation have been carried out; among the control group households, the awareness level about FOs is only 56%. This difference may be the impacts of SIDA's social mobilization efforts during the implementation of WSIP-I. On the other hand, the fact that 22 % of sample households are not aware of FOs in their areas indicates further need for social mobilization activity on the ground.

The farmers were asked about the impacts of FO formation on equitable distribution of water. The question was asked in the project and non –project areas who were aware of the FOs in their areas. According to the findings, 9% of the respondents were found to be highly satisfied with FO services in water distribution, 78% of respondents reported their satisfaction and only 13% percent were not satisfied with the FO's services in distribution of water. In the control group, the percentage of those not satisfied with the FO's services increased to 40%.

Analysis was also made on the leadership (Board of management-BoM) of FOs. The BoM is consisting of six members of farmers which run the show and undertake day to day management tasks. It was myth that large land owners or feudal have captured the system and poor farmers or small farmers have not been part of this. Sindh historically is the area where land hold holdings are bigger than other areas. The table-2 gives some features of landholding of members of FO management.

***Table-2:*** Land holding of FO BoM members

| S# | Land Holding of Chairman | Percentage of Office Bearers | | | | | |
|---|---|---|---|---|---|---|---|
| | | Chairman | Vice Chairman | Secretary | Treasurer | Member-1 | Member-2 |
| 1 | Upto 16 Acres | 20% | 40% | 60% | 40% | 6 | 40% |
| 2 | 17 to 50 Acres | 40% | 30% | 20% | 20% | 10% | 40% |
| 3 | 51 to 100 Acres | 30% | 20% | 10% | 20% | 20% | 0 |
| 4 | Above 100 Acres | 10% | 10% | 10% | 20% | 10% | 20% |




## Conclusion

The main conclusion of study and sample surveys and interviews have indicated that institutional reforms in irrigation and farmer managed system is worth to opt and beneficial for not only farmers but overall economy of Sindh. The state may get benefit from such interventions by accelerating reform process and transferring irrigation system to farmers. Presently, FOs/WCAs have emerged as ongoing functional identities meeting most of the "necessary condition" of institutional reform. Nevertheless, any FO/WCA will have to qualify the final stage of "sufficient conditions" which as, having organizational adaptability to local, physical, social and cultural environment; goal attainment, integration of beneficiaries' vision, attitude, value system; and knowledge for keeping independent identity at the community level.

The improved water service delivery covering areas of efficiency, reliability, equity, users' satisfaction, etc would certainly lead to increased yield and improved living. However, on broader terms there is a need to undertake study with regard to poverty reduction under farmers managed system.

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**Paper No. 783**

# CROP WATER BUDGET ESTIMATION USING GIS

**Muhammad Awais, Muhammad Mohsin Munir**

# CROP WATER BUDGET ESTIMATION USING GIS

By

**Muhammad Awais[1] Muhammad Mohsin Munir[2]**

## ABSTRACT

Pakistan lies on the verge of world largest aquifer but on papers we are defined as water stressed country and crop stressed region. Institutional and individual attitude and practices in consuming water for domestic and agriculture needs can be blamed as reason behind this water stressed condition. To calculate this crop stressing, a sample area was selected in Sadiqia Canal, Hakra Branch for studying crop water requirement, its deficit and the actual water supplied by irrigation system and tube wells. It was calculated that the command area was crop stressed due to shortage of water and less tube wells were used to overcome the deficit. Based on preliminary study it is recommended that the distributary should be lined to decrease the losses hence decreasing water deficit. Further it is recommended that crop water reporting must be done for the irrigation areas for calculation of actual crop water requirement for best efficiency of irrigation system.

**Keywords**: Water Balance, Irrigation, Crop Water Requirement, Crop Deficit, Blaney Criddle, GIS, Satellite Imagery

## 1. INTRODUCTION

As the population of our country is on an increasing trend, the country is threatened to face an acute and persistent water crisis in the upcoming years due to improper management of water resources. Being an agrarian economy, we need to consider and address the water issue at both individual and institutional level for the sake of stable economy. Poor water resource management, system losses, outdated irrigation practices, unregulated domestic water use and ground water pumpage, political influences, water thefts, absence of rule of law are among the factors which can be held responsible for water stressed conditions. Keeping in view the above-mentioned conditions, crop water budget estimation is the way to estimate the water requirement of any specific crop throughout the crop season and allow us to regulate irrigation system in accordance to cultivated crop and cropping intensity. Hence, it will improve system efficiency and enable us to regulate the ground water pumping.

Agriculture is basis of Pakistan's economy. It accounts for 21% of the GDP and together with agro-based products fetches 80% of the country's total export earnings. More than 48% of the labour force is engaged in this sector.

The Punjab province has about 29% of the total reported, 57% of the total cultivated and 69% of the total cropped area of Pakistan. It contributes a major share in the agricultural economy of the country by providing about 83% of cotton, 80% of wheat, 97% fine aromatic rice, 63% of sugarcane and 51% of maize to the national food production. Among fruits, mango accounts for 66%, citrus more than 95%, guava 82% and dates 34% of total national production of these fruits.

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The mission is to sustain food security and support to national economy, making agriculture cost effective and knowledge based, with emphasis on farmer's welfare and maintenance of the yield potentials.

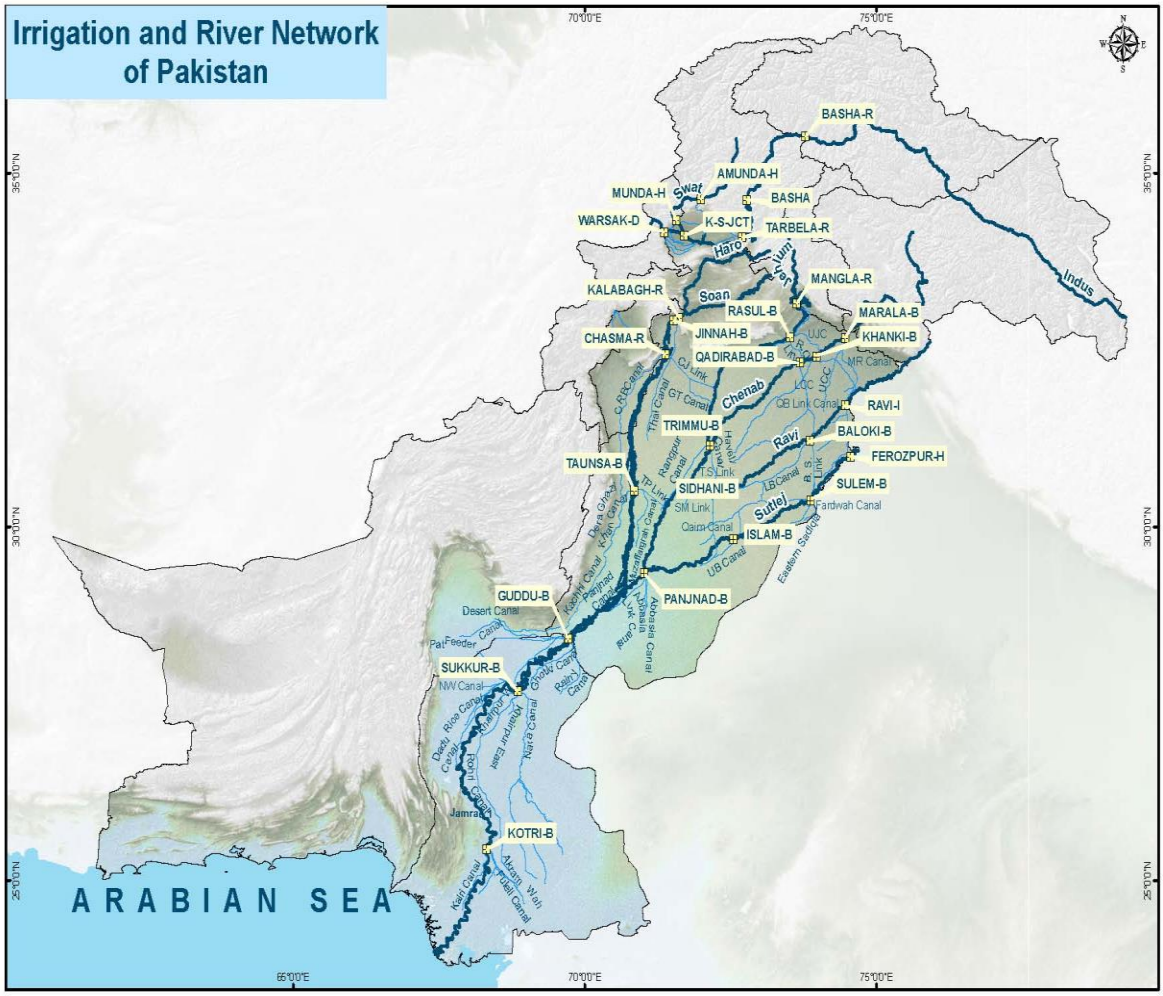



***Figure 1:*** Irrigation Network of Pakistan

The project has various on and off field applications. Crop Mapping using GIS and drones can be used in field by irrigation department for crop reporting and water course surveillance. Accurate and readily available data enable us to evaluate any on field scenarios with respect to time. The other major objectives of this project are as follows

- Computation of Crop Water Requirement and its Importance for crop yield
- Ground water Utilization and Depletion of water table
- Importance of Water Balance and Selection of appropriate Irrigation Scheme with respect to crop cultivated.
- Importance of drainage in irrigation System.
- Effect of Water Logging and Salinity on Crop yield.
- Crop Intensity and its Effect on Crop Yield.

## 1.1 PROBLEM STATEMENT

The water requirement of command area of 3-L distributary of Hakra Branch needs to be computed, hence comparing it with the actual water supplied to command area and recommending feasible solutions for the crop stressed and water stressed regions.

## 2. LITERATURE REVIEW

Irrigation is the measured application of water to meet crop water requirements not satisfied by rainfall. Careful and efficient irrigation is becoming increasingly important throughout the world and such irrigation can improve crop yields and quality, optimize water use, and protect natural resources. Many farmers determine how much water to apply and how often to apply it through a water budgeting approach. Water budgeting strives to balance water use with water inputs. This method is similar to balancing a checkbook. Water inputs may include rainfall, irrigation, and capillary rise from ground water. The outputs may include evaporation, transpiration (evaporation through plant tissues, most often leaves), runoff, and percolation below the root zone.

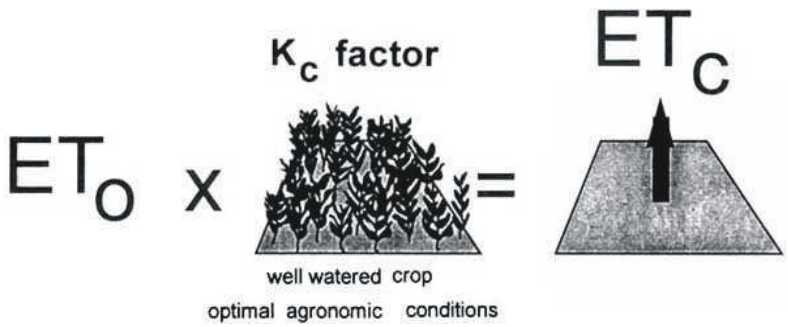



***Figure 2:*** Actual crop evapotranspiration (ETc)

(Janelle Montgomery, 2013) says that Water Use Efficiency (WUE)/Crop Water Budgeting is a concept that has historically caused much misunderstanding for natural scientists, water suppliers and end users alike. Much of this confusion has stemmed from the range of terms available to describe water use efficiency and a lack of understanding of what WUE represents.

Water Use Efficiency is itself a generic label that encompasses an array of performance indicators used to describe water use within a cropping system.

The performance indicators can be categorised as one of the following:

- Water Use Index (WUI) (relating production to water use)
- Irrigation System Efficiency (relating water inputs to water outputs at different locations)

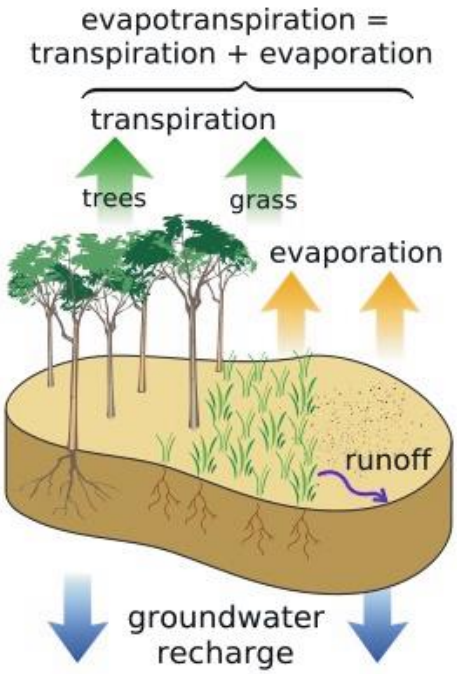



***Figure 3:*** Mechanism of Evapotranspiration

- Distribution Uniformity (DU) (a measure of how even an irrigation application is).

(Janelle Montgomery, 2013)

## 2.1 CURRENT RESEARCH ON CROP WATER BALANCE AND IMPLICATIONS FOR THE FUTURE

The most important single factor governing agriculture production in the world is insufficient supply of water. The magnitude and dynamics of different components of crop water balance is crucial for development of technological options for sustainable management of soil and water resources. In addition to rainfall, detailed and practical knowledge is required of seasonal values of evaporation, weed and crop transpiration, runoff, deep drainage and soil water storage. The objective of soil and crop management is to increase crop transpiration, plant biomass, and the harvest index. Background resource on climatic data, water balance and soil hydrological characteristics for the region is grossly inadequate.

Major problems with available weather data include: (a) incomplete, insufficient and discontinuous data; (b) unsatisfactory data obtained by non-standard methods; and (c) data of low scientific credibility. A few lysimetric studies have been conducted to evaluate crop water use and water use efficiency. The water use efficiency depends on time of sowing, tillage methods, residue management, and fertilizer application. Information on rooting characteristics and soil water extraction patterns is not known for any of the major crops. A few experiments conducted to evaluate runoff have been done either on small plots or using a rainfall simulator. Little information exists on soil physical and hydrological properties measured *in situ.* (LAL, 1991)

## 2.2 PROJECTIONS OF CROP WATER REQUIREMENT IN PAKISTAN UNDER GLOBAL WARMING

Globally Climate change is becoming a challenging threat to humanity. The anthropogenic activities are the driving causes besides others. Its impacts on various sectors, especially the agriculture, are more pronounced. Due to fact that the weather/climate has no political boundaries, the developed industrial nations are polluting their regional environment and ecology. In this way the under developed and poor nations, like Pakistan, have to suffer severely. Besides this the availability and reservation of water resources are also a giant challenge in future, on international as well on regional level. Also, due to political and strategic relationship to the neighbouring countries, water availability would be a big problem for the country in near future. Future projections for climate are showing increasing trend in surface temperatures, which will ultimately augment some agrometeorological elements like reference crop evapotranspiration (ETo) etc. The current study is carried out to analyse the subsequent effects of increasing temperatures on ETo and on the agriculture water demand in the country. In the light of climate projections from various authentic sources like IPCC, increasing trends of 1-3°C have been studied which is likely in next 50 years. It has been concluded that the southern half of Pakistan may be the least affected areas in the time to come. However, in northern half, a sharp increase in water demand as compared to the present will appear leading to the enhanced risk of crop failure in rain fed areas

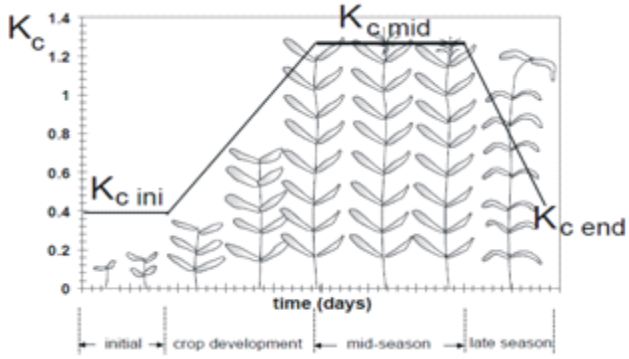



***Figure 4:*** Crop Co-eefficient Curve (FAO)

where supplementary irrigation is not available. This study will help in crop monitoring and in the assessment, that how much water available in future for crops; which type of crops would suit the climate, Better management and building of new water reservoirs may help to cope the situation for an improved agriculture growth. (Ghazala Naheed, 2016)

## 2.3 IRRIGATION SYSTEM OF PUNJAB

### 2.3.1 OVERVIEW

An overall overview of complete irrigation network is shown in **Figure 5**

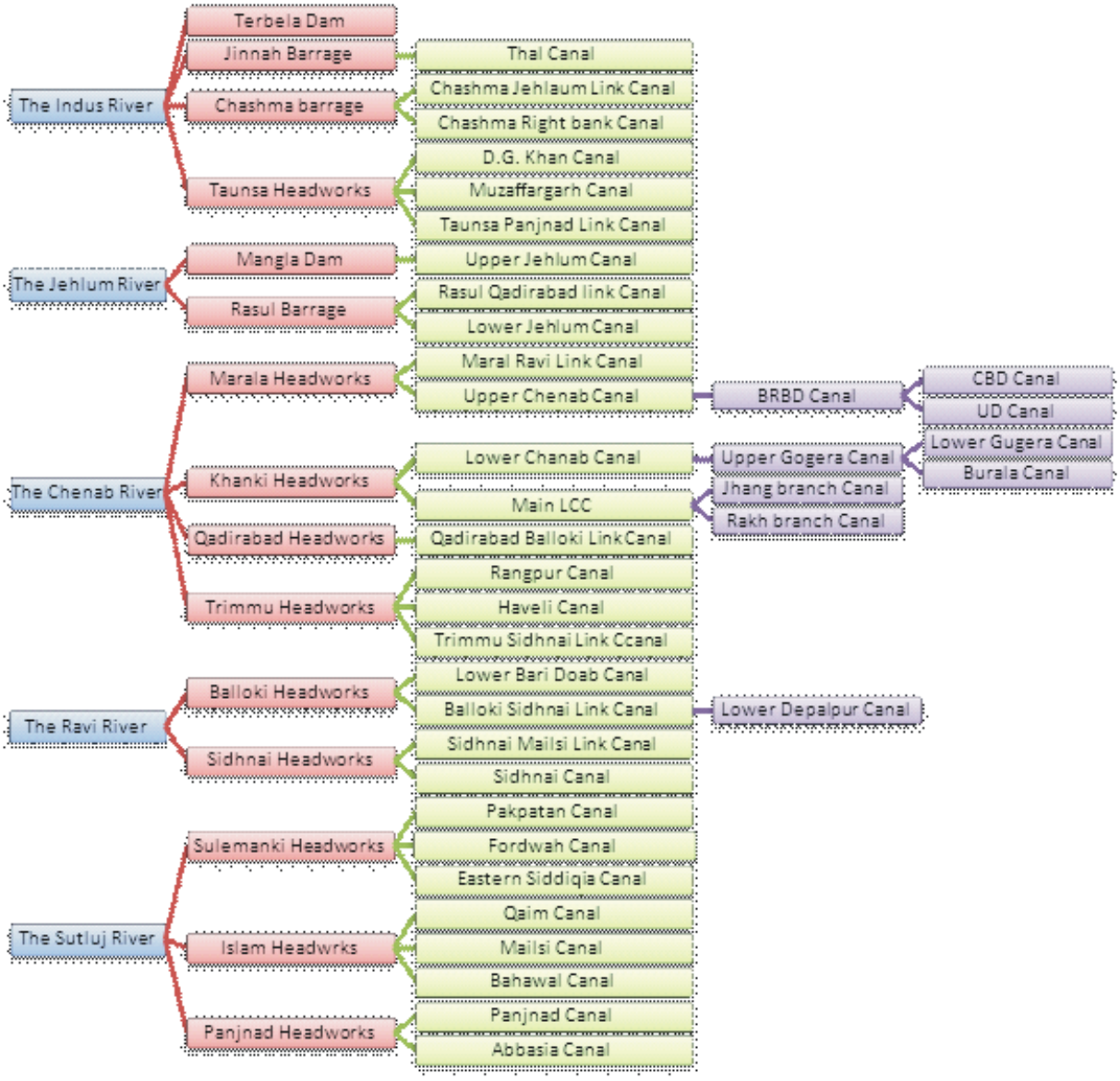



***Figure 5:*** Irrigation Network Layout of Punjab

## 2..3.2 SOURCES OF IRRIGATION

| | Pakistan (Area in million hectares) | Punjab (Area in million hectares) |
|---|---|---|
| Canals | 6.36 | 3.35 |
| Tube wells | 3.79 | 2.94 |
| Wells | 0.31 | 0.20 |
| Canal tube wells | 7.60 | 7.60 |
| Canal wells | 0.25 | 0.25 |
| Other sources | 0.31 | 0.09 |
| Total irrigated area | 18.63 | 14.41 |

(Agricultural Statistics of Pakistan 2011-12) (Punjab, n.d.)

## 2.4.3 GROUNDWATER PUMPAGE

The Indus Plain is a vast gently sloping plain extending from the foot of the Himalayan mountainous to the sea. The plain is underlain by deep deposits of unconsolidated and highly permeable alluvium consisting primarily of fine to medium sand, silt and clay. Fine-grained deposits of low permeability are generally discontinuous so that the sands and silts serve as a unified highly permeable aquifer. The thicknesses of the alluvium have been found to be in excess of 900 m (3,000 ft) in Punjab and upper Sindh, but as little as 90 m (300 ft) in lower Sindh.

Due to seepage from the vast irrigation system that has been constructed in the Indus Plain, besides rainfall and inflow from the rivers flowing through the plain, the alluvium is filled with water almost up to the ground surface, thereby creating a vast reservoir of groundwater

Tube wells are the major source, after canal water, to irrigate the cultivated land. This unregulated water pumpage is not only lowering the water table but also creating water stressed situation for Pakistan. Let us consider few details about groundwater and tube wells in Pakistan and Punjab respectively.

The below mentioned trends are showing data for 2008-09 but currently the number of tube wells has been increased to 1 Million.

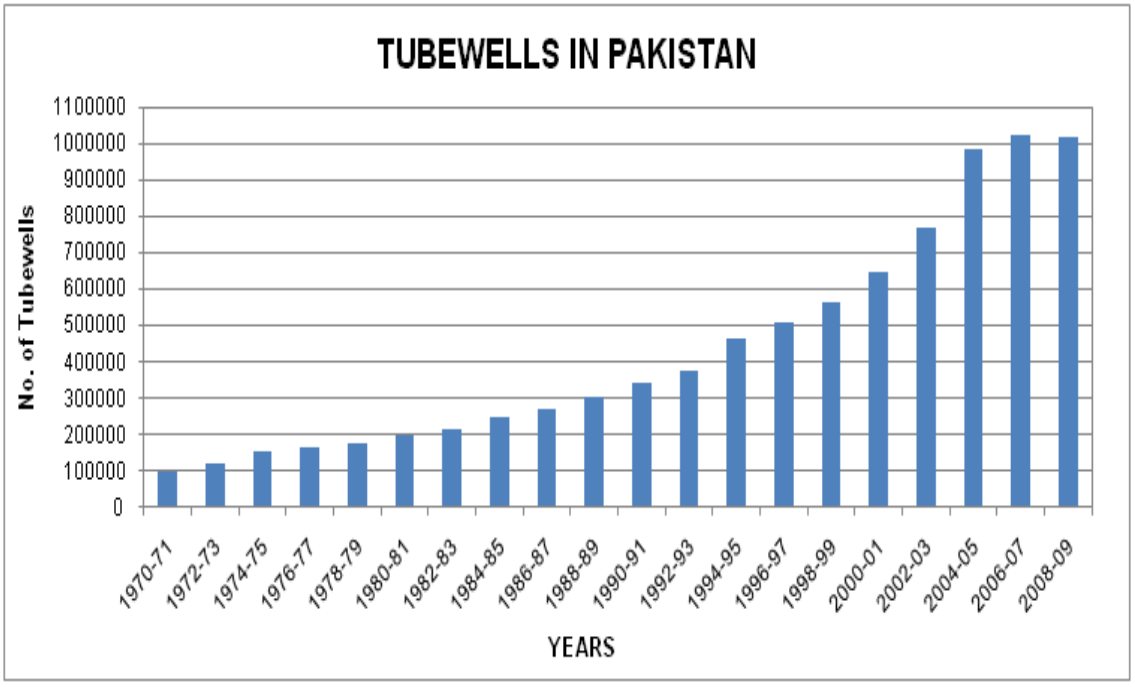



***Figure 6:*** Trends of Tube well in Pakistan

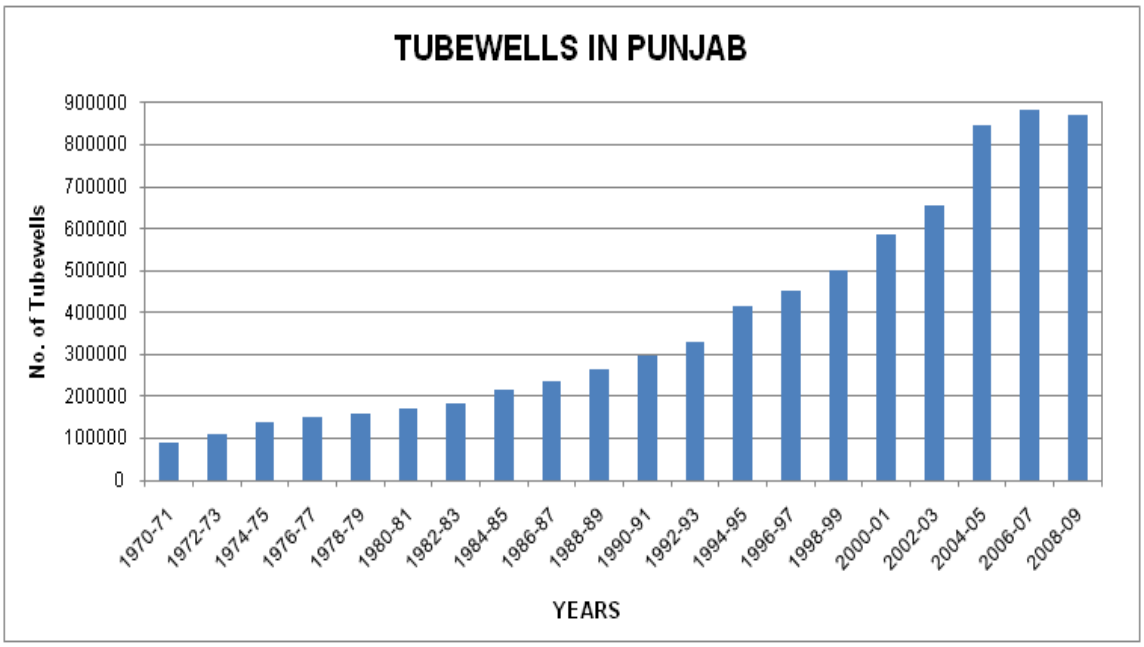



***Figure 7:*** Trends of Tube wells in Punjab (PMPIU)

## 2.4.4 WATER QUALITY IN PAKISTAN

Each year when the river is high the land is flooded and drains off again when the floods have passed. Evaporation occurs from the west soils and salt is deposited in them. When the next flood arrives, the salts are re-dissolved and pass down to join the ground water. Since the ground water is slowly flowing down the valley, it picks up more and more salts as it advances. In addition, the ground water reacts with aquifer material and picks up salts from this source also. It would therefore be expected that from source to outfall, in an alluvial river valley system,

there would be an increase in ground water salinity. This is observed to be the case in the Indus Plain. Super-imposed on this overall ground water pattern are the areas of comparatively fresh water associated with direct seepage from the rivers. Besides this percolation of fresh water occurs from rainfall and also from the irrigation system which keeps on increasing with the spread of the irrigation system. As a result of this process fresh ground water in the Indus Plain is found to be underlain with saline ground water, the layers of fresh ground water being thick near to the rivers and other sources of recharge and of almost negligible thickness in areas where recharge is small. Fresh water is therefore found up to considerable depths in wide belts paralleling the major rivers

### 2.4.5 WATER LOGGING AND SALINITY

Seepage and percolation is resulting in rise in water table at various location causing water logging and salinity issues. This problem not only makes land unfavorable for cultivation but also reduce the crop yield and production. The Area of 3L Disty of Hakra branch is severely affected by water logging and salinity issue. The problem can be easily resolved by incorporating effective drainage system.

## 3. CASE STUDY PROJECT

Crop water budget estimation is done with the help of FAO-24 (Blaney Criddle) of 3L distributary of Hakra branch, Sadiqia Canal, Punjab, Pakistan. GIS, Satellite Imagery were incorporated as major tools for crop mapping for above mentioned project. Excel Sheets were developed in order to compute water requirement and system deficit for a specific crop throughout the crop period. Site visit was conducted in order to verify crop requirement, irrigation system layout, ground water and other field related information. Satellite Imagery with the help of GIS map wasused to identify crops on the basis of physical appearance, data acquisition for water budget estimation and crop reporting.

### 3.1 PROJECT SCOPE AND SIGNIFICANCE

Crop water budget estimation is basically estimating the water requirements of crop by computing the evapotranspiration (ET) and then evaluating it with reference to water available in irrigation system. Standard procedures are implemented along with various techniques and technologies as Satellite imagery and GIS for computations and calculations. Furthermore, drones are used for acquiring imagery for crop classification and mapping. FAO-24 (Blaney Criddle method) is applied for computation of daily ETo and ETc respectively and later on water deficit is computed with respect to available water. Water deficit allowed us to evaluate the various sources which may be utilized by the natives to fulfill this water shortage. The major source is ground water pumpage through tube wells. On the contrary, the reasons of this water deficit are also discussed which come out to be political influence, water thefts and inefficient system.

### 3.2 PROJECT AREA

3L distributary, Hakra Branch of East Sadiqia Canal, Tehsil Haroonabad, District Bahawalnagar was selected as the project area. **(Figure 1).**The Coordinates of the site are measured as Latitude 29’26’58’’ and Longitude 73’9’44 with an elevation of 118 m. The main reason of selection this site is the availability of discharge and weather data. The command area was calculated to be 1787 Acres with an authorized discharge of 9.68 cfs. The Approximate Length of distributary was measured to be 2.58 miles (4.15 Km). A total number of 6 water courses were there to irrigate the command area. The Major Crops cultivated in the region were wheat (Rabi) and Cotton (Kharif) with fodder and some part of vegetables. The ground water is brackish and in the project area crop yield is effected due to this severe water logging and salinity issues.

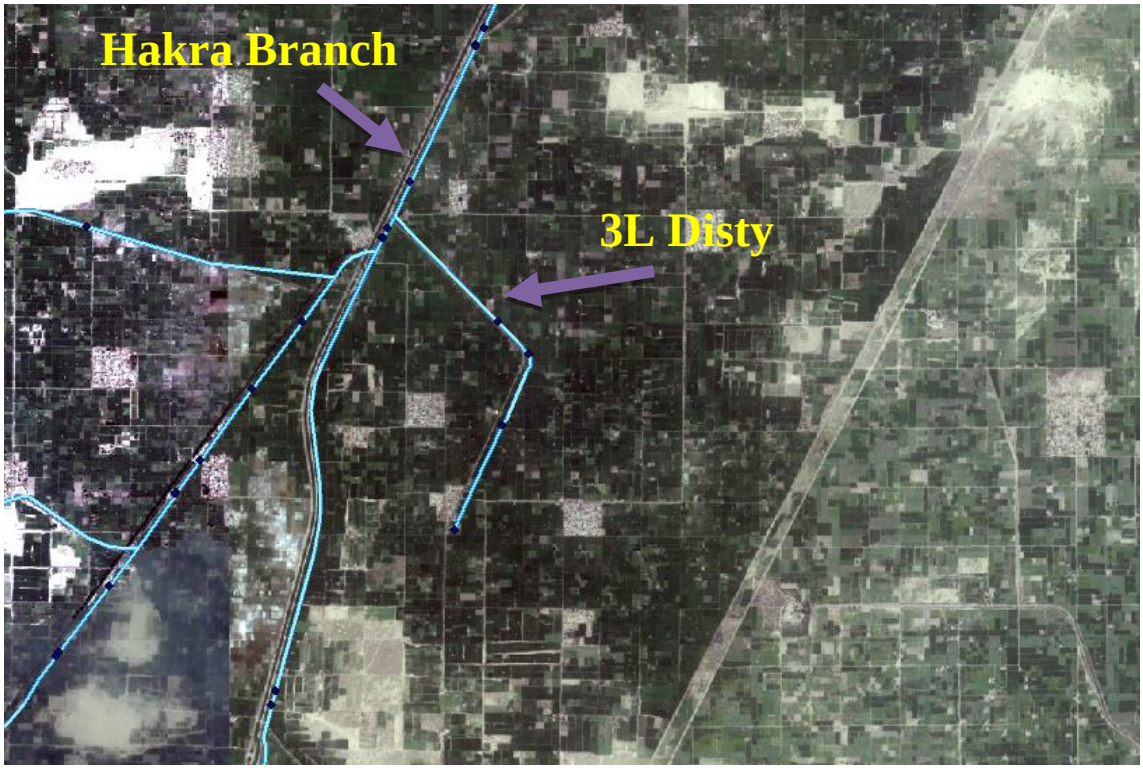
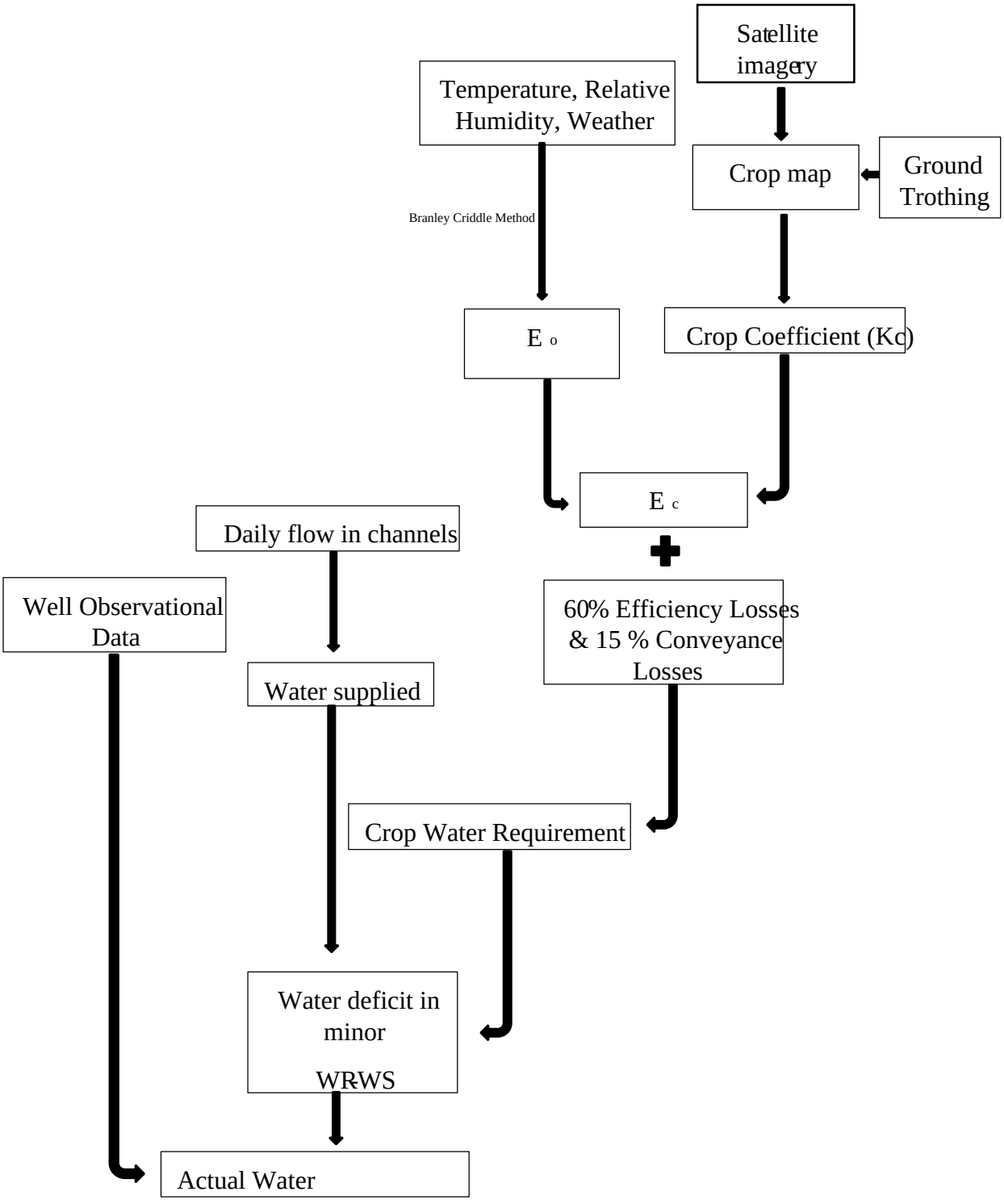



***Figure 8:*** GIS map layout of 3L Distributary, Hakra Branch

## 4. APPROACH AND METHODOLGY

Evapotranspiration is calculated by Blaney Criddle Method for computing crop water requirement and total volume of water deficit is calculated by subtracting the amount of water supplied. As per requirements of our project, a visit was conducted to proposed site Hakra branch Canal situated about 34 kilometres from main Haroonabad city Bahawalnagar at 17th of January, 2016.

The project flow and approach is discussed in flow chart for further clarification of project.


Satellite imagery
Temperature, Relative Humidity, Weather
Crop map
Ground Trothing
Branley Criddle Method
E o
Crop Coefficient (Kc)
E c
Daily flow in channels
Well Observational Data
60% Efficiency Losses & 15 % Conveyance Losses
Water supplied
Crop Water Requirement
Water deficit in minor
WRWS
Actual Water

## 4.1 MAIN HAKRA BRANCH CANAL

Hakra branch canal is a perennial distributary whose present working discharge is about 2708 cfs having two outlets at each RD. The required discharge is about 3000 cfs but countering the collapse of canal that took place at high discharge (2800 cfs), the discharge was kept below the required level. The overall feedback from farmers and residential was satisfactory in comparison with other branched canals. Distributary water is fine and is suitable for irrigation. Although feedback was satisfactory, yet some issues were still there. Political influence and water theft complaints were the major issues that disturbed the efficiency of the distributary.

People also complained about rotation plans and needed that to be properly optimized. The major losses that we came across were due to damages in the sections of the canal where proper maintenance were required.

## 4.2 3-L DISTRIBUTARY

It is our main Area of interest which is the minor of Main Hakra branch towards the left. The major command area is towards the left of the distributary. The number of outlets that originate from this disty is 6. 5 of them feed the left side of the disty while one is towards the right. Head regulator is shifted recently to the downstream side of the disty and it was also lengthened in 1997. Every year it is closed for a month for maintenance purpose. There are no gauges at outlets so flumes are used for measurements when needed.




***Figure 9:*** Head of 3-L Disty,Hakra Branch

## 4.3 COMMAND AREA DETAILS

The total area which is in the command of 3-L disty is approximately 1787 acres. Most of the area is towards the left of the distributary. It is majorly irrigated by flooded method. Rotation technique is carried out for irrigation which is also called Warabandi. Major crops that are cultivated are wheat and cotton. According to the farmers the area is waterlogged and having salinity. The underground water is blackish so the use of tube wells is rare. Water from tube wells is mixed with surface to use it for irrigation purpose.

## 4.4 MAJOR CROPS AND THEIR SEASONS

As mentioned above, major crops that are cultivated are wheat and cotton. The Duration, sowing and harvesting of these crops, and their yield are tabulated below.

| | Rabi Crop | Kharif Crop |
|---|---|---|
| Crop Name | 80 % Cotton, 20 % Fodder/mix vegetables | 80 % Wheat, 20 % Fodder/mix vegetables |
| Sowing Date | April Start | Mid-November |
| Harvesting Date | November | April End |
| Crop Season Duration | Approx. 215 days | Approx. 150 days |
| Irrigation Type | Flooding | Flooding |
| Crop Yield | 35 monnes / acre | 20-30 monnes /acre |

## 4.5 NUMBER OF TUBE WELLS:

Due to the poor quality of underground water the numbers of tube wells are less. Total of 8 to 10 tube wells are located in the total command area. Mainly diesel engines are used to pump the water up. Consumption of diesel is about 4 to 5 liter per acre. The pipe used at the outlet of each tube well is having a diameter of 4 inches. It takes 3 to 4 hours to irrigate an area of 1 acre. Water is blackish and saline, so it is used in mixture with surface water.

## 4.6 OUTLETS AND THEIR FEATURES:

There are total of 6 outlets that distribute the water in their respective commanding areas. 5 of them are feeding the left side of the disty while only one irrigates the right side. Going along the distributary, we calculated the cross-sectional dimensions and co-ordinates of each outlet. These are tabulated as under.

| Outlet No. | Length | Width | Depth | Area Occupied | Coordinates |
|---|---|---|---|---|---|
| MR-1 | 2 km | 3.4 ft | 3 ft | 340 acres | 322623 N,3258307 E |
| ML-2 | 4 km | 2 ft (lined) | 3 ft | 900 acres | 322900 N,3258007 E |
| ML-3 | 1.5 km | 3 ft | 3 ft | 200 acres | 322647 N,3257370 E |
| ML-4 | 3 km | 3.5 ft | 3 ft | 345 acres | 322228 N,3256509 E |
| ML-5 | | 2.5 ft | 2.5 ft | | 322228 N,3256509 E |
| MT-5 (Tail ) | | 2.5 ft | 2.5 ft | | 322205 N,3255862 E |

## 5. ANALYSIS AND DISCUSSION

Water balance systems, are supportive in deriving different means of solution of complex hydrological problems and examining the various components of the hydrologic cycle i.e. precipitation, precipitation, evapotranspiration, and runoff. Quantitative calculation of water resources and its variations under the influence of human activities, changes in ground water levels, and variant crop water requirements forms a hydrologic verification for use, control and redistribution of water resources w.r.t time and space I. e inter-basin transfers, stream flow control etc. Knowledge of the water balance assists the prediction of the consequences of

artificial changes in the regime of streams, lakes, and ground-water basins. Basins can be used for operational management of reservoirs and for the compilation of hydrological forecasts for water management

With water balance data it is possible to compare individual sources of water in a system, over different periods of time, and to establish the degree of their effect on variations in the water regime. Further, the initial analysis used to compute individual water balance components, and the co-ordination of these components in the balance equation make it possible to identify deficiencies in the distribution of observational stations, and to discover systematic errors of measurements. Finally, water balance studies provide an indirect evaluation of an unknown water balance component from the difference between the known components (e.g. long-term evaporation from a river basin may be computed by the difference between precipitation and runoff).

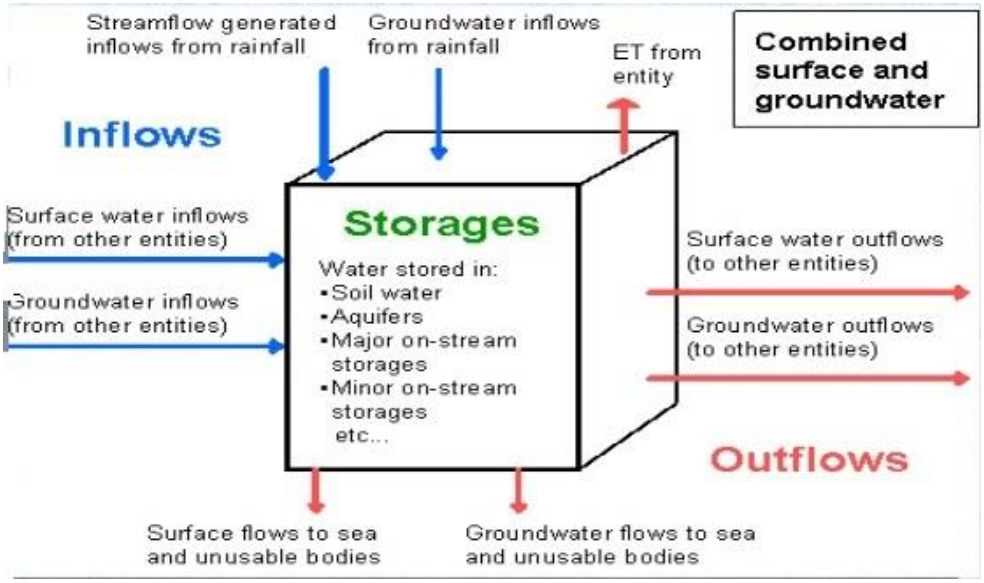



***Figure 10:*** Water Balance Model

## 5.1 METHODS OF CALCULATING EVAPOTRANSPIRATION

The methods for calculating evapotranspiration are classified into three types:

### 5.1.1 DIRECT METHODS (FOR RESEARCH PURPOSE)

- Lysimetric method
- Field experimentation method
- Soil water depletion method
- Inflow-outflow method

### 5.1.2 PAN EVAPORIMETER METHOD

- USWB class-A pan evaporimeter

### 5.1.3 EMPIRICAL METHODS

- Blaney-criddle method

- Penman method
- Modified penman method
- Radiation method
- Penman Monteith equation

There are many other methods, only some methods are mentioned here. For accurate and more precise results, empirical methods are used as it involves the different weather parameters along with a grass-reference crop.

## 5.2 FAO-24 BLANEY CRIDDLE METHOD

FAO-24 blaney criddle method was opted for calculating the water requirements in the project area. The reason for choosing this method is due to the availability of weather data for its calculations. Penman Monteith equation and radiation method requires sunlight radiations data which is available for limited weather stations.

### 5.2.1 ABOUT METHOD

In 1977, Doorenbos and Pruitt proposed the fundamental revision of the Blaney Criddle Method since its introduction. FAo-24 Blaney Criddle Method requires the intermediate step of estimating a grass-related reference crop evapotranspiration, ETo, prior to applying crop coefficient, Kc, to obtain crop ET, Etc. Older versions of Blaney Criddle Method did not deal with use of reference crop evapotranspiration, ETo.

### 5.2.2 CALCULATION PROCEDURE

After calculation of ETo, crop coefficients should be used by FAO-24 to estimate crop ET, Etc. This method is based on linear relationship between measured reference Evapotranspiration Et and Blaney Criddle factor.

Using data from different world-wide sites, various classifications based on ranges of daytime wind speed, minimum Relative humidity and sunshine expressed as n/N an expression is developed for Blaney Criddle Method as follows;

$E_{to} = a + b*f$ ***(Eq. 4.1)***

$f = p\ (0.46T + 8.13)$ ***(Eq. 4.2)***

$a = 0.0043*RH_{min.} - (n/N) - 1.41$ ***(Eq. 4.3)***

Where,

$E_{to}$= Grass reference evapotranspiration in mm $day^{-1}$,

P= mean daily percent of annual daytime hours,

T = mean daily temperature in $^0C$

$RH_{min.}$ = Minimum Relative humidity in percent

n/N = Possible to actual sunshine hours

Ud = Daytime wind at 2 – m height (m $sec^{-1}$)

A computer program was developed for interpolating values of b. Frevert et al. presented following expression for b;

$b = a_0 + a_1RH_{min.} + a_2\ 1.07(n/N) + a_3\ U_d + a_4RH_{min.}\ (n/N) + a_5\ RH_{min.}U_d$ ***(Eq. 4.3)***

Frevert regression coefficients $a_i$ rounded by Cuneca in 1987 are;

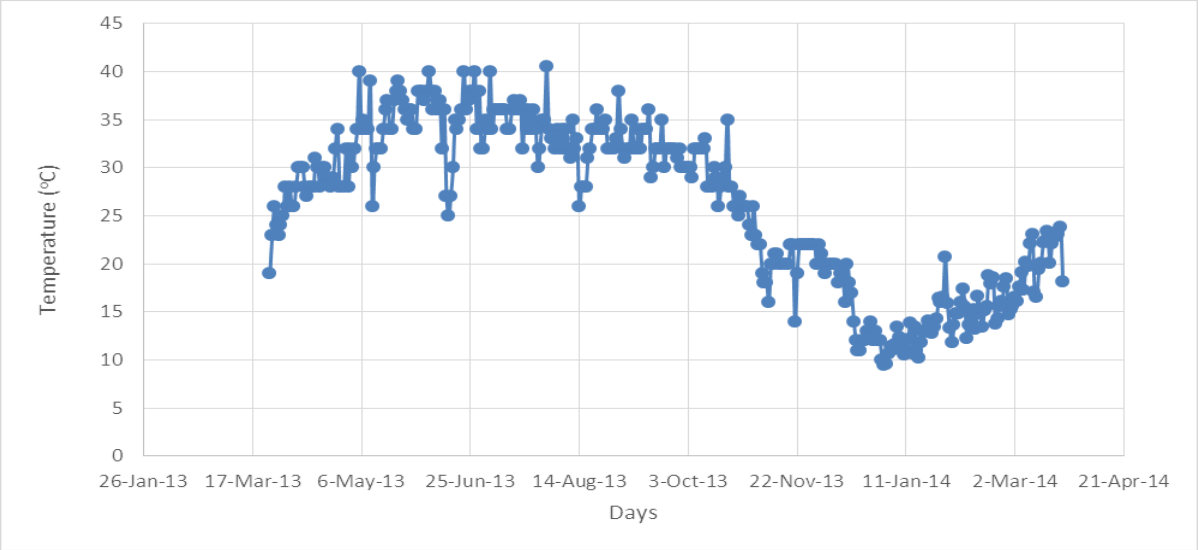
$a_0 = 0.82$, $a_2 = 1.07$, $a_3 = -0.041 \times 10^{-2}$, $a_4 = -0.6 \times 10^{-}$, $a_5 = -0.6 \times 10^{-3}$

### 5.2.3 COMMENTS ON WEATHER INPUT DATA

As clear from Eq.4.1,4.2 & 4.3, that ETo & ETc determination involves different weather parameters so variations of temperature and other weather parameters are discussed as under.

### 5.2.4 TEMPERATURE VARIATIONS IN BAHAWALNAGAR

For Bahawalnagar weather station the daily temperature variations from 25th March 2013 till 24th March 2014 are shown in graph 4.1. Maximum Temperature during whole crop period was 40.5 ºC and Minimum Temperature during the period was 9.5 ºC .




***Graph 1:*** Temperature Data

### 5.2.5 WIND SPEED

According to Blaney Criddle method, winds peed (Ud) in eq. 4.3 is daytime wind speed at height of 2 m. Maximum wind speed during the crop seasons from 25th March 2013 till 24th March 2014 is 12 $msec^{-1}$.

### 5.2.6 MINIMUM RELATIVE HUMIDITY

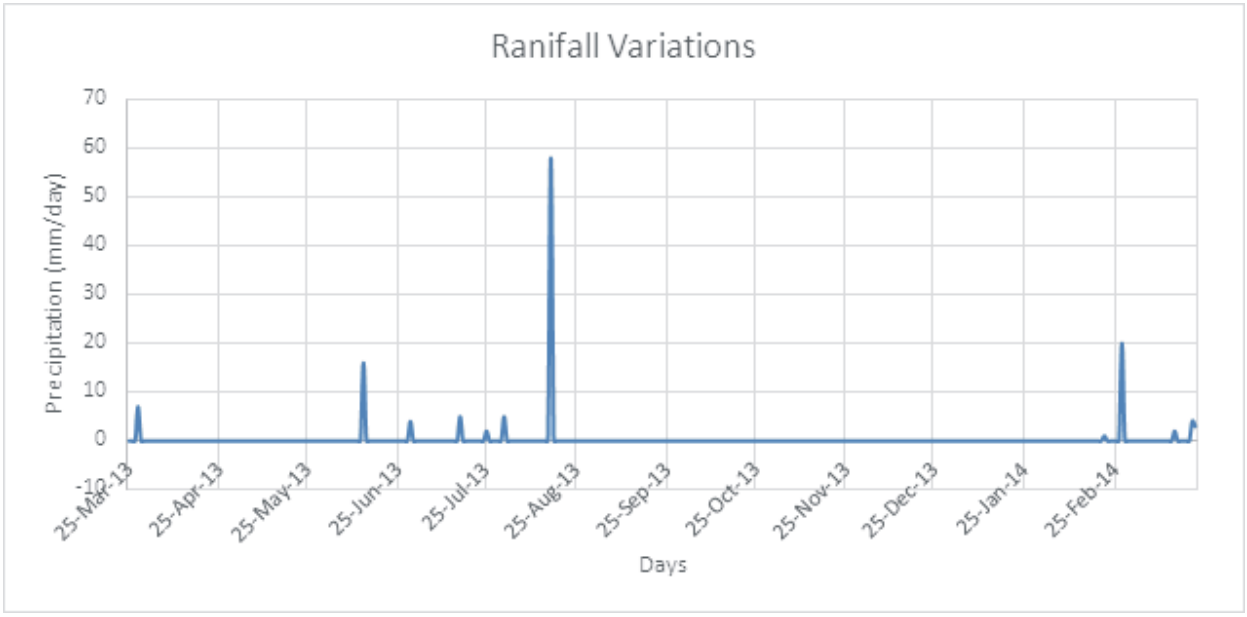
Minimum Relative Humidity in Eq. 4.2 & 4.3 refers to the minimum percentage of relative humidity during whole day. Humidity percentage in project area is average.

### 5.2.7 PERCENTAGE SUNSHINE HOURS

Blaney Criddle method provides the following table which refers to percentage sunshine hours for different latitudes.

***Table 1:*** Mean Daily Percentage (p) of Annual daytime hours for different latitudes

| Latitude | North | Jan | Feb | Mar | Apr | May | June | July | Aug | Sept | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | South | July | Aug | Sept | Oct | Nov | Dec | Jan | Feb | Mar | Apr | May | June |
| 60° | | 0.15 | 0.2 | 0.26 | 0.3 | 0.38 | 0.41 | 0.4 | 0.3 | 0.28 | 0.2 | 0.17 | 0.13 |
| 55 | | 0.17 | 0.2 | 0.26 | 0.3 | 0.36 | 0.39 | 0.38 | 0.3 | 0.28 | 0.2 | 0.18 | 0.16 |
| 50 | | 0.19 | 0.2 | 0.27 | 0.3 | 0.34 | 0.36 | 0.35 | 0.3 | 0.28 | 0.2 | 0.2 | 0.18 |
| 45 | | 0.2 | 0.2 | 0.27 | 0.3 | 0.34 | 0.35 | 0.34 | 0.3 | 0.28 | 0.2 | 0.21 | 0.2 |
| 40 | | 0.22 | 0.2 | 0.27 | 0.3 | 0.32 | 0.34 | 0.33 | 0.3 | 0.28 | 0.3 | 0.22 | 0.21 |
| 35 | | 0.23 | 0.3 | 0.27 | 0.3 | 0.31 | 0.32 | 0.32 | 0.3 | 0.28 | 0.3 | 0.23 | 0.22 |
| 30 | | 0.24 | 0.3 | 0.27 | 0.3 | 0.31 | 0.32 | 0.31 | 0.3 | 0.28 | 0.3 | 0.24 | 0.23 |
| 25 | | 0.24 | 0.3 | 0.27 | 0.3 | 0.3 | 0.31 | 0.31 | 0.3 | 0.28 | 0.3 | 0.25 | 0.24 |
| 20 | | 0.25 | 0.3 | 0.27 | 0.3 | 0.29 | 0.3 | 0.3 | 0.3 | 0.28 | 0.3 | 0.25 | 0.25 |
| 15 | | 0.26 | 0.3 | 0.27 | 0.3 | 0.29 | 0.29 | 0.29 | 0.3 | 0.28 | 0.3 | 0.26 | 0.25 |
| 10 | | 0.26 | 0.3 | 0.27 | 0.3 | 0.28 | 0.29 | 0.29 | 0.3 | 0.28 | 0.3 | 0.26 | 0.26 |
| 5 | | 0.27 | 0.3 | 0.27 | 0.3 | 0.28 | 0.28 | 0.28 | 0.3 | 0.28 | 0.3 | 0.27 | 0.27 |
| 0 | | 0.27 | 0.3 | 0.27 | 0.3 | 0.27 | 0.27 | 0.27 | 0.3 | 0.27 | 0.3 | 0.27 | 0.27 |




Daily percentage sunshine hours are calculated as;

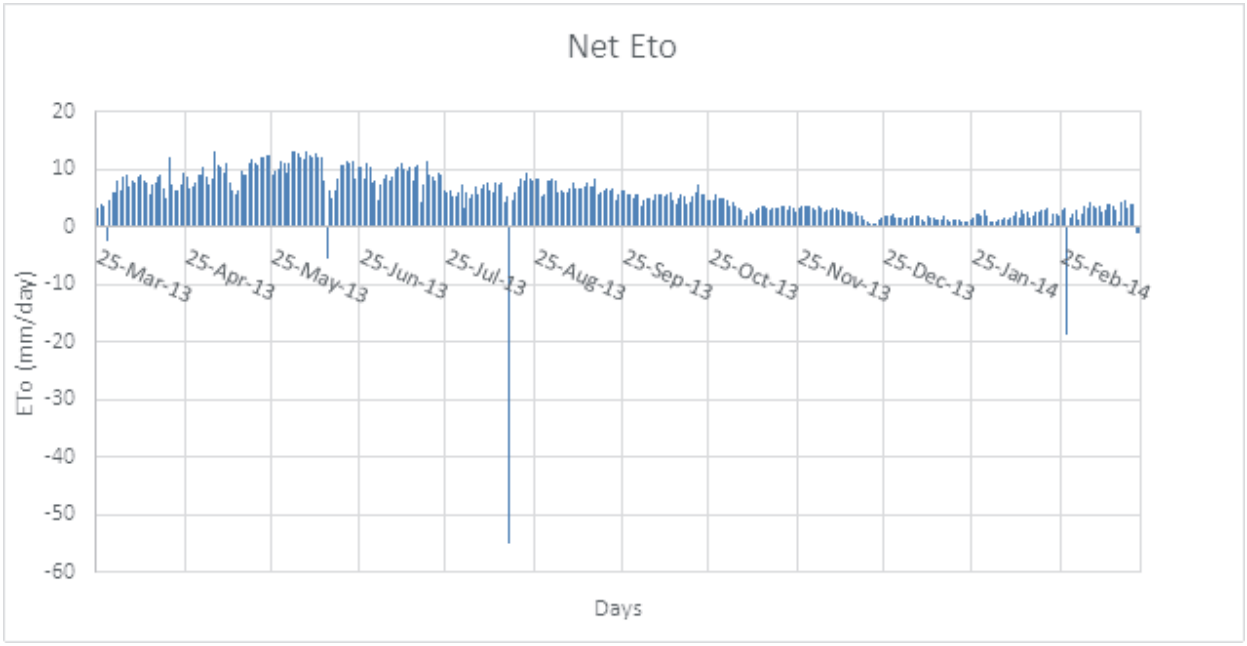
$$p_{daily} = p_{monthly}/30$$

### 5.2.8 PRECIPITATION

Rainfall data is plotted (Graph 4.2), highest rainfall of 58 mm was recorded on August 13,2013 in Bahawalnagar. Rainfall is subtracted from ETo to calculate Net Eto for further calculations.

## 5.3 NET POTENTIAL EVAPOTRANSPIRATION (ETO) RESULTS

The results of net Eto calculated by Blaney Criddle Method is shown in Graph 4.3. Eto is calculated using the daily weather data and by using equation 4.1 ,4.2 & 4.3 (FAO-24 Blaney Criddle Equations) .Net Eto is then calculated by subtracting rainfall from Eto.




## 5.4 CROP COEFFICIENTS -K-

FAO provides crop coefficients for calculating water requirements for different crops. Crop Development stages are divided into following phases [1]:

- Initial
- Crop Development
- Mid-Season
- Late

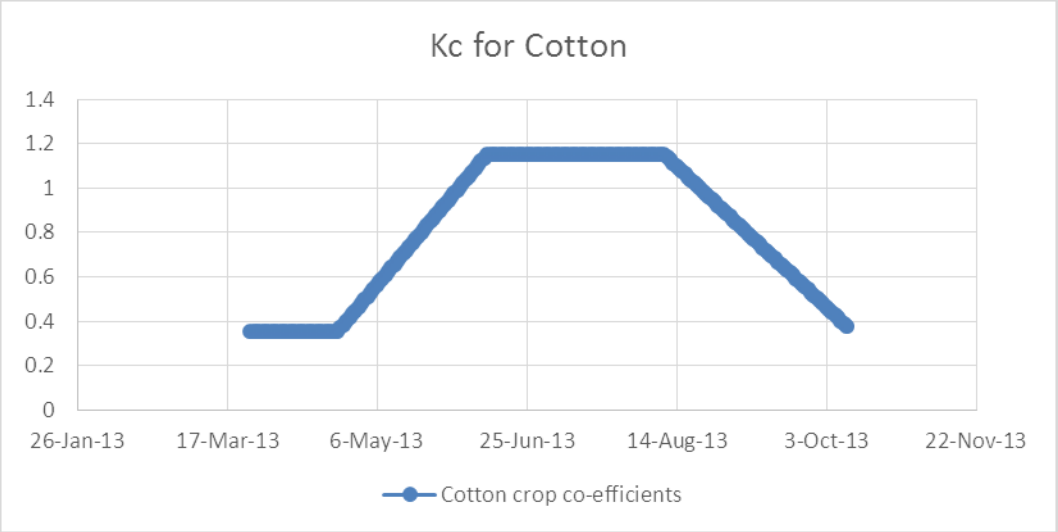
The major crops in project area are cotton and wheat with a small percentage of fodder. So crop coefficients variations and periods of different development stages are mentioned as under.

### 5.4.1 COTTON

Cotton is grown for fiber and seed. The length of the total growing period is about 150 to 200 days, therefore crop period is taken 200 days for the project area. Depending on temperature and variety, no. of days for different development stages vary respectively.

***Table 2:*** Crop Characteristics of Cotton provided by FAO

| Crop Characteristics | Initial | Crop Development | Mid-Season | Late | Total Days | Period |
|---|---|---|---|---|---|---|
| Stage Length, Days | 30 | 50 | 60 | 60 | 200 | 25 Mar'13 -- >10 Oct'13 |
| Crop Coefficient | 0.35 | >> | 1.15 | >> | 0.37 | |

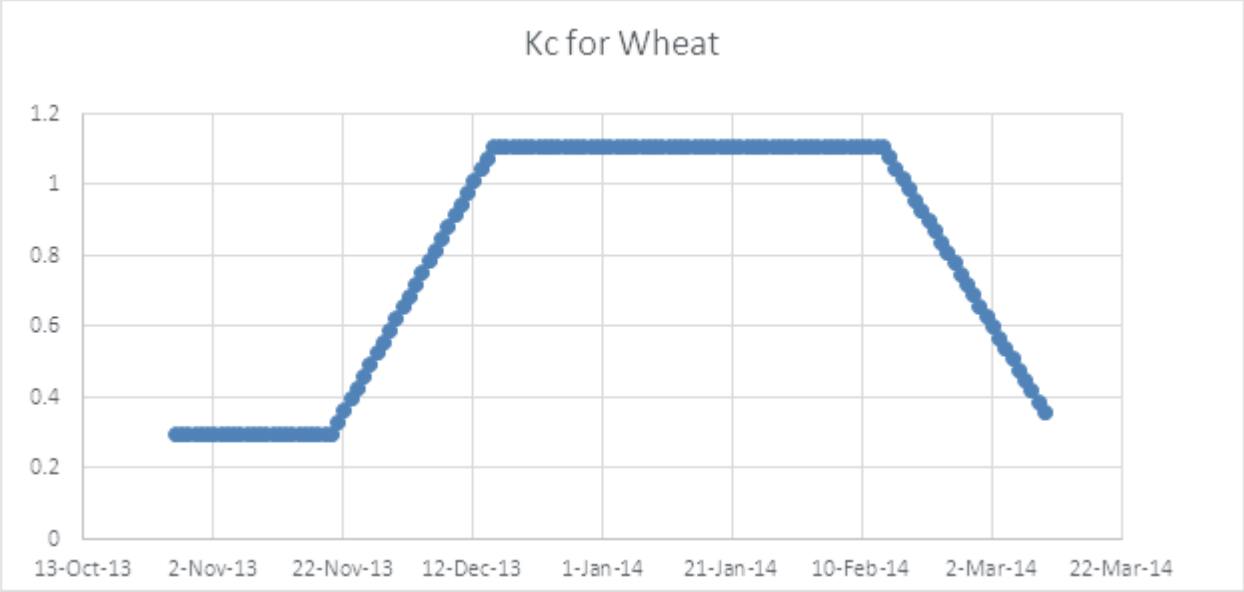



### 5.4.2 WHEAT

Wheat is major crop in the project area during winters. The length of the total growing period of spring wheat ranges from 100 to 130 days while winter wheat needs about 180 to 250 days to mature. [1]. Day length and temperature requirements are key factors in variety selection.

***Table 2:*** Crop Characteristics of Wheat provided by FAO

| Crop Characteristics | Initial | Crop Development | Mid-Season | Late | Total Days | Period |
|---|---|---|---|---|---|---|
| Stage Length, Days | 20 | 25 | 60 | 30 | 135 | 27 Oct'13 --> 24 Mar'14 |
| Crop Coefficient | 0.3 | >> | 1.15 | 0.35 | | |

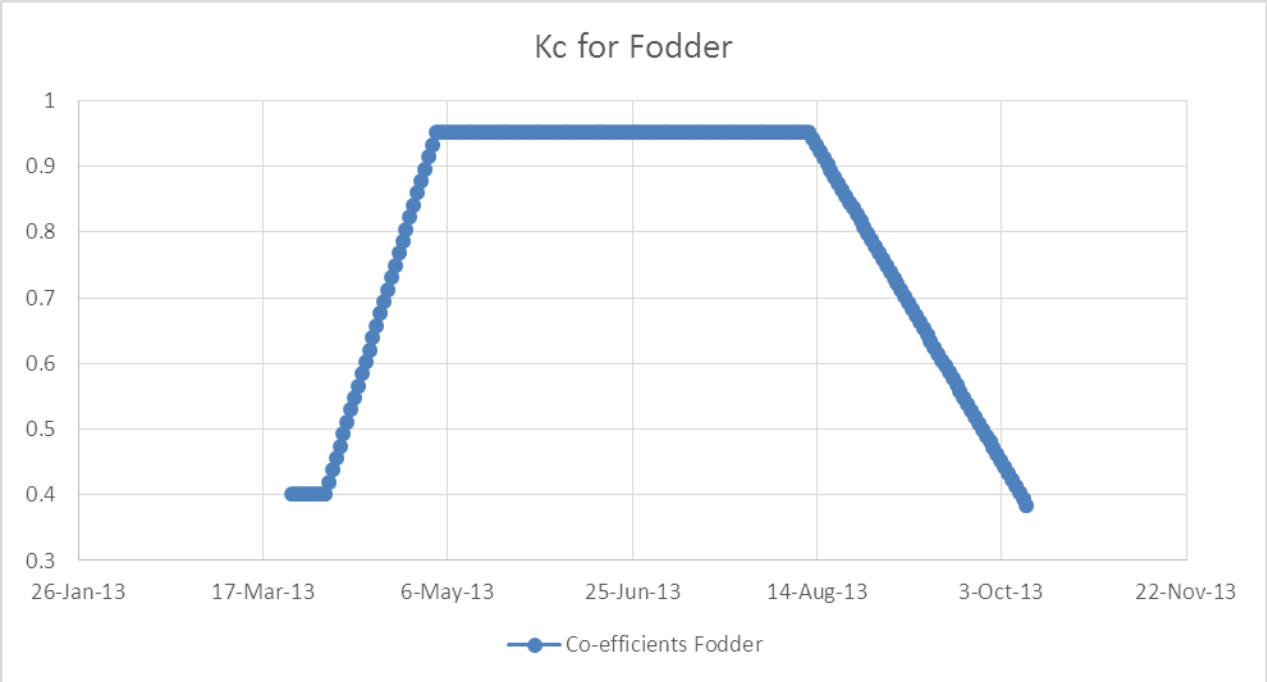



***Graph 1:*** Variation of crop-coefficients w.r.t days

### 5.4.3 FODDER

In project area, fodder includes alfalfa and different vegetables. Length of growth of alfalfa varies and it is sowed in both Rabi and Kharif season.

| Crop Characteristics | Initial | Crop Development | Mid-Season | Late | Total Days | Period |
|---|---|---|---|---|---|---|
| Stage Length, Days | 10 | 30 | variable | variable | variable | 25 Mar'13 -->10 Oct'13<br>27 Oct'13 --> 24 Mar'14 |
| Crop Coefficient | 0.3 | >> | 1.15 | 0.35 | | |

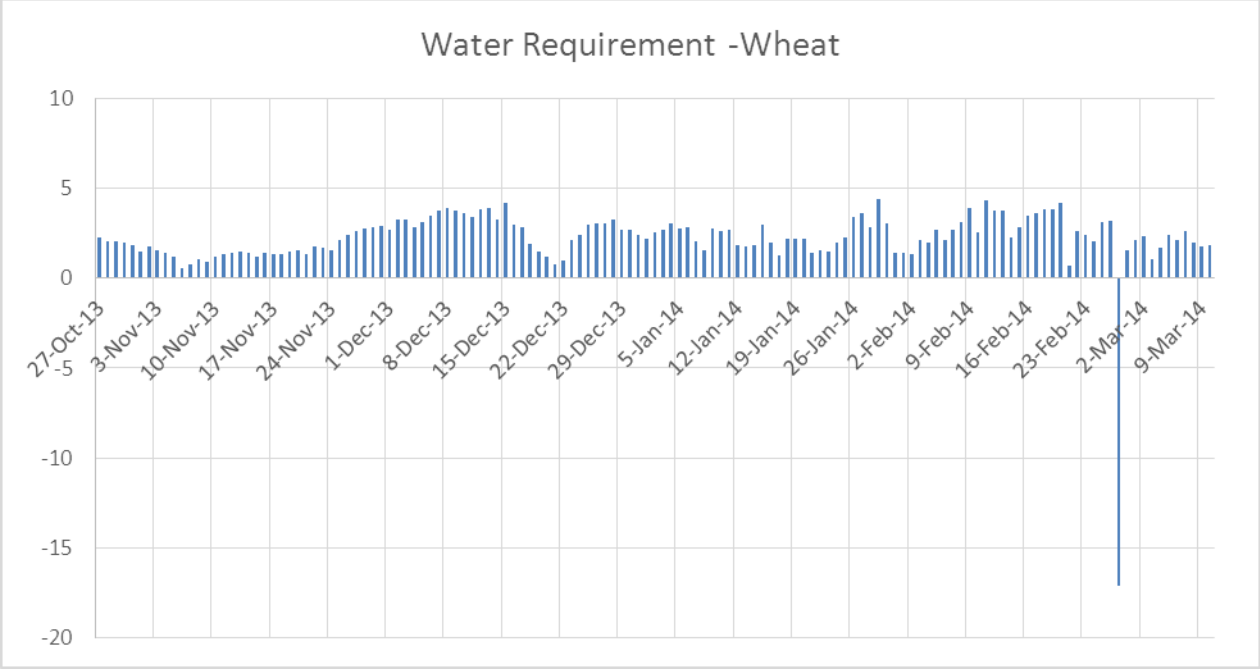


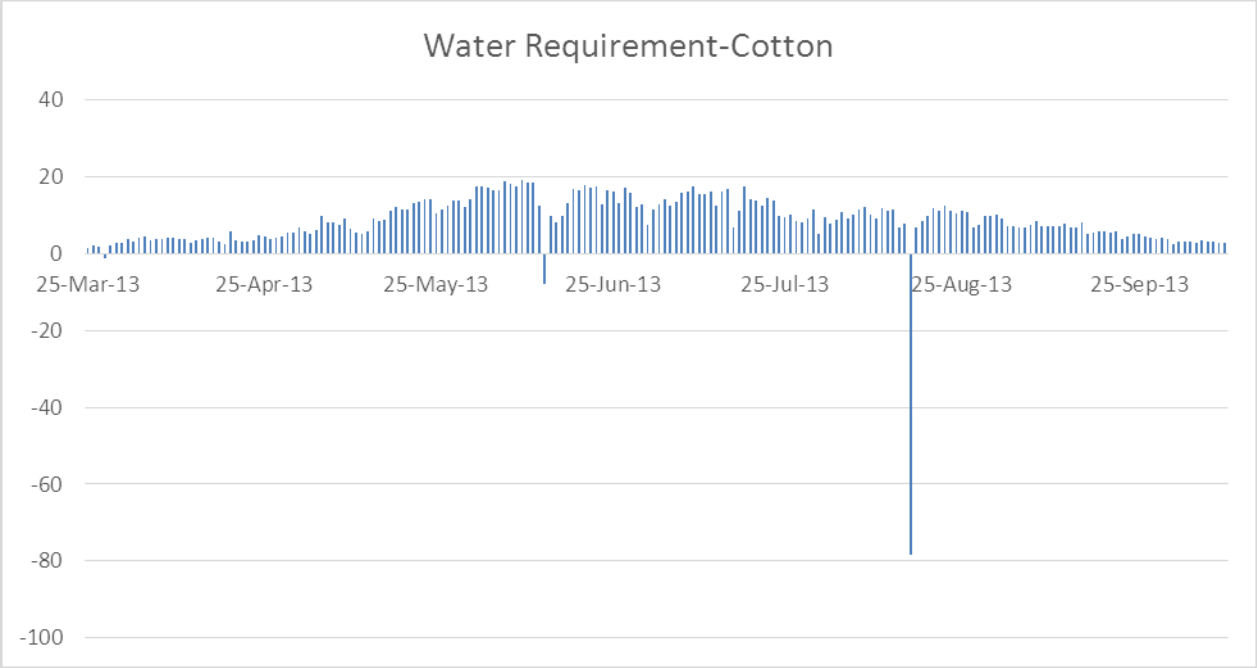

***Graph 2:*** Variation of crop-coefficients w.r.t days

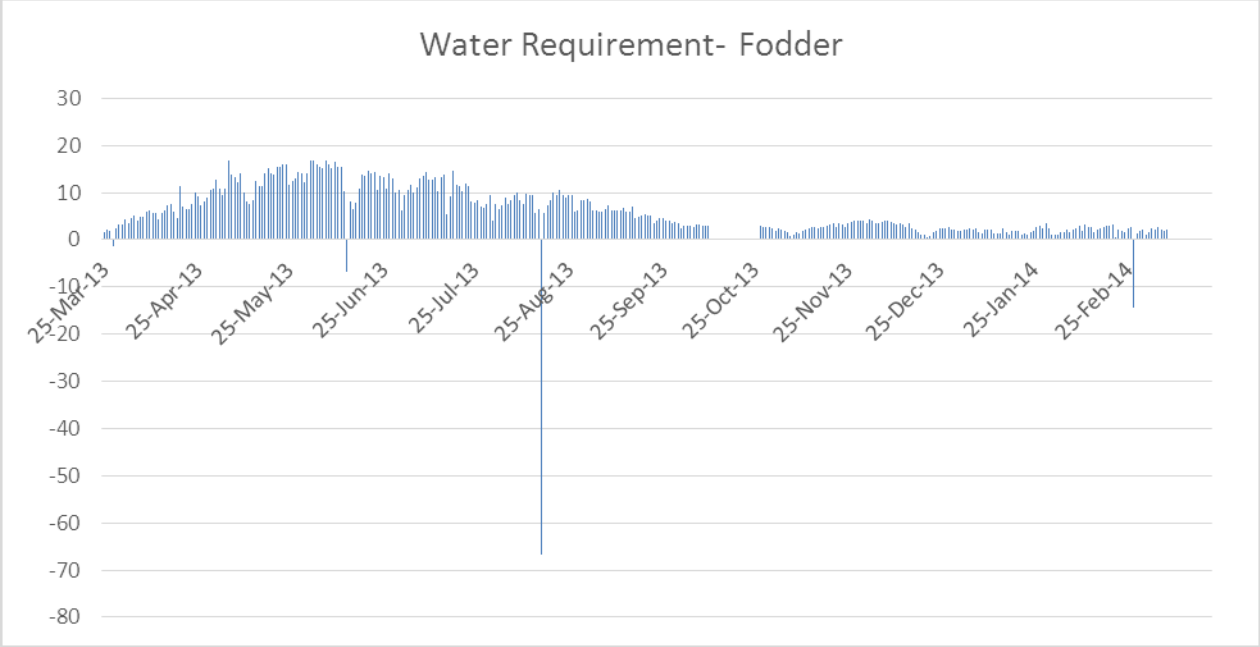
This crop co-efficient are multiplied by potential evapotranspiration, ETo to get crop evapotranspiration, Etc.

## 6. RESULTS

The results for water requirement for cotton, wheat and fodder with 15 % conveyance losses and 60 % efficiency losses, are as under.






## 6.1 WATER DEFICIT CALCULATION

Water deficit is calculated as:

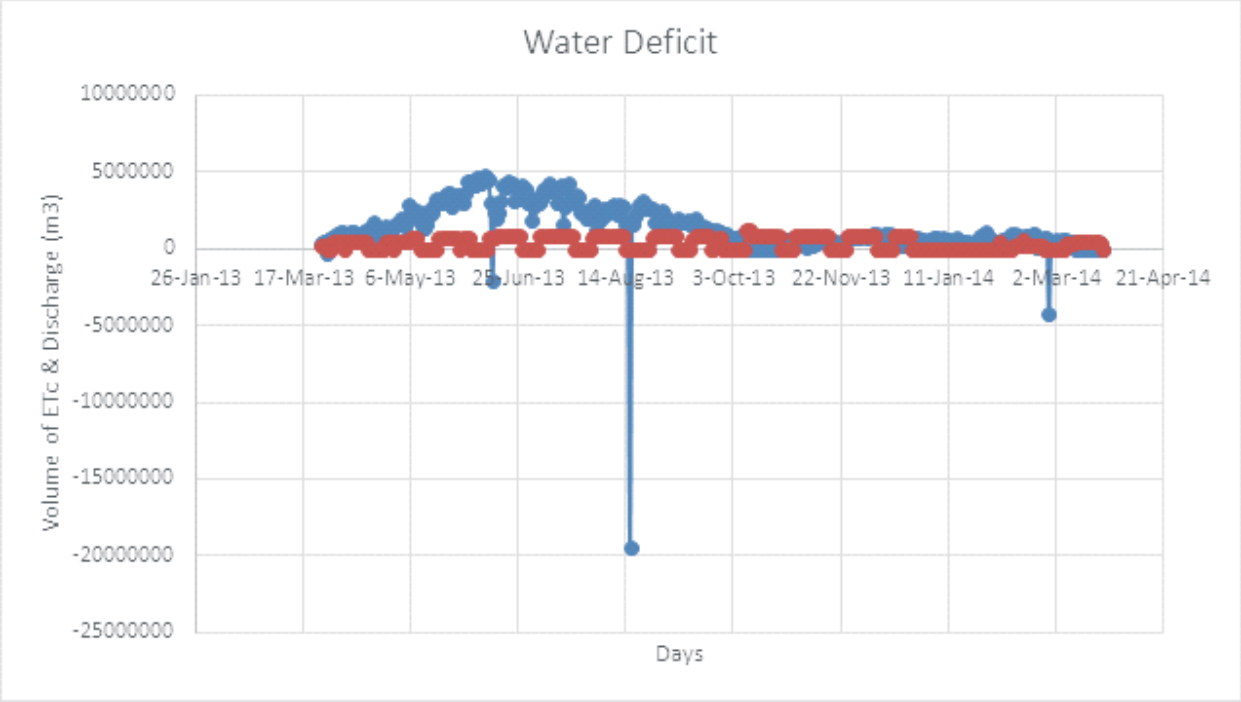
Water Requirement – Supplied Discharge through Distributary

For the project area annual water requirement, discharge and deficits comes as under;

| Water Requirement (cft.) | Discharge (cft.) | Water Deficit (cft.) |
|---|---|---|
| 506007283.3 | 144288000 | 361719283.3 |

Results show that 361719283.3 cft. Of water is in deficit as per the requirement of the area, which shows two possible outcomes;

- Either crop is under stressed; or
- Deficit shortage is fulfilled by ground water.




***Graph 3:*** showing water deficit for the project area

## 6.2 GROUND WATER ADJUSTMENTS FOR WATER DEFICIT

According to the reported data, 13-15 tube wells are used in 3-L distributary command area, but results show that up to 40- 45 tube wells can fulfill the deficit.

Results show that during Kharif season, more percentage of ground water is used than surface water to fulfill the deficit for the crops, whereas in Rabi season less percentage of ground water is used to fulfill deficit than surface water.

## 6.3 LIMITATIONS

There can be uncertainties in water balance system calculations. These can occur when;

1. The data sources are not accurate.
2. Temporal and spatial boundaries are not defined.
3. Quality of input weather and discharge data is poor.
4. Inappropriate extrapolation of data to field levels.
5. Political or institutional pressures.

## 7. CONCLUSIONS & RECOMMENDATIONS

Crop water budget estimation allows us to evaluate the crop water requirement and examine the water resources for their efficiency and functionality. After detailed study on command area,

cropping patterns, water discharge available, system functionality, irrigation methodology and other site attributes we can conclude our working in following major points

The area is crop stressed due to difference in requirement and discharge available. The requirement is much more than the actual available discharge in the distributary. The deficit is being fulfilled either by tube wells or water theft.

System efficiency is not adequate to meet the requirement of area. Various structural damages were reported due to theft, animals and political influences. Seepage is another major issue which is contributing to the rise in water table in the region.

Ineffective drainage system is causing various problems in the region which mainly includes water logging, salinity and decrease in crop yield.

Water logging is major issue in the area of 3L Disty. Which is mainly due to poor drainage, system losses (Seepage). After 2 feet of digging we can get water in the command area of 3L Disty. This rise in water table is causing immense salinity in the region causing decline in crop yield.

Uncertainties about various factor among the authorized personal is alarming. During our site visit, the responsible individual from relevant department was uncertain about various factor including No. of tube wells and command area details. These uncertainties can cause variation in results and computations.

The ground water is brackish and the farmers prefer the missing of canal water and ground water for irrigation. The expected ratio of mixing is 1:1 (Ground water : Canal Water). This Brackish water is also causing less yield and

On the basis of all the computational and field work, we must recommend following major points for future studies and research

A proper drainage system is need of the hour. The drainage system of command area of 3L Disty need some serious attention from relevant department for improving crop yield, eradicating salinity and overcoming water logging. This drainage can either be done by surface drainage or shallow surface drainage. Pumping can also be incorporated for effective drainage works.

The brackish ground water can be drained out and can be used for irrigation after treatment. The most appropriate steps can be as follows

1. Pumping out the water and utilizing it after proper treatment. A treatment plant can be setup near the vicinity
2. Drain out the water and mix it with canal water until the mixture gets neutralized. Divert the flow for irrigation purposes once it is stable and ready to use.

Drones can be used effectively for following major tasks

1. Data collection for crops, command area, water course condition etc.
2. Crop reporting and documentation
3. Crop mapping and classification
4. Monitoring and surveillances

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**Paper No. 784**

# ROOF TOP RAINWATER HARVESTING OPTIONS FOR WATER CONSERVATION – A CASE STUDY OF LAHORE

**Dr. Ghulam Nabi & Muhammad Tarique**

# ROOF TOP RAINWATER HARVESTING OPTIONS FOR WATER CONSERVATION – A CASE STUDY OF LAHORE

By:

**Dr. Ghulam Nabi[1], Muhammad Tarique[2]**

## Abstract:

This study aims to assess the rainwater harvesting options and issues, in big city of Pakistan. There is a huge potential; for this option. As an example high annual rainfall makes rainwater harvesting a logical solution to overcome water shortage in Lahore. The ground water resources are also depleting due to higher usage. As the growing population in the urban areas and higher demand of water for crops are putting increased load on underground aquifer, whereas in the last decade, the groundwater level in Lahore has decreased by more than 30 feet. The annual rain fall that the city receives may be the best management practice to recharge the aquifer. Rainwater harvesting during the rainy season can reduce the load on sewage and rainwater collection system which helps in ground water build up. Data from primary as well as secondary sources was used for this study, which aims to find out the potential of rainwater harvesting in Lahore and proposes various options after conducting analysis on; (i) land use and rainfall pattern of the study area of Lahore and their results, (ii) simulation of rainfall runoff and estimation of rainfall in the Central Drain Catchment using HEC-HMS, (iii) different design options of various rainwater harvesting systems consisting of groundwater recharge and rooftop rainwater harvesting structures, (iv) implementation of proposed options and estimation of subsequent impacts in the study area. The results showed that the rainfall pattern in the study area does not have a particular trend but there is significant potential for the implementation of rainwater harvesting systems in Lahore. Various beneficial design options are introduced as pilot study which demonstrate beneficial techniques.

**Keywords:** Rainwater Harvesting, Rainfall, Land Use, Trend Analysis, Mann-Kendall Test, Potential, Design Options.

## 1. INTRODUCTION

Rainwater harvesting is a tool for achieving sustainable water supply system. Rainwater harvesting is a method of collecting, storing and conserving local roof top and surface rainfall runoff for future domestic utilization as well as groundwater recharge. Rainwater is an easy source of water and it is mostly in its potable form as it does not have turbidity, color, FC, TC and odor. Being a sub-tropical country, Pakistan receives considerable rainfall across the year, which can be used for domestic purposes, gardening, industries and groundwater recharge. Rainwater can be utilized for groundwater injection and recharge which can uplift groundwater depletion of urban areas. Pakistan is one of the top ten water scarce countries of the world and it is the sixth largest countries of the world by population (UN, 2009). Lahore is the second largest city of Pakistan, where water scarcity and accelerating demand of water are major concerns. Groundwater is the only source of fresh water being used for drinking and other domestic purposes. Unfortunately, a large volume of runoff generated from annual rainfall

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ranging from 600 mm to 1600 mm is wasted after being mixed with the sewerage water and is discharged into the river Ravi. Lahore mainly receives its rainfall during the monsoon season (June till September) and in winter season (December till February). The average monsoon precipitation is 470.1 millimeters (PMD, Lahore). This rainwater can be harvested by technically feasible rainwater harvesting structures which could be engineered to be used for different purposes.

In many parts of the world, rainwater is frequently used to deal with water scarcity and urban flooding. Rainwater harvesting is necessary in Lahore for the mitigation of existing water crisis and raising of groundwater level. Although Lahore is potentially candidate for rainwater harvesting, but its success might get hindered due to technical and socio economic problems. The purpose of this study is to evaluate the different options of rainwater harvesting being utilized in the world and to identify the different factors that will lead to a successful implementation of rainwater harvesting in selected areas of Lahore. This study reviewed the literature and assessed the potential for implementation of different types or methods of rainwater harvesting; roof top rainwater harvesting, surface runoff rainwater harvesting and solutions of the problems associated with rainwater harvesting.

This study focuses on the selected urban area of Central Drain Catchment of Lahore which might be; (i) useful to evaluate the effectiveness of existing rainwater harvesting and groundwater recharge structures, (ii) also different practicable rainwater harvesting options and issues were evaluated to design new rainwater harvesting structures, (iii) identifying the best rainwater harvesting potential sites and management options for groundwater recharge structures to conserve the groundwater resources, (iv) GIS 9.3 Tool was used for study area demarcation, assessment of different land cover types and land use patterns, (v) Rainfall runoff modelling was done using HEC-HMS Software.

## 2. STUDY AREA

Lahore is the capital of Punjab province, located at geographical coordinates 31°15'-31°45' N (latitude) and 74°01'-74°39' E (longitude), having an estimated population of 10.05 million with total area of 1772.5 square km. It is administratively divided into ten towns. Lahore is surrounded by Kasur District on the south, Wagah on the east and Sheikhupura District on the north and west. The Ravi River is located and flows on the northern side of Lahore. It flows to the South and South-West direction with gradient of one meter per 3 km (JICA 2009). Lahore city consists of a total land area of 1014.5 km² and it is still growing. The study area is situated near river Ravi. Average elevation of the study area is 217 meters above mean sea level as it is shown in the Figure 1.

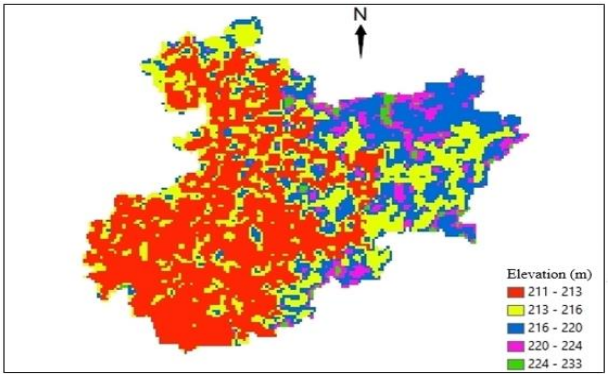



***Figure 1:*** Digital Elevation Model (DEM) of Central Drain, Lahore Basin.




***Figure 2:*** Location of Central Drain, Lahore Basin (Courtesy of Google Earth)

Catchment of Central Drain, Lahore was selected as the study area having an area of 9.45 $km^2$ shown in Figure 1.2. Central Drain is a secondary drain and it discharges its water into Cantonment Drain, which is a primary drain in Lahore. Cantonment Drain, ultimately, discharges into the river Ravi. The Study area receives an average annual rain fall of 623 mm (1960-2011). Due to high intensity (288 mm/24 hour) rainfall during Monsoon, water gathers in some of the low lying areas of Lahore and causes spreading of different water borne diseases like malaria, dengue and roads get flooded resulting into traffic jam. There are many low lying locations, situated in commercial, industrial and residential areas in the catchment of Central Drain which cause urban flooding. However, Water and Sanitation Agency (WASA) manages water disposal system by pumping rainwater into the municipal sewers and uses excessive pumping to drain the flooded locations. This rainwater disposal practice results in wasting the useful source of water. Rainwater harvesting can be the best management practice (BMP) to mitigate the problems related to water scarcity and urban flooding. Figure 2 shows the location of study area.

## 3. MATERIAL AND METHODS

Several types of data were collected from various departments and sources in the form of excel sheets, digital maps, published and unpublished reports, papers and other related information about rainwater harvesting, storm water management and development of the selected urban area of Lahore and were analyzed, under a framework of rainwater harvesting options and issues to achieve the specific objectives of this study. The following collected data were analyzed to examine the different patterns of land use, rainfall and runoff quantity. (i) SRTM DEM/ Mask DEM data were required to analyse the topography of the study area. These data were obtained from USGS Earth Explorer website. Digital Elevation Model (DEM) data consists of a digital model or 3D representation of a terrain's surface, commonly for a planet (including Earth), moon, or asteroid, created from terrain elevation data. (ii) Landsat data/land use data were required for the land cover analysis. These data were downloaded from the official website of USGS, http://glovis.usgs.gov. Landsat data were obtained through satellites and it consists of imagery of Earth's surface. It is managed by United States of America (USA). (iii) Hourly and daily precipitation data for the study area were required for rainfall analysis. These data were collected from Pakistan Meteorological Department (PMD), Lahore. The first objective, to find the land use, rainfall and runoff patterns, was achieved by analyzing the Landsat 8 land use data of August 2015 and daily rainfall data from 1960-2011. The second objective was achieved by analyzing the literature data to find out the issues of rainwater harvesting, subsequently, potential harvesting sites were selected by analyzing the results of HEC-HMS runoff modelling for the selected area. Different options were assessed on the basis of these results. For achieving third objective, different designs were worked out for rooftop rainwater harvesting and groundwater recharge structures. It is discussed in detail as given below;

(a) Landsat data were analyzed for land use pattern in study area, using remote sensing tools. Mask DEM was used in Arc GIS 9.3 tool to demarcate the catchment of the study area of Central Drain, Lahore and land use distribution maps were also prepared by using GIS.

(b) The rainfall patterns in the study area were modelled through temporal analysis of rainfall data on daily, monthly, seasonally and yearly basis. Rainfall analysis included rainfall pattern, its frequency, number of rainy days and maximum rainfall in a day, its variation with time etc. Rainfall Frequency analysis was carried out on the basis of past record of extreme rainfall events. Using annual series data of extreme rainfall events, their return periods were estimated by Weibull method.

(c) The runoff quantity was estimated using HEC HMS model. To find out the potential of rainwater harvesting and design capacity of different rainwater harvesting systems in the study area, the extreme rainfall event of 200 years return period was used. The details of these processes of data collection, modeling and analysis are provided below.

## 3.1 LANDUSE CLASSIFICATION

The Landsat 8 satellite data of the year 2015 for the monsoon months were collected from earthexplorer.usgs.gov. For the land use classification, only those images were selected and used in this study, which were of good quality and free of clouds. The images of November, 2015 were used and analyzed for classifying the different land uses. The Landsat-8 image is shown in Figure 3.

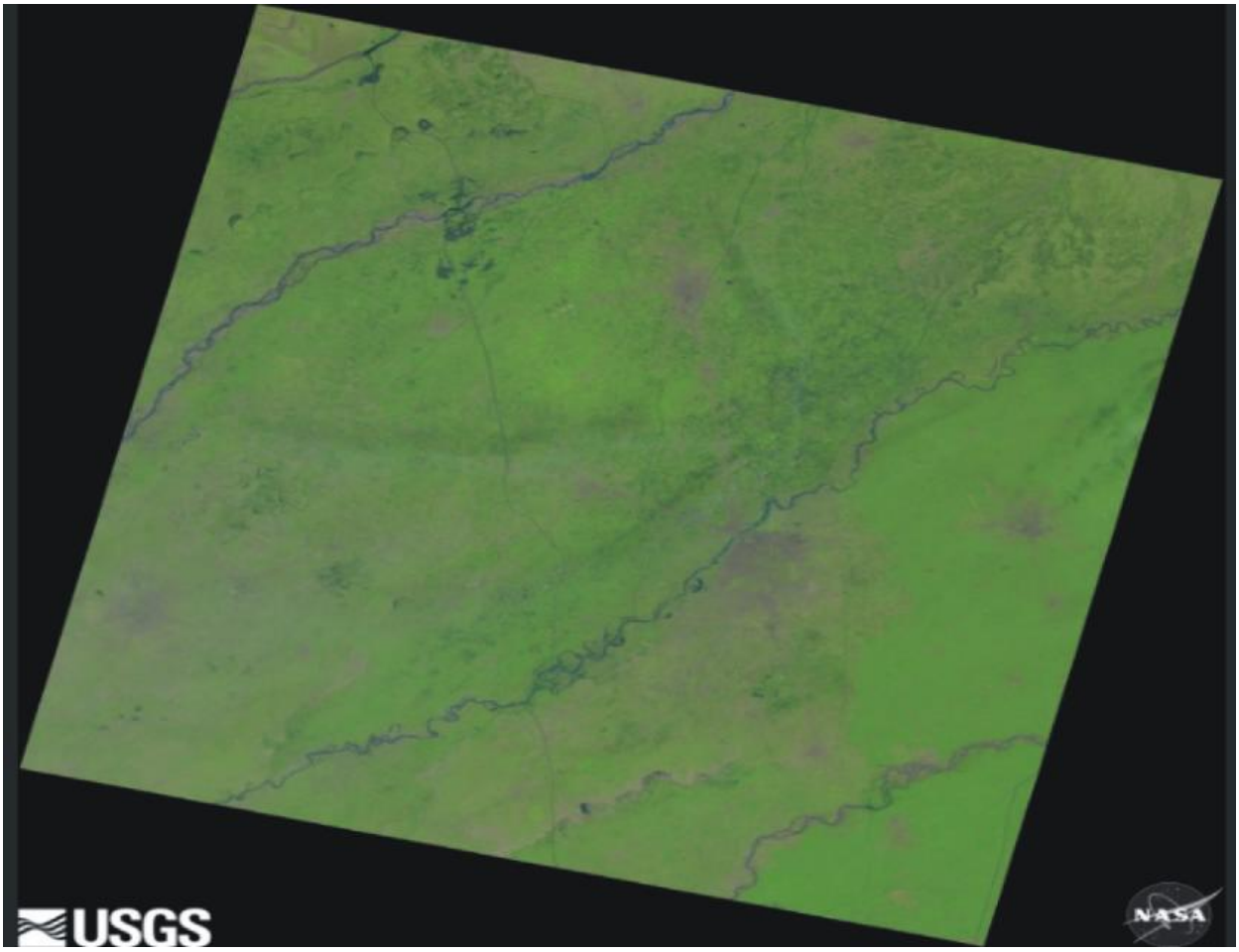



***Figure 3:*** Landsat-8 Image Downloaded from USGS Website.

## 3.2 RAINFALL RUNOFF PATTERN ANALYSIS

Rainfall is a key factor for the estimation of runoff potential of any catchment. Rainfall data on daily and monthly basis were collected from Pakistan Meteorological Department (PMD) Lahore from years 1961 to 2011 and 1931 to 2013, respectively. Statistical and graphical analysis of the rainfall data were carried out on daily, monthly and yearly basis. One day maximum rainfall events data for 52 years (1960-2011) were sorted out and frequency analysis was done to estimate the return periods for different

extreme events to select the design storm for the HEC-HMS modeling. Similarly, trend analysis was done using the Mann-Kendall Trend Test to estimate the rainfall patterns and trends for one day maximum rainfall events and rainy days. Mass curve was constructed for the selected rainfall event.

The graphical analysis of rainfall pattern was done using excel sheets for six time steps: (i) average monthly rainfall pattern analysis, (ii) average annual rainfall pattern analysis, (iii) one day annual maximum and minimum rainfall pattern analysis, (iv) ten year rainfall pattern analysis, (v) total monthly rainfall pattern analysis and (vi) annual total rainfall pattern analysis. The detail of the above mentioned rainfall analysis is given in sub-sections.

### 3.2.1 Rainfall Trend Analysis

A trend indicates the significant change in any variable over time. Trend can be detected by different types of statistical tests; (i) parametric test and (ii) non-parametric test. Parametric test shows the magnitude of the trend while the non-parametric test is used to find out the statistical significance of the trend. In this research study, the Mann-Kendall test was used which is non-parametric test and gives the statistical significance of the trends of different variables. Mann- Kendall test is a statistical method which is mostly used to check the null hypothesis of no trend versus the alternative hypothesis of the existence of monotonic increasing or decreasing trend of hydro-climatic time series data. MAKESENS (FMI, 2002), the Excel template application based on the nonparametric Mann-Kendall test for the trend and the nonparametric Sen's method for the magnitude of the trend, performs two types of statistics depending upon the number of data values, (i) S – statistics is used if number of data values are less than 10 while, (ii) Z – statistics (normal approximation/distribution) for data values greater than or equal to 10. A positive or negative value of S indicates an upward (increasing) or downward (decreasing) trends, respectively. If number of data values are 10 or more, the S – statistics approximately behave as normally distributed and test is performed with normal distribution. Statistically the significance of trend is assessed using Z- value. A positive value of Z shows upwards (increasing) trend while the negative value indicates downward (decreasing) trend.

### 3.2.2 Rainfall Frequency Analysis

Rainfall data of climatic station at Lahore was selected to determine the design storms for the study area at various return periods. Rainfall data was available from years (1961-2011) to project design storm at higher return periods. Frequency analysis of one-day annual maximum rainfall data was carried out using Gumbel's Extreme Value Type-I distribution and Pearson's type-III distribution.

In log Pearson type- III distribution, the value of variate 'X' (rainfall) was transformed to logarithm (base 10). The expected value of rainfall '$X_T$' was obtained by the following formulae,

$$X_T = \text{Antilog } X \quad (1)$$

$$\text{Log } X = M + K_T S \quad (2)$$

Where, 'M' is the mean of the logarithmic values of observed rainfall and 'S' is the standard deviation of these values.

Frequency factor $K_T$ was taken from Benson (1968) corresponding to coefficient of skewness ($C_s$) of transformed variate as;

$$K_T = \frac{2}{C_s}\left[\left\{\left(z - \frac{C_s}{6}\right)\frac{C_s}{6} + 1\right\}^3 - 1\right] \quad (3)$$

In Gumbel distribution, the expected rainfall '$X_T$' was computed by the following formula;

$$X_T = \overline{X}(1 + C_V K_T) \quad (4)$$

Where, X is the mean of the observed rainfall, $C_v$ is the coefficient of variation and $K_T$ is the frequency factor which was calculated by the formula given by Gumbel (1958);

$$K_T = -\frac{\sqrt{6}}{\pi}\left\{0.5772 + \ln\left[\ln\left(\frac{T}{T-1}\right)\right]\right\} \quad (5)$$

Plotting positions were computed by Weibull's formula. The results of frequency analyses are presented in next sub-section 4.3.1, in the form of tables, plotting positions and fitted line graphs.

### (i) Selection of Storms

The historical daily rainfall data base for the station (Lahore Central Drain Catchment), were collected for past 52 years (1960-2011). This bulk of rainfall data were used for the selection of major storm events. The daily rainfall data for Lahore Central Drain Catchment were collected from the Pakistan Meteorological Department (PMD), Lahore. The daily rainfall record (1960-2011) was examined and most of the days were rainless. By grouping the data on basis of daily maximum storm event, the highest storm values were extracted for each year. All the annual maximum storm events were considered on daily basis for the Lahore Central Drain Catchment and details of the selected storm are shown in Table1.

***Table 1:*** Value of Extreme One Day Annual Rainfall Event Selected from Daily Rainfall Data (1960-2015).

| Extreme One Day Rainfall Event at Lahore | | | | |
|---|---|---|---|---|
| **Year** | **Rainfall (mm)** | **Rainfall (inches)** | **Date of Storm** | **Duration (hours)** |
| 2014 | 288 | 11.34 | 5 September | 24 |

### (ii) One Day Maximum Annual Rainfall Event Analysis

Analysis of selected one day maximum/extreme annual rainfall event is based on the construction of one day maximum annual rainfall event hyetograph and the mass curve. The hyetograph of extreme rainfall event of one day annual maximum rainfall was constructed with intensity of rainfall versus time. Similarly, the mass curve was constructed by accumulating the rainfall with respect to time intervals. Its results are discussed in sub-section 4.2.1.

## 3.2.3 RAINFALL RUNOFF MODELING

Runoff starts when the rainfall rate exceeds the rate of infiltration and urban storm runoff is surface runoff of rainwater created by urbanization. This runoff is a major source of urban flooding and water pollution in urban communities. Urban runoff is a major cause of inundation of land or property in a built-up environment caused by rainfall overwhelming the capacity of drainage systems and also affects the vehicular traffic on

roads. But this urban storm water runoff affects the traffic-carrying capacity through; (i) Sheet flow across the road surface, (ii) Channel flow along the road, (iii) Ponding of runoff on road surfaces and (iv) Flow across traffic lanes.

Traditionally, the urban storm runoff (USR) becomes the part of sewer. City managers should try to build hydraulic structures to save this fresh rainwater and make structures for recharging of ground water. For this purpose, in this thesis, urban storm runoff (USR) modelling has been done by HEC-HMS using SCS Curve Number Method. As mentioned earlier, the runoff quantity was obtained by SCS Curve Number Method through HEC HMS Model Simulation for different extreme rainfall events. The runoff collection pattern was analyzed by using the GIS application to observe the common points/ponds where runoff gets accumulated from the selected study area.

### (a) Estimation of Runoff by HEC-HMS Hydrological Model

For the estimation of runoff at Central Drain, Lahore Catchment, the hydrological model HEC- HMS (Hydrologic Modeling System) was used. Total drainage area of the Central Drain, Lahore Catchment shown in Figure-2, is 9.45 sq. kilometer, which is further sub-dived into three land use classes based on its different land uses; (i) buildings area, (ii) roads area and (iii) vegetation area. The rainfall run-off model was run against a maximum recorded rain fall of 288 mm in a day for a storm duration of 24 hours. The hourly rainfall data of extreme events and other meteorological data of the watershed for the year 2014 were collected from the PMD, Lahore office which is located near to the watershed. The maps like topographical, soil type, land slope, land use/pattern, drainage network, watershed boundary etc. for the watershed were extracted using Survey of Pakistan maps and GIS model which is used to create the basin model input for HEC-HMS software.

### (b) Model Simulation

Model was run for hourly basis rainfall data to know the rainwater harvesting potential in the study area. The results of the total estimated run-off of the selected catchment area of Central Drain, Lahore were prepared in the form of graphs and tables which are discussed in the results and discussion section.

## 3.3. DESIGN OPTIONS FOR RAINWATER HARVESTING SYSTEMS

Different design options for RWH systems were worked out and proposed subsequent to the rainwater harvesting potential was estimated from the results of HEC-HMS model simulation. Issues related to the RWH were also assessed. The details of different design options are given in results and discussion section.

### 3.3.1 Design Option for Roof Top RWH System

In the Lahore Central Drain Catchment, water is mainly required for domestic and industrial purposes and is drawn from ground water sources. Roof top rainwater harvesting is an ideal alternative in such area. Appropriate storage facilities can be created to store roof top rainwater depending on availability of space. Rainwater harvesting in the study area will partially help the water requirement. It will also prevent flooding of roads during heavy rains. It will also reduce the pumping costs and reduce the stress on ground water resources. Roof catchment and rainfall are important parameters for designing these systems. Therefore, for a particular roof area and average annual rainfall, the runoff was estimated using suitable runoff coefficient and storage tank with adequate volume was proposed to store the harvested rainwater.

### 3.3.2 Design Option for Harvesting Surface Runoff

Surface runoff that is generated in heavy rainfalls, can be harvested from roads, vegetation areas, lawns and parking lots. The potential of rainwater harvesting from roads and parking lots was estimated in the study area to work out the capacity of groundwater recharge systems.

### 3.3.3 Design Option for Groundwater Recharge System

Once the rainwater harvesting potential was estimated, different design options were considered to utilize this runoff to recharge the ground, which would reduce the roads flooding and over flowing of the Central Drain, Lahore and augment the water levels. To propose the recharge well for the installation, the most suitable and feasible design of the recharge well was selected and modified according to the requirements of the selected site in the study area of the Central Drain Catchment, Lahore. The detailed results of the above design option are given in Chapter 4, sub-section 4.6.3.

## 4. RESULTS AND DISCUSSION

The landuse data of Central Drain Lahore Basin (CDLB) comprises the area under different uses which consist of vegetation cover area including play grounds, grassy plots, parks, trees, populated area including commercial and residential buildings roof top area, roads and paved streets in selected urban study area. For the analysis of land use at spatial scale, different Remote Sensing (RS) techniques were used. Generally the area under different land uses changes with time, as the vegetation cover area changes with seasons and water availability. It was analysed and observed that the area under different land uses in selected urban study area of Lahore basin has less temporal changes. It was due to the dominance of urbanization and most of the study area consists of roads and buildings. The land use analysis of the CDLB is explained as under.

### 4.1 Landuse Analysis of Central Drain Lahore Basin

For the analysis of land use, the satellite data of Landsat 08 of August, 2015 was used. It was observed that the area under different land uses of CDLB was generally of three types; (i) vegetation cover area including play grounds, grassy plots, parks, trees, (ii) buildings area including commercial and residential buildings roof top area and (iii) roads area including paved streets area. The Land use classification of Landsat data was done using supervised classification technique in ERDAS Imagine software.

The analysis of Landsat image of Monsoon, August, 2015 is shown in Figure-4, which illustrates that most of the area of Central Drain, Lahore Basin consists of buildings (residential area) and streets, which is 79% (7.47 $Km^2$) of total area (9.45$Km^2$) of Central Drain Lahore Basin. Vegetation land is 9.77 % (0.92 $Km^2$) and the roads area is 11.21 % (1.06 $Km^2$) of the total area. The land use classification in term of percentage areal distribution in CDLB for the year 2015 is given in the Table 2.

***Table 2:*** Areal Distribution of Land Use Classes in Central Drain Lahore Basin.

| Land Use Class | Area ($Km^2$) | Percent of Total Area |
|---|---|---|
| Buildings Area | 7.47 | 79.02 |
| Vegetated Area | 0.92 | 9.77 |
| Roads Area | 1.06 | 11.21 |
| Total | 9.45 | 100 |

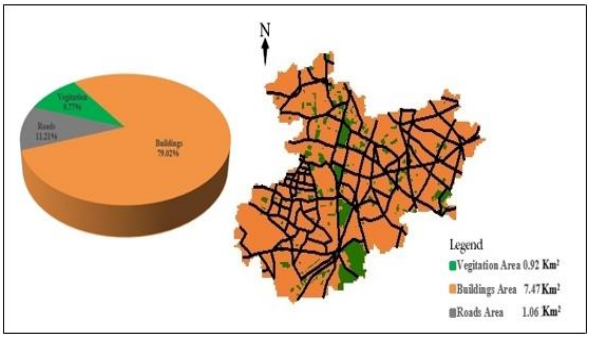



***Figure 4:*** Landuse Classification of Central Drain Lahore Basin (CDLB) 2015.

## 4.2 RAINFALL RUNOFF PATTERN ANALYSIS

Rainfall is a key factor for runoff modelling for any catchment. For CDLB, the rainfall data on daily and monthly basis was collected from Pakistan Meteorological Department (PMD) Lahore from years 1961 to 2011 and 1931 to 2013 respectively. This data was analyzed as given below;

### 4.2.1 One Day Annual Maximum Rainfall

One Day Annual Maximum Rainfall Data of 52 years (1960-2011) for Lahore is shown in Figure-5.

From the analysis of the one day annual maximum rainfall data (Figure 5), it is observed that the maximum one day rainfall, 211.1 mm, occurred in 1976 and minimum one day rainfall, 29.4 mm, occurred in 2002.

### 4.2.2 Monthly and Annual Rainfall

Monthly total, average, minimum, maximum rainfall from 1931 to 2013 and its range analysis is summarized as given in the Table 3 and Table 4. Table 5 shows the number of extreme events from 1960 to 2011 and total monthly and average monthly rainfall is shown in Figures 6 to 9.

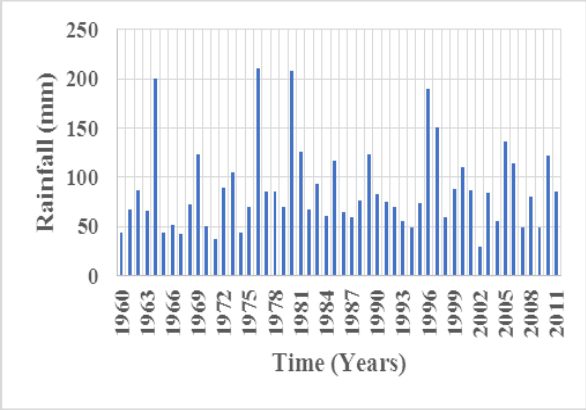



***Figure 5:*** One Day Annual Maximum Rainfall for 52 Years from 1960 to 2011.

***Table 3:*** Monthly Total, Average, Minimum, Maximum Rainfall and Rainfall Range Analysis.

| Time Series | Total (mm) | Mean (mm) | Maximum (mm) | Minimum (mm) | Range (mm) |
|---|---|---|---|---|---|
| **JAN** | 2073.8 | 24.99 | 121.2 | 0.2 | 121 |
| **FEB** | 2297.6 | 27.68 | 117.5 | 0.5 | 117 |
| **MAR** | 2530.4 | 30.49 | 166.7 | 0.2 | 166 |
| **APR** | 1414.8 | 17.05 | 141 | 0.3 | 140.7 |
| **MAY** | 1288.6 | 15.52 | 108.8 | 0.2 | 108.6 |
| **JUN** | 3870 | 46.63 | 208.6 | 0.3 | 208.5 |
| **JUL** | 13675.5 | 164.76 | 477.9 | 1.3 | 476.6 |
| **AUG** | 12933.6 | 155.83 | 640 | 2.5 | 637.5 |
| **SEP** | 5874 | 70.77 | 525.5 | 0.8 | 524.7 |
| **OCT** | 945.8 | 11.40 | 155 | 0.2 | 154.8 |
| **NOV** | 375.3 | 4.52 | 77.9 | 0.2 | 77.7 |
| **DEC** | 890.4 | 10.73 | 111.8 | 0.3 | 111.5 |
| **Annual (J- D)** | 48170.8 | 580.37 | 1232.5 | 277.2 | 955.3 |

***Table 4:*** Five Yearly Total, Average, Minimum, Maximum Rainfall and Rainfall Range Analysis.

| Time Series | Total (mm) | Mean (mm) | Maximum (mm) | Minimum (mm) | Range (mm) |
|---|---|---|---|---|---|
| **1960-1964** | 2601.5 | 520.3 | 800.8 | 297.5 | 503.3 |
| **1965-1969** | 2380.8 | 476.16 | 536.5 | 442.2 | 94.3 |
| **1970-1974** | 2349 | 469.8 | 826.7 | 344.5 | 482.2 |
| **1975-1979** | 3484.5 | 696.9 | 1117.5 | 459.3 | 658.2 |
| **1980-1984** | 4111.2 | 822.24 | 1015.6 | 645.2 | 370.4 |
| **1985-1989** | 3280.3 | 656.06 | 815.3 | 491.1 | 324.2 |
| **1990-1994** | 3021.2 | 604.24 | 955.2 | 374.9 | 580.3 |
| **1995-1999** | 4213.3 | 842.66 | 1232.5 | 473.7 | 758.8 |
| **2000-2004** | 2549.4 | 509.88 | 627.5 | 333.7 | 293.8 |
| **2004-2009** | 3049.5 | 609.9 | 750.8 | 372.2 | 378.6 |
| **1960-1969** | 4982.6 | 498.26 | 800.8 | 297.5 | 503.3 |
| **1970-1979** | 5833.5 | 583.35 | 1117.5 | 344.5 | 773 |
| **1980-1989** | 7391.5 | 739.15 | 1015.6 | 491.1 | 524.5 |
| **1990-1999** | 7234.5 | 723.45 | 1232.5 | 374.9 | 857.6 |
| **2000-2009** | 5598.9 | 559.89 | 750.8 | 333.7 | 417.1 |

***Table 5:*** Number of Extreme Events from 1960 to 2011.

| Years | | Number of Extreme Events (> 45 mm) |
|---|---|---|
| From | To | |
| 1960 | 1969 | 22 |
| 1970 | 1979 | 14 |
| 1980 | 1989 | 33 |
| 1990 | 1999 | 38 |
| 2000 | 2011 | 30 |
| Total (1960 to 2011) | | 137 |

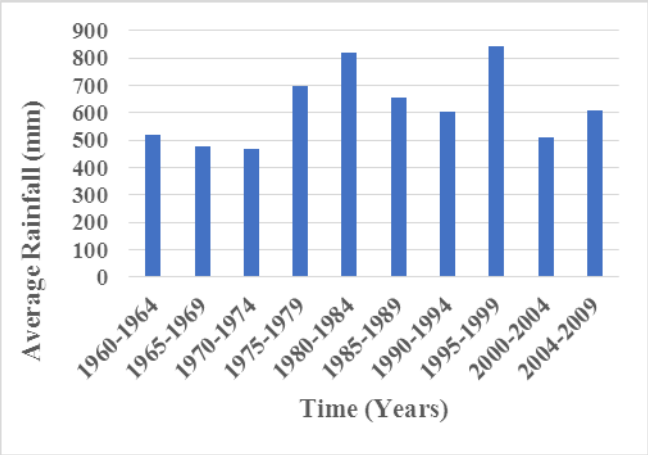



***Figure 6:*** Five Year Average Trend.

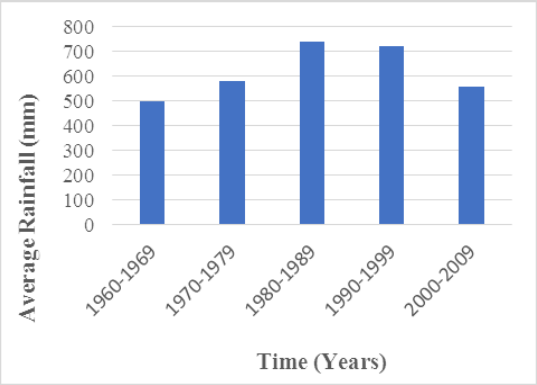



***Figure 7:*** Ten Year Average Trend.

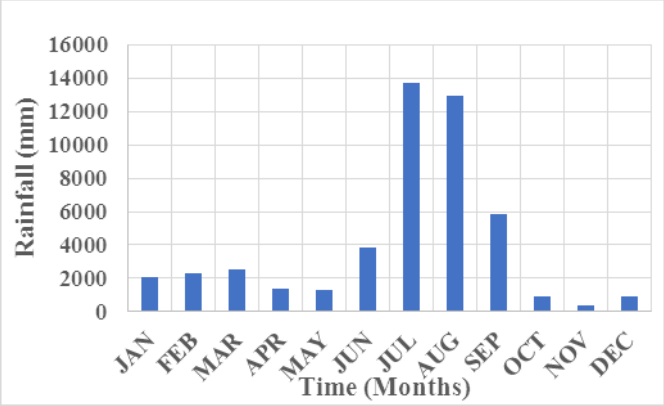



***Figure 8:*** Plot of Monthly Total Rainfall from Year 1931 to Year 2013.

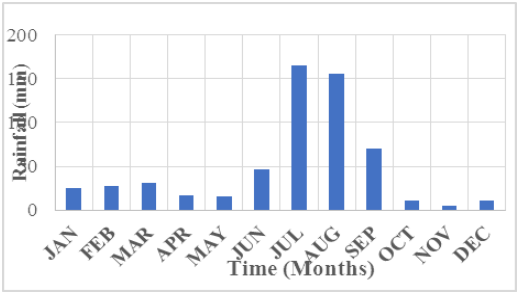



***Figure 9:*** Plot of Monthly Average Rainfall for Duration of 82 Years starting from Year 1931 to Year 2013.

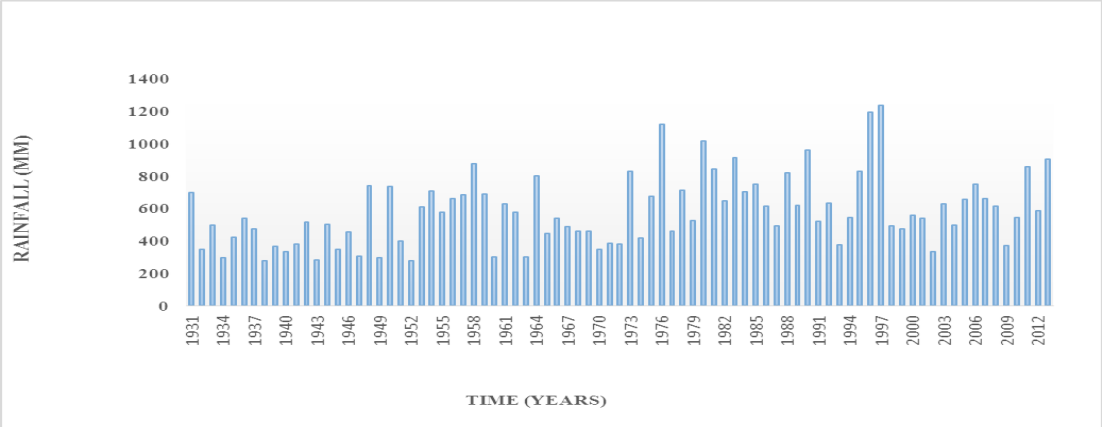



***Figure 10:*** Annual Total Rainfall for a Duration of 82 Years (1931 to 2013).

It is observed from the above Figure 9, that the maximum monthly average rainfall, 164.77, mm occured in July and the minmum average monthly rainfall, 4.47 mm, occured in November.

From the analysis of the annual total rainfall data (Figure 10), it is observed that the maximum annual total rainfall occurred in 1997 (1232.5 mm) and minimum annual total rainfall occurred in 1938 (277.2 mm).

### 4.2.3 Rainfall Trend Analysis

Trend analysis for one day annual maximum rainfall trend, total annual rainy days trend and ten yearly rainy days trend, was done using the Mann-Kendall trend test. It produced the results which are discussed below.

### (i) One Day Annual Maximum Rainfall Trend

Mann-Kendall test was applied on the one day annual maximum rainfall data to find its trend. The results showed that there is no statistically significant trend in one day annual maximum rainfall at selected design significance level of 5%. The trend is shown in the Figure 11.

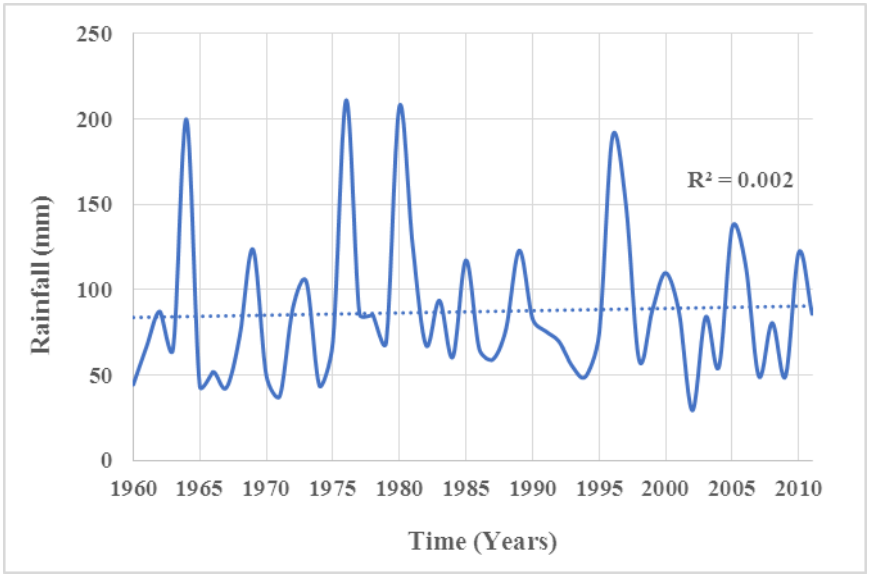



***Figure 11:*** One Day Annual Maximum Rainfall Trend for a Duration of 52 Years Starting from 1960 to 2011.

## (ii) Total Annual Rainy Days Trend

By applying the Mann-Kendall test was also applied on the total annual rainy days data to find their trend. The results showed that there is no statistically significant trend in total annual rainy days from 1960-2011 at selected design significance level of 5%.The total annual rainy days trend is shown in Figure 12.

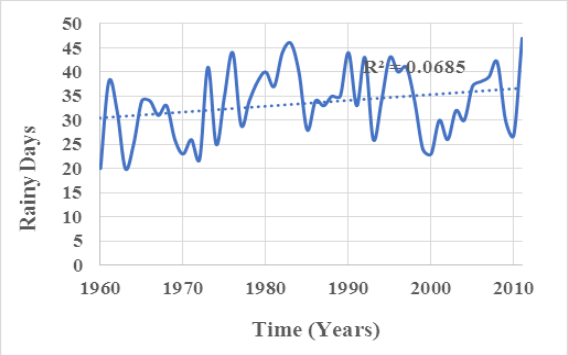



***Figure 12:*** Total Annual Rainy Days Trend

Similarly, the following trends are found in the data of 5 groups, 10 years rainy days in each group, as it is described in Figures 4.10 to 4.14. No statistically significant trend is found from 1960-1969 at significance level of 5%. It is given in the Figure 13.

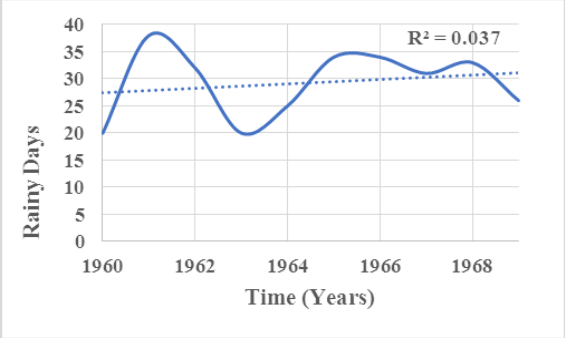



***Figure 13:*** Rainy Days Trend During the Period 1960 to 1969.

There was no statistically significant trend found from 1970-1979 at significance level of 5%. In years 1972, 1974 and 1977, negative trend is detected while in years 1973 and 1976, positive trend is found. But, the overall trend is neither positive, nor negative. This is shown in Figure 14.

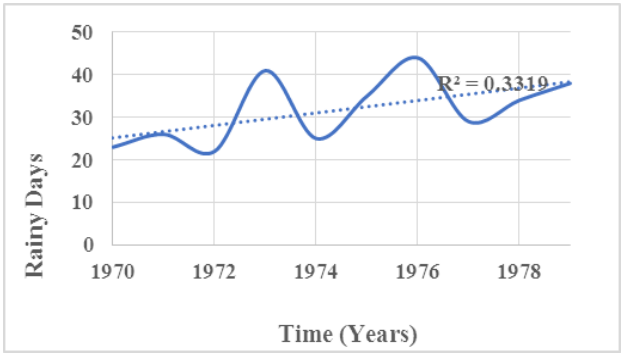



***Figure 14:*** Rainy Days Trend During the Period 1970 to 1979.

There was a statistically significant decreasing trend found for years 1980-1989 at significance level of 5%. Although, a positive trend is found in year 1983, but it is negative in years 1981 and 1985. Therefore the overall trend is negative. It is shown in Figure 15.

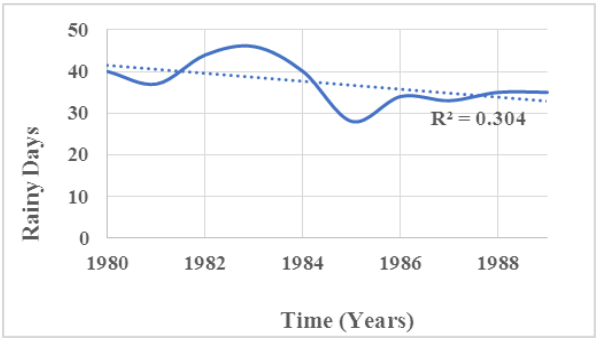



***Figure 15:*** Rainy Days Trend During the Period 1980 to 1989.

There is no statistically significant trend in the rainy days for years 1990-1999, at significance level of 5%. In years 1991 and 1993, the trend is negative and in years 1992, 1995 and 1997, it is positive. But, the overall trend is still neither positive, nor negative. It is shown in the Figure 16.

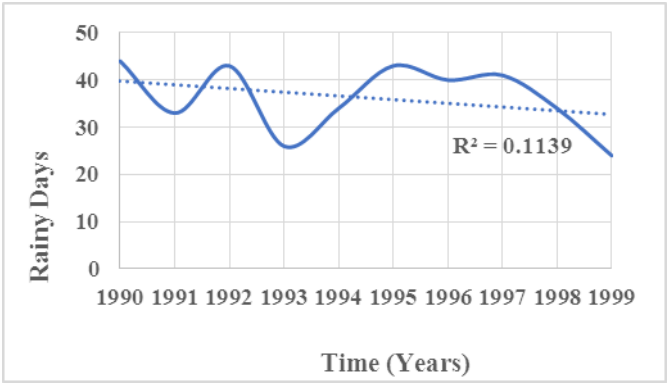



***Figure 16:*** Rainy Days Trend During the Period 1990 to 1999.

There was a statistically significant increasing trend found for years 2000-2009 at significance level of 5%. Although, a negative trend is found in years 2002 and 2004, but it is positive in years 2001,2003, 2005, 2006, 2007 and 2008. Therefore the overall trend is positive. It is shown in Figure 17.

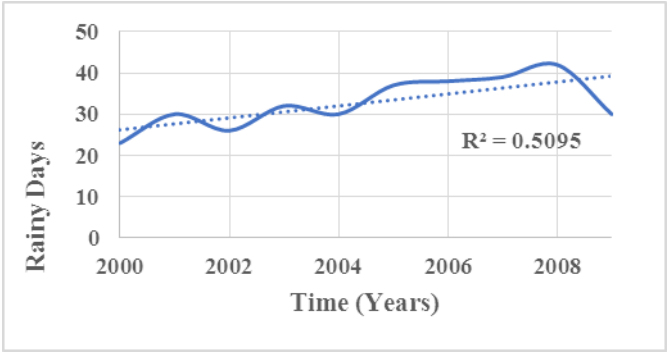



***Figure 17:*** Rainy Days Trend During the Period 2000 to 2009.

### 4.2.4 Extreme Rainfall Event Data Analysis

Extreme rainfall event of one day annual maximum rainfall occurred on 5 September, 2014, at Lahore and on that day the recorded cumulative rainfall was 288 mm. There was a continuous rainfall for about 16 hours on that day. The hyetograph and the mass curve were constructed for this event as given below.

### (i) Hyetograph of Extreme Rainfall Event

The hyetograph of one day maximum annual rainfall event was constructed for rainfall intensity with respect to time. The hyetograph shows the highest rainfall intensity of 47 mm/hour, for one day maximum annual rainfall (MAR) event on 5 September, 2014 at Lahore. Figure 18 shows the hyetograph of one day MAR event.

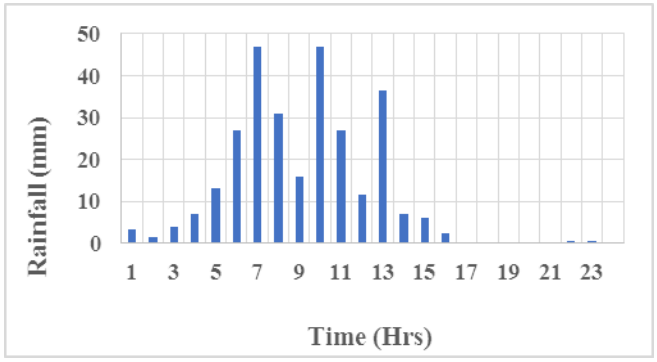



***Figure 18:*** Hyetograph of One Day MAR Event.

### (ii) Mass Curve of Extreme Rainfall Event

Mass curve was constructed by accumulating the rainfall with respect to time. Figure 19 shows the mass curve of 5 September, 2014 rainfall event. It was the maximum rainfall event for the duration of 1961 to 2015 years data for one day annual maximum rainfall at Lahore climate station.

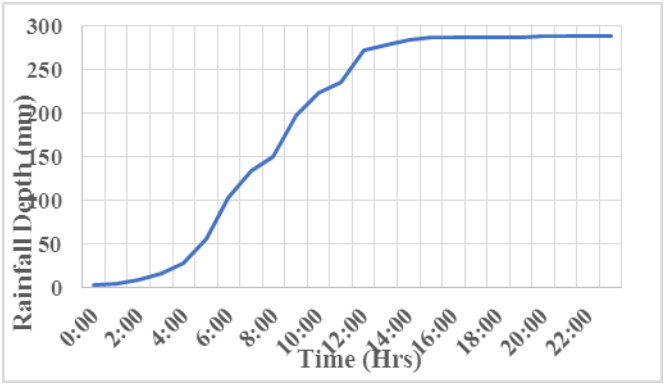



***Figure 19:*** Mass Curve of One Day MAR Event at Lahore.

## 4.3 STATISTICAL ANALYSIS OF EXTREME RAIFALL EVENTS

Statistical analysis was carried out by using daily rainfall data collected from PMD, Lahore for the 52 years period (1960-2011). Annual maximum storm events of each year at the selected station were sorted out and annual series was made on the basis of daily annual maximum storm events. Then rainfall frequency analysis using Gumbel's Extreme Value Type-I distribution, Weibull's formula and Log Pearson's Type-III distribution was carried out to estimate the return period values of daily annual maximum storm events obtained from the

analysis of daily rainfall data as discussed previously. The frequency analysis results are discussed in next sub-section.

### 4.3.1 Rainfall Frequency Analysis

Rainfall data of climatic station at Lahore were selected to determine the design storms for study area at various return periods. Rainfall data was available from years (1961-2011) to project design storm at higher return periods. Frequency analysis of one-day annual maximum rainfall data was carried out using Gumbel's Extreme Value Type-1 distribution and Log Pearson's type-111 distribution. Plotting positions were computed by Weibull's formula. The results of frequency analyses are presented in Table and plotting positions and fitted lines as in Figure 20.

The annual series of the daily annual maximum storm event was used with Extreme Gumbel's distribution to carry out the storm frequency analysis. The frequency factor values of different return periods say 2, 5 10, 20, 25, 50, 100 and 200 were determined using Gumbel's method of frequency analysis.

### (i) Weibull Distribution

Weibull distribution is a continuous probability distribution. It is named after Swedish mathematician Waloddi Weibull. He described it in detail in 1951. This distribution is widely used in reliability engineering and elsewhere due to its versatility and relative simplicity. It was applied on the one day maximum annual rainfall data to assess the exceedance probability for different amounts of rainfall. It was found that there is 98.11 % probability that the rainfall will exceed 29.4 mm every year and 1.88 % probability that it will exceed 211.1 mm every year. It is shown in Figure 20.

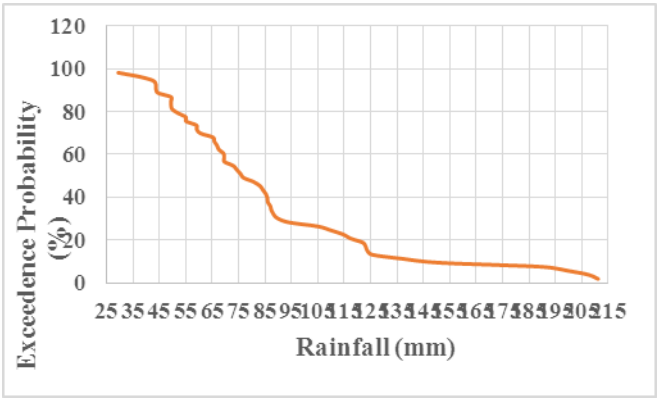



***Figure 20:*** Probability Versus One Day Maximum Annual Rainfall for 52 Years.

### (ii) Gumbel's Extreme Value Type-I Distribution

In probability theory and statistics, the Gumbel distribution (Generalized Extreme Value distribution Type-I) is used to model the distribution of the maximum (or the minimum) of a number of samples of various distributions. To model the minimum value, the negative of the original values is used. This distribution might be used to represent the distribution of the maximum level of a river in a particular year if there was a list of maximum values for the past ten years. It is useful in predicting the chance that an extreme earthquake, rainfall, flood or other natural disaster will occur. The Gumbel distribution was used to assess the one day maximum annual rainfall depths (mm) for different return periods. The results are given in Table 6.

***Table 6:*** Gumbel's Extreme Value Type-1 Distribution

| Gumbel's Extreme Value Type-1 Distribution | |
|---|---|
| **Return Period (Years)** | **Rainfall Depth (mm)** |
| 2 | 80.56 |
| 5 | 122.70 |
| 10 | 150.60 |
| 50 | 212.01 |
| 100 | 237.97 |
| 200 | 262.48 |

Table 6 shows that the one day maximum annual rainfall, computed through Gumbel distribution, is 80.56 mm for return period of two years and it is 262.56 mm for return period of 200 years. While, the one day maximum annual rainfall, computed through the equation that was developed from observed one day maximum annual rainfall data, is 76.9 mm for return period of two years and 294.89 mm for the return period of 200 years. Figure 21 shows the comparison between the observed one day maximum annual rainfall and that of computed using Gumbel distribution, at different return periods.

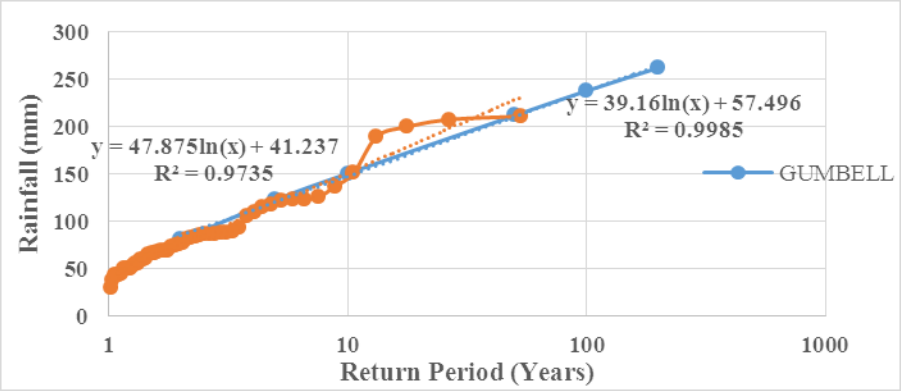



***Figure 21:*** Return Period Versus One Day Maximum Annual Rainfall for 52 Years.

## (iii) Log Pearson's Type-III Distribution

Log Pearson's Type-III distribution is one of the family of continuous probability distributions. It was first published by Karl Pearson in 1895 and subsequently extended by him in 1901 and 1916 in a series of articles on biostatistics. The Log Pearson's Type-III distribution has been one of the most frequently used distributions for hydrologic frequency analyses since the recommendation of the Water Resources Council (1967, 1982) of the United States as to its use as the base method (V.P. Singh,1998).

The Log Pearson's Type-III distribution was used to assess the one day maximum annual rainfall depths (mm) for different return periods. The results are given in the Table 7.

***Table 7:*** Log Pearson's Type-III Distribution.

| **Log Pearson's Type-111 Distribution** | |
|---|---|
| **Return Period (Years)** | **Rainfall Depth (mm)** |
| 2 | 76.81 |
| 5 | 113.92 |
| 10 | 142.11 |
| 25 | 181.97 |
| 50 | 214.86 |
| 100 | 250.59 |
| 200 | 289.55 |

In Table 7, it is found that the one day maximum annual rainfall is 76.81 mm for return period of two years and it is 289.55 mm for return period of 200 years, computed through Log Pearson's Type-III distribution. While, the one day maximum annual rainfall, computed through the equation that was developed from the observed one day maximum annual rainfall data, is 76.9 mm for return period of two years and 294.89 mm for the return period of 200 years as mentioned earlier. Figure 22 shows the comparison between the observed one day maximum annual rainfall and that of computed using Gumbel distribution, at different return periods.

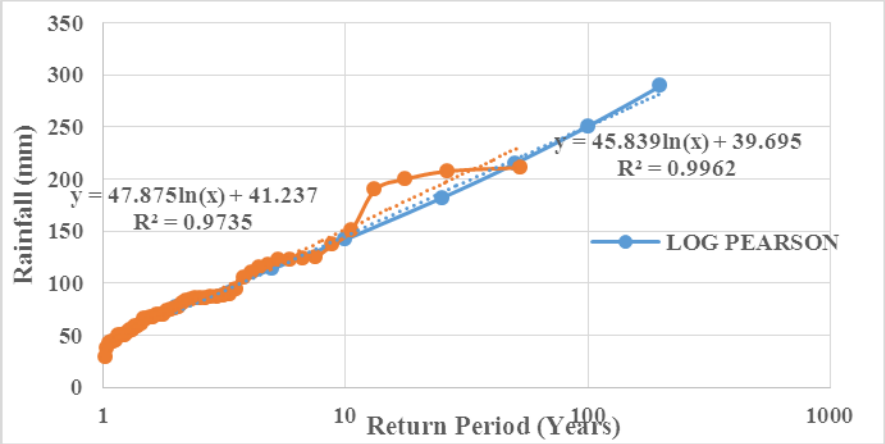



***Figure 22:*** Return Period Versus One Day MAR for 52 Years Data

## 4.4 RAINFALL RUNOFF MODELING USING HEC-HMS MODEL

HEC-HMS (Hydrologic Modeling System) was used for rainfall runoff modeling of selected catchment of Central Drain, Lahore. SCS Curve Number Method was used in HEC-HMS Model as discussed earlier. Different parameters used in this method are given below.

### 4.4.1 Composite Curve Number Estimation

Composite Curve Number of CDCL was calculated by using weighted average method and is given in the Table 8.

***Table 8:*** Calculation of Composite Curve Number for CDCL

| Basin Name | Landuse | Area ($Km^2$) | CN | Weight (AxCN) |
|---|---|---|---|---|
| **Central Drain, Lahore** | Buildings | 7.47 | 98 | 732.06 |
| | Roads | 1.06 | 89 | 94.34 |
| | Vegetation | 0.92 | 69 | 63.48 |
| | Total | 9.45 | | 889.88 |
| **Composite CN = Total Weight / Total Area = 889.88/9.45 = 94.17** | | | | |

### 4.4.2 Basin Parameters

Basin Parameters which were used in HEC-HMS, include area, loss method, transfer method, base flow method, composite curve number, % impervious area and lag time. Table 9 shows the basin parameters used in the HEC-HMS modelling of CDCL.

***Table 9:*** Basin Parameters Description.

| Name | Area ($Km^2$) | Loss Method | Transfer Method | Base flow Method | Composite Curve Number | % Impervious Area | Lag Time (min.) |
|---|---|---|---|---|---|---|---|
| Central Drain Basin | 9.45 | SCS Curve Number | SCS Unit Hydrograph | None | 94.17 | 90 | 72.75 |

### 4.4.3 HEC-HMS Model Simulation

Model was run for hourly basis rainfall data and the simulated results in the graphical and tabular format show the runoff discharge and total runoff volume of the selected urban catchment area of CDCL as given in Figure-23.

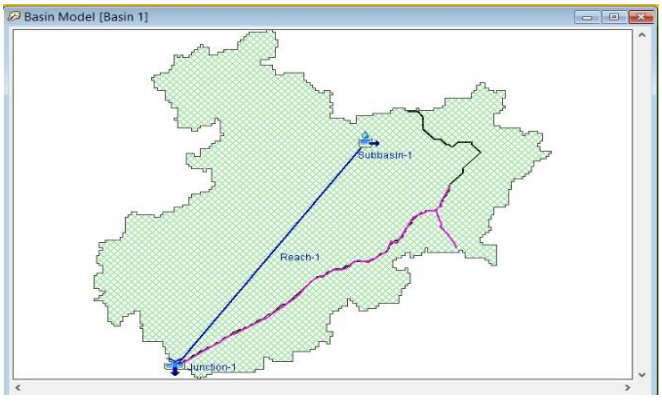



***Figure 23:*** Schematic Diagram of Catchment Area of CDCL.

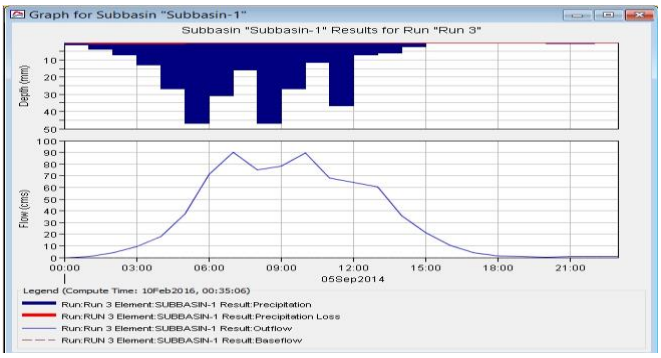



***Figure 24:*** Simulated Rainfall Hydrograph of CDCL.

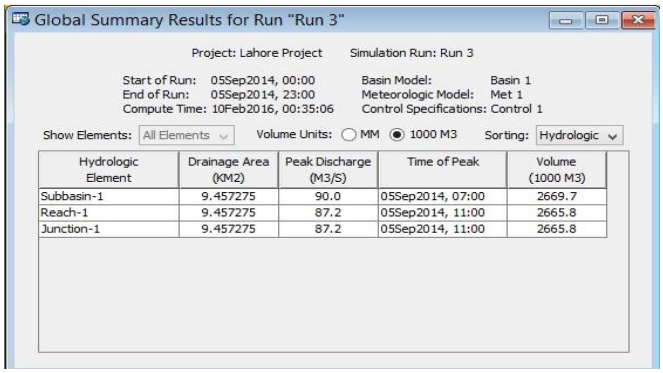



***Figure 25:*** Global Summary Results.

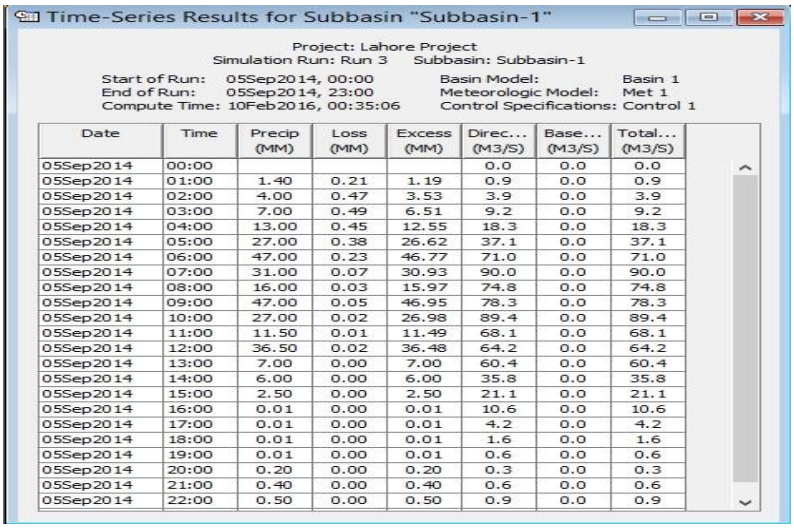



***Figure 26:*** Time Series Results.

The simulated peak discharge calculated through HEC-HMS is 90 $m^3$/sec and the total simulated run-off volume of selected catchment area of CDCL is 2669.7 (1000 $m^3$) as given above (Figures 4.25 and 4.26).

## 4.5 RAINWATER HARVESTING POTENTIAL

The simulated rainfall runoff volume of the selected catchment of the study area for selected design storm is 2669700 $m^3$. Assuming the collection efficiency of 50 % of the total runoff

generated in the catchment for the selected design storm, the Rainwater Harvesting Potential for the selected area was calculated as given below;

Rainwater Harvesting Potential = 2669700*0.50 = 1334850 **$m^3$** = 1.335 **MCM.**

= 1334850000 liters = 1334.85 million liters.

### 4.5.1 Roof Top Rainwater Harvesting Potential

The rooftop rainwater harvesting potential is calculated as given below;

Total built up area = 7.47 $Km^2$ = 7470000 $m^2$

Assuming that 65 % of the total built up area is occupied by constructed buildings, then,

Total Rooftop Area = 4855500 $m^2$

Taking Runoff coefficient as 0.85, total rainwater harvested volume is

Rainwater Volume = Area* Runoff coefficient for Rooftop* Annual Rainfall

= 4855500*0.85*0.623*= 2571230 **$m^3$** = 2.57123 **MCM.**

= 2571230000 liters =2571.23 **million liters.**

For a house of 5 marlas or 126.5 $m^2$, the volume of harvested rainwater is,

Harvested Rainwater Volume = 126.5*0.85*0.623*= 67 **$m^3$**

= 67,000 liters = 0.067 million liters.

### 4.5.2 Availability of Rainwater for Roof Top Rainwater Harvesting

Availability of rooftop rainwater harvesting for different rooftop areas and rainfall was estimated using runoff coefficient as 0.85. The results are given in the Table 10.

***Table 4.9:*** Availability of Rainwater for Roof Top Rainwater Harvesting.

| Rainfall(mm) | 100 | 200 | 300 | 400 | 500 | 600 | 700 | 800 | 900 | 1000 | 1100 | 1200 | 1300 | 1400 | 1500 | 1600 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof top Area (Sq. m) | Harvested Water from Roof top (Cu. m) | | | | | | | | | | | | | | | |
| 20 | 1.7 | 3.4 | 5.1 | 6.8 | 8.5 | **10.2** | 11.9 | 13.6 | 15.3 | 17 | 18.7 | 20.4 | 22.1 | 23.8 | 25.5 | 27.2 |
| 30 | 2.55 | 5.1 | 7.65 | 10.2 | 12.75 | **15.3** | 17.85 | 20.4 | 22.95 | 25.5 | 28.05 | 30.6 | 33.15 | 35.7 | 38.25 | 40.8 |
| 40 | 3.4 | 6.8 | 10.2 | 13.6 | 17 | **20.4** | 23.8 | 27.2 | 30.6 | 34 | 37.4 | 40.8 | 44.2 | 47.6 | 51 | 54.4 |
| 50 | 4.25 | 8.5 | 12.75 | 17 | 21.25 | **25.5** | 29.75 | 34 | 38.25 | 42.5 | 46.75 | 51 | 55.25 | 59.5 | 63.75 | 68 |
| 60 | 5.1 | 10.2 | 15.3 | 20.4 | 25.5 | **30.6** | 35.7 | 40.8 | 45.9 | 51 | 56.1 | 61.2 | 66.3 | 71.4 | 76.5 | 81.6 |
| 70 | 5.95 | 11.9 | 17.85 | 23.8 | 29.75 | **35.7** | 41.65 | 47.6 | 53.55 | 59.5 | 65.45 | 71.4 | 77.35 | 83.3 | 89.25 | 95.2 |
| 80 | 6.8 | 13.6 | 20.4 | 27.2 | 34 | **40.8** | 47.6 | 54.4 | 61.2 | 68 | 74.8 | 81.6 | 88.4 | 95.2 | 102 | 108.8 |
| 90 | 7.65 | 15.3 | 22.95 | 30.6 | 38.25 | **45.9** | 53.55 | 61.2 | 68.85 | 76.5 | 84.15 | 91.8 | 99.45 | 107.1 | 114.75 | 122.4 |
| 100 | 8.5 | 17 | 25.5 | 34 | 42.5 | **51** | 59.5 | 68 | 76.5 | 85 | 93.5 | 102 | 110.5 | 119 | 127.5 | 136 |
| 150 | 12.75 | 25.5 | 38.25 | 51 | 63.75 | **76.5** | 89.25 | 102 | 114.75 | 127.5 | 140.25 | 153 | 165.75 | 178.5 | 191.25 | 204 |
| 200 | 17 | 34 | 51 | 68 | 85 | **102** | 119 | 136 | 153 | 170 | 187 | 204 | 221 | 238 | 255 | 272 |
| 250 | 21.25 | 42.5 | 63.75 | 85 | 106.25 | **127.5** | 148.75 | 170 | 191.25 | 212.5 | 233.75 | 255 | 276.25 | 297.5 | 318.75 | 340 |
| 300 | 25.5 | 51 | 76.5 | 102 | 127.5 | **153** | 178.5 | 204 | 229.5 | 255 | 280.5 | 306 | 331.5 | 357 | 382.5 | 408 |

## 4.6 DESIGN OPTIONS FOR RAIN WATER HARVESTING SYSTEMS

Few design options of rainwater harvesting systems were worked out on the basis of results obtained in this study. Different design options of rainwater harvesting systems include design

option of rooftop rainwater harvesting systems, vegetation or parking lots rainwater harvesting systems and groundwater recharge structures for the harvested rainwater, as discussed below.

### 4.6.1 Design Option for Rooftop Rainwater Harvesting

The design of a roof water harvesting system for a house having a concrete roof with an area of 50 $m^2$ and a family of 5 members is given below,

The average annual rainfall in the study area is 623 mm and the maximum rainfall intensity was 47 mm/hour.

For the roof area of 50 $m^2$ and rainfall of 623 mm, the available water works out as 26.5 cum or 26,500 litres.

Allowing for a consumption of 20 lpcd, this water should be sufficient for 265 days or at least 8.5 months. Height of the tank may be limited to 1.6m with water storage up to 1.4m height. A tank of 3.2 m dia and 1.4m height should be adequate for storing the water. However, an extra 0.2 m height may be provided to allow for fixing overflow pipe and dead storage below the outlet (tap). Thus, a tank having 3.2 m diameter and 1.6m height can be constructed for the purpose.

During heavy rains having intensity of 47 mm/hour, the runoff coefficient was taken as 0.85 (assuming a net loss of 15% of rainfall).

Assuming instant generation of run-off, the maximum rate of runoff from the roof on either side from the roof area of 50 $m^2$ was worked out as,

Roof Area ($m^2$) x Rainfall intensity (m/sec) x Runoff Coefficient

= 50*(47/1000*60*60)*0.85 =0.56 liter/sec.

To carry this peak discharge to the storage tank, a pipe with diameter of 75 mm will serve the purpose.

### 4.6.2 Design Option for Harvesting Surface Runoff

For harvesting surface runoff, the potential of rainwater harvesting from parking lots and roads was estimated for an area of parking lots and roads of 500 $m^2$. The runoff potential was estimated by using runoff coefficient of 0.80 as runoff coefficient of both roads and parking lots ranges from 0.75 to 0.90. It is estimated as given below,

Rainwater Volume = Area* Runoff Coefficient* One Day Maximum Rainfall

Rainwater Volume = 500*0.80*288

= 115.2 $m^3$ = 115200 liters.

The results show that there is a lot of potential for rainwater harvesting from parking lots and roads as this rainwater produces pondage on roads and low lying parking lots. To harvest this rainwater, a recharge well was designed and proposed whose detail is given below,

### 4.6.3 Design Option for Groundwater Recharge System

The best possible design approach was worked out and is given as s sample for RWH structure at the Lakshmi Chowk, Lahore which is a low lying area situated in the study area where rainwater gets ponded after every heavy rainfall particularly in Monsoon.

This design approach works on the principle that surface water runoff flows into an underground tank that is 6 feet deep and 8 feet wide. The main recharge pit is connected with silt trapper which provides the silt free water for recharging. These silt

trapper may be provided on both side of pits. The annular space between the bore hole and the casing pipe is filled with the fine aggregate gravel and developed with a compressor till it gives clear water. This tank is equipped with 2 ft. deep boulders and 1 ft. deep fine gravels. This coarse filter mechanism is used to avoid debris, clay or any other choking material. The rain water runs into the tank though holes provided in to the upper slab 4 -5 inches thick. The horizontal filtration mechanism provide silt and clay free water for recharge the ground water. The water will get drained into groundwater due to head difference. In this way, inflow rate will approximately be equal to recharge rate. According to site and space availability this design can be used underground structure or open to atmosphere. This design is depicted in the figure 27 below:

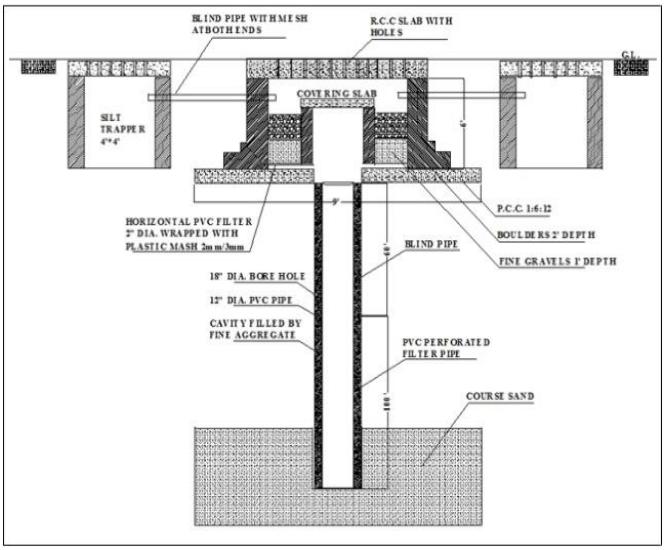



***Figure 27:*** Drawing of Groundwater Recharge System.

## 5. CONCLUSIONS

**1.** The results of rainfall analysis on temporal basis for the duration 1931 to 2013 depicted that there is substantial year to year variability in rainfall pattern and rainfall pattern is generally erratic in nature. The mean monthly rainfall temporal pattern (1931 to 2013) of the selected rain gauge station showed that more than 70 % rainfall occurred in Monsoon Months (Jun-Sep). The intensity of rainfall gradually goes on increasing from June to September and decreases sharply by the month of October. July is the highest rainfall recording month among all the months. The average annual rainfall of selected basin ranges 581 mm (1931-2013) to 623 mm (1960-2011) for different interval duration of years.

**2.** The results of one day maximum annual rainfall and average annual rainfall trends of selected basin using Mann-Kendall trend test, showed that there is no statistically significant trend in rainfall for long term duration (1960-2011) at selected design level of significance of 5%. However there is an increasing trend at 10% level of significance which is usually not used for the design purposes due to more chances to falsely reject the null hypothesis. Similarly no particular trend was found for rainy days. It is concluded that, the overall rainfall trend of the basin is increasing due to positive values of Z – statistics but it is not statistically significant.

**3.** Three landuse classes were classified (Vegetation land, Roads land and Buildings or urban built up land i.e. Residential areas etc.) using data of Landsat 5, 7 and 8. It was

analysed and observed that the area under different landuses of the basin did not change with time due to fully developed old urban study area. The populated area also did not change with time and locations. Generally most of the area of the basin is covered by buildings and other urban structures. The residential area is about 79% of the total area, while the roads area was found to be 11.21% of the total area and rest 9.77% is vegetation area.

**4.** The topographic analysis (DEM) of the basin indicated that elevation difference ranges from 211 to 233 m i.e. the selected area is nearly flat.

**5.** The analysis of the hydrological simulation results on selected extreme event basis indicated that the total 24 hour rainfall of 288 mm, produced 2669700 $m^3$ of surface runoff volume and 87 cms peak flow. Hence, results showed lot of potential of rainwater harvesting in the study area. On the basis of these results, few design options for different rainwater harvesting systems were also prepared for the study area.

These results obtained and expressed in this study can be a comprehensive and effective tool for learning and designing rainwater harvesting systems both for the users and for the professionals in Pakistan. Using the local water demand and rainfall, a rainwater water harvesting method can be designed even for a different location. Along with that, the City Government can use this research as a guideline to calculate the possible amount of supply water conserved by the rainwater harvesting as well as the decrease in load on the ground water to advocate this method to be included in housing policy. These results can also be an effective teaching tool for the fields like Sustainable Architecture, Water Conservation, Green building etc. where alternative technologies like rainwater harvesting are getting more and more importance.

## 6. RECOMMENDATIONS

**1.** This study could be further enhanced for other cities on larger scale for urban watershed water management practices to check the potential and effects of using rainwater harvesting systems to reduce the load on water supply system in the cities.

**2.** High resolution soil and Landsat data could be used to classify the different land uses to distinguish more precisely the different types of buildings and their roof materials to find out more accurately the exact potential and design capacity of roof top rainwater harvesting systems.

**3.** This study further may be elongated to assess the impact of climate change on water availability, demand and water scarcity in future for certain areas.

**4.** More accurate results could be achieved by simulating the model using 5 or 10 minutes interval rainfall intensity data of extreme evets to check the watershed response more clearly visible.

**5.** Runoff measurement instrumentations should be installed on urban drains to measure the drains runoff to obtain better results.

**6.** Steps like promotion, education, formulation of government policies, pilot studies etc. should be taken to address the issues and hindrances in the implementation of RWH systems in the study area.

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**Paper No. 785**

# AWARDING CONTRACT WITHOUT COMPETITIVE BIDDING: IMPLICATIONS

**Engr. M. Mazhar-ul-Islam**

# AWARDING CONTRACT WITHOUT COMPETITIVE BIDDING: IMPLICATIONS

By:

**Engr. M. Mazhar-ul-Islam[1]**

## INTRODUCTION:

Development of a country is dependent on construction of infrastructure and buildings to establish better communication means, generation of electricity, dams, reservoirs, airports, industries which contribute to the economy and standard of living of a developing Nation. For such construction, procurement of goods, works and services are realized through contract.

A contract is a set of conditions at a price agreed between the seller and the buyer following certain pre-determined rules, regulations and laws. Compliance of such regulations not only provide best value of money but also ensure transparency, quality of goods and services; and completion of a project within the stipulated time.

International best practices in modern contracting are the award of contract under competitive bidding. The same practices are also adopted, notified and followed in Pakistan. Pakistan Country regulations envisage in certain cases, award of contract without competition on direct contracting for specialized product and services where no competition is required, for small procurements .However, manipulation and misinterpretation of this provision is made in many cases, generally by the project executing agencies.

Contrary to the above stated best practices, some disturbing trend of manipulative procurement has been witnessed recently in Pakistan in public sector procurement. Consequences of such practices of “awarding contract without competition” are many, eg. transparency is compromised, genuine bidders refrain from participations, delay in project execution and reduction in quality leading to loss in national exchequer. Additionally, in case of awarding contract without competition; the rate of utility charges, toll taxes, per capita loan etc. increases which is ultimately borne by the public in general.

In the latest budget of 2017-18; PSDP allocation alone is Rs. 2.140 trillion coupled with announced $ 46 billion China-Pakistan Economic Corridor (CPEC) funding; and significant provincial allocations, the total construction sector volume is one of the highest in the history of Pakistan. To spend such huge amount through the process of procurement; the relevant regulations should be transparent, equitable to contracting parties and in conformance with International best practices. However, as discussed here below in detail, the Punjab PPRA Rules after several amendments have become inconsistence with similar federal regulations; and also misinterpreted and manipulated. Also in many cases, Foreign Direct Investment (FDI) is attracted by awarding a contract at the terms and conditions of the investors not in the interest of Pakistan. FDI will automatically flow in to Pakistan if law and order condition is normalized resulting into lower risk factors; and malpractices are eliminated and governance is improved.

Recent trend of the governments; Federal as well as the Provincial, to introduce several amendments in their procurement regulations has destroyed the spirit of competitive bidding

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and violated the country regulations; such as, Public Procurement Regulatory Authority (PPRA), Pakistan Engineering Council (PEC), and Competition Commission of Pakistan (CCP). Examples of the relevant provisions for requirement of award of contract by competitive bidding and the manipulated amendments to avoid competition are elaborated here under:

## PROCUREMENT PROCESS:

Engineering Procurement is a process of buying goods, works and services following various steps such as, procurement planning, advertising, prequalifying, issuing bidding/RFP documents; and thereafter, evaluating bids or proposals, announcing the bid evaluation result and finally Award of Contract.

Bidding is a part of procurement process in which Procuring Agency (PA) prepares bidding or RFP documents based on Standard Notified Bidding documents. Bidding documents means "a document or a set of documents prescribing the quality, quantity, characteristics, conditions and procedures of the transactions prior to the actual procurement; and on the basis of which bidders prepare their bids". A standard bidding document (SBDs) contains Invitation for Bids (notice), Instructions to Bidders, Bidding Data and various forms related to bid evaluation and proposals. Instructions to bidders includes stipulations regarding the contents of the bidding documents, eligibility of the bidders, method for preparation of bids, submission of bids, bid opening & evaluation, validity of bids and award of Contract. Bidding data contains forms of the bid such as, special stipulations, price adjustments provisions, bill of quantities, proposed construction schedule; and also various standard forms for bid security, performance security, contract agreement and mobilization advance guarantee. A bidding document also includes General Conditions of contract, Particular Conditions of Contract, Special Provisions, Technical Specifications and Drawings. In summary, a bidding documents or request for proposal document shall contain all the components according to the requirement of the specific projects which will ultimately be the part of contracts. A bidder should know in advance all the conditions that he will encounter on which he will submit his bid.

The most of the bidding as per Pakistan Country Regulations is for **open competitive bidding** in which the prequalification applications, completed bids are opened publically and results are announced in presence of the bidders who choose to witness the process. Close Competitive Bidding is also held in a rare occasion in which only the committee of the PA remain present to witness the bid opening. There are also provisions for direct contracting in certain specific circumstances under the defined criteria in which a bidding document or RFP is given to the specific bidder and who after completing the bid submit to the PA. For such type of direct contracting, bid is awarded after due diligence based on justifications. However, awarding a Contract without the bidding process is illegal which is also upheld in various occasions by the High Courts and Supreme Courts of Pakistan. The primary reasons that without preparing bidding documents, a PA cannot show its intention what to procure and how to procure and also transparency in the procurement cannot be ensured.

## PROCUREMENT RULES AND REGULATIONS:

Rules define the principles and regulations provide detailed mechanism for procurement. Where ever this Paper refers to a rule i.e. that should be with reference to the Punjab Procurement Regulatory Authority (PPRA) Rules; hereinafter will be called PPRA. Nevertheless, this acronym is not in consonance with the acronyms of Federal PPRA.

After enactment of Pakistan Engineering Council (PEC) in 1976; only the PEC Regulations and specific rules were contained in the Standard Bidding Documents and various byelaws and procedures until the year 2002 when World Bank initiated PPRA Ordinance was promulgated. These PEC procurement regulations as Notified and under the jurisdiction of the PEC Act are

applicable for Federal, Provincial and District Governments also. PEC also prepared the FIDIC based bidding documents and notified after the approval of Executive Committee of the National Economic Council (ECNEC), first in the year 2002 and subsequently revised documents in the year 2008. Updating of these documents are about to be approved and notified which will be based on latest versions of the FIDIC MDBs (Multilateral Development Banks) document 2010 .In 2004, Federal PPRA issued the Procurement Rules for Federal Procurements and subsequently in 2010 issued Regulations for Procuring Engineering Consultancy Services. After harmonizing the Federal PPRA Rules and the Regulations with PEC, initially majority of the provincial Governments notified the same Federal PPRA Rules in their respective provinces for making public sector procurement; however, thereafter several revisions have been made. Government of the Punjab has made the significant changes which are discussed here in below.

After the 18th Constitutional Amendment, there remain various controversies regarding the jurisdictions of Federal and the Provincial Governments viz-a-viz maintaining the regulations. Item-6 Part-II of Fourth Schedule of the constitution when read with Article 143 of the Constitution, it is clearly stated that the regulations are the business of Federation. And, a Provincial Government cannot make regulations in conflict with the regulations maintained by the Federation. There are verdicts of the higher judiciary in this regard.

## PERMITTED EXCEPTIONS IN PUNJAB PROCUREMENT RULES:

There are various provisos to make exceptions to the stated rules which are briefly narrated below:

- **Rule-5: International Government commitment –** PPRA stipulates that where there is a conflict between the Country Law and that of the International Commencement in case of procurement, the International commitment of the Government shall prevail. This provision in principle seems to be logical but in practice, the Country jurisdictions in many areas of procurement are compromised. However, PEC from last one decade has coordinated with the funding agencies and most of them have agreed to abide by the Country Laws in case of conflict with other country laws. Accordingly, to harmonize such acceptance by the MDBs, PEC has adopted the Multi Development Banks' FIDIC based (MDBs) procurement documents as Country document by incorporating Pakistani conflicting provisions such as; adherence to Country laws, PEC licensing, eligibility criteria.
- **Rule-13:** Advertisements not required in case of national security breach & protecting intellectual property.
- **Rule-55 (A):** A single complying proposal subject to Rule-35 (Rejection of Bids) may be awarded the contract**.**
- **Rule-64 (A):** A procuring agency may assign their tasks to other procurement agency.

## MISINTERPRETATIONS & MANIPULATIONS OF PPRA RULES:

**Article 143 of the Constitution:** Several judgments and observations of the High Court and the Supreme Court stipulate that provinces cannot maintain conflicting laws for which federation is entrusted under the 18th Constitutional Amendment by abolishing the concurrent list. A close review of Article 143 of the constitution of Pakistan stipulated – "If any provision of an Act of a Provincial Assembly is repugnant to any provision of an Act of Majlis-e-Shoora (Parliament) which Majlis-e-Shoora (Parliament) is competent to enact, then the Act of Majlis-e-Shoora (Parliament), whether passed before or after the Act of the Provincial Assembly shall prevail and the Act of the Provincial Assembly shall, to the extent of the repugnancy, void".

While reading the above Article of 143 in conjunction with Item-6 PART-II of Fourth Schedule of the Constitution, it is abundantly clear that any province cannot make any legislation/make or amend legislation pertaining to the Regulations which are established under the Federal Law. Item-6 stated – "All regulatory authorities under a Federal Law". Whereas, Federal PPRA and PEC Regulations which regulate the procurement of Good, Works and Services under the Federal Law; as such, provinces cannot maintain any conflicting regulations with that of the Federal Regulations. This matter has been observed time and again by the court of law in various litigations.

- **Reshuffling of PPRA Rules:** Initially Government of Punjab notified the same Federal PPRA Rule 2004 in the jurisdiction of Government of Punjab. Thereafter until 2016, there have been at least four amendments to this rule and the sequence and composition of the PPRA Rules have been reshuffled which made it difficult to compare between the Federal and the Provincial Rules. For example, alternative methods of procurement originally were in Rule-42, which is now placed as Rule-59.Nevertheless, a common sense does not allow us to understand what the motive behind this reshuffling is.
- **Chapter –VII: Procurement of Consultancy Services** - This chapter of the PPRA Rule now stipulates provisions for procurement of consultancy services. However, unlike Federal PPRA it did not make the rule exhaustive. Federal PPRA has made the rules for procurement of consultancy service and notified in 2010 as an independent regulation for hiring of consultancy services. In PPRA, things are mix up. For example, requirement of bid security, performance guarantee, retention money, liquidity damages although are for the construction contract as per Federal PPRA, PEC Regulations and international practices but in many cases these instruments are asked in case of consultancy services without differentiating the requirement for construction and consultancy services. Even in the segregated Chapter-VII of Punjab PPRA, if it is understood that they have separated the provisions for procurement of consultancy services under this chapter; nowhere such requirements are stipulated therein.
- **Rule 4: Principles of Procurement** – Under this rule a procuring agency shall ensure procurement in fair and transparent manner; resultantly to bring value for money; also the procurement shall be efficient and economical. On the basis of our analysis of the present form of the PPRA Rules, it could be safely concluded that Rule-4 is often compromised or ignored.
- **Rule 13: Exemptions from Advertisement** – Publication of advertisement is made mandatory under Rule-12.However, under Rule-13 this requirement has been dispensed with, if the procurement is of National Security Concern; and the procurement falls under the Proprietary Intellectual Property which is available from a single source**.** As for the exemption for National security concern is understandable; however, for Proprietary Intellectual Property is neither conceivable nor meet the requirement of bidding. Bidding document must have to be prepared even for procuring from a single source for transparency and value of money. Honorable Chief Minister of Punjab Mr. Shahbaz Sharif during his inauguration of Haveli Bahadur Shah Gas Power Plant in the District Jhang on 07.07.2017 stated that "Awarding Contract without bidding is great offense and the person responsible should be put in Jail".
- **Rule 14 & 48(2):** Response time of 14 and 30 days commencing from the date of publication of the latest advertisement has not been clarified both for prequalification and proposal.Rule-14 when read in conjunction with 48(2) provides that this minimum

response time of 14 days for local bidders and 30 days for international bidders should be considered separately for both prequalification application and submission of proposal.Rule-14(1) is often min-interpreted that submission of proposals does not required separate response time.

- **Rule 15: Frame of Contract –** Globally and under the PEC Regulations, framework contract means standard global documents which are used to prepare project specific documents. Standard Bidding/Contract Documents prepared by PEC are the framework documents. Nevertheless, this substitution in PPRA in 2014 may need careful examination and harmonization with the other Country Laws and globally acceptable definition of framework contract.
- **Rule 16(6) - Annual Prequalification:** In the latest amendment PPRA has amended this rule and given the sole jurisdictions to P & D department for preparation and maintenance of prequalified consultants for use of Line Departments at their option. This practice is against the real spirit of the Federal as well as Provincial regulations, in which case prequalification for the complex and large projects are required to be made on project to project basis. Further, by inviting prequalification by P & D huge amount of evaluation efforts are required within a short span of time which may overlook some of the evaluation criteria for considerations. Again if any consultant left behind to apply, he is to wait for another complete year even though there might be very relevant suitable and doable projects available for them during this period.
- **Rule-21 (3): Direct request for Tender – bid price as sole evaluation criteria**. This provision for making bid price as sole evaluation criteria will be logical only if the tender documents specify baseline criteria that are generally mentioned in a bidding/RFP document. In that case, this procurement is synonymous to Least Cost Method (LCM) of procurement. This method may also refer in direct contracting under Rule-59.
- **Rule 31(3):** Procuring Agencies refer this rule for this newly inserted provision for procuring through direct request in which the bid price will be the sole evaluation criteria. This is similar to the method under rule 45(2) i.e. least cost selection which is specified for selection of Consultants for simple Engineering design or supervision of non-complex work, where the well-established practices and standards exists. However, in rule 31(3) is also applicable for procurement of works without giving any qualification of technical requirements like that of 45(2).According to the international practices, in case of Least Cost Method of selection, there must be technical evaluation to determine the minimum slab of technical score (say 70%) below which no bid will be considered for evaluation and award.
- **Rule 34: Discriminatory and difficult conditions –**procuring agencies often ignore this provision and interpret this rule at their own suitability. Example of such difficult conditions are asking unprecedented higher bid security, annual turnover, even for consultancy bid which is difficult to meet by an ordinary contractor. The PPRA has itself given the interpretation that any conditions that are imposed above the ordinary practices of that trade, manufacturing, construction business or engineering services, to which that particular procurement is related, will be termed as difficult conditions. Although, Rule-8 & 9 of PPRA Rules stress for procurement planning in the stipulated matter but that is not mostly followed by the procuring agencies. They also make the procurement package so huge that a Pakistani contractor cannot provide the bid security to participate in the project. Such huge packaging is against the spirit for developing construction sector in Pakistan.

- **Rule 35**: Rejection of bid to be notified to each bidder and not just hoisting to website. However, most of the procuring agencies put the table for evaluation on the website of PPRA and their own website and do not notify promptly as required under Rule 35(4) of the PPRA Rules.
- **Rule 37:** Announcement of evaluation to be notified to each of the bidders not just hoisting to the website. It has been observed that the bid evaluation report even in its concise form in the form of a table for each of the participating bidders is not notified through the individual letters to the bidders participated rather they hoist it to the PPRA and their own website; whereas, the bidder after submitting the bids wait for months that they would receive the intimation regarding the status of their bids. Even they hoist to the website, it should be intimated promptly to each of a bidder through direct communication that the result could be seen through the following link; but this practice too is not transparent and not in accordance with the spirit of the rule. First of all, the websites are under the control of the procurement agencies which they can manipulate, secondly, not all the bidders to be considered internet efficient and have their capacity of accessing the website.
- **Rule 45 (5) - Direct awarding of contract to Govt. organizations :** Direct award of consultancy contract without competitive bidding has been permitted to the Government owned consultants by an amendment in the original PPRA Rules which is manipulative and conflicting with Principal Country Laws. Pursuant to Rule 45(5), a procuring agency in a complex project and with prior approval of the Government engage, through direct contracting to an organization (consultancy organization) owned or controlled by the Federal or Provincial Governments. This provision is against the spirit of Principles of Procurements enumerated under Rule-4 of the PPRA Rules. This is against the Section 6(3)(h) issued by Pakistan Engineering Council vide S.R.O No 809 (I)/86 dated 24th August, 1986 which stated as under:

  ***"Selection of consulting engineers is done through open and fair competition and no preference is given to any consulting engineer for reason of his being in the public or the private sector".***

  Global best practices also discouraged engaging a Government owned consultancy without open competitive bidding. Peshawar High Court in its order in July, 2017 has prevented engaging a Government sector contractor. This practice is not only injustice of doing business but will also prevent growth of consultancy sector in Pakistan.

  Honorable Chief Minister of Punjab Mr. Shahbaz Sharif during his inauguration of Haveli Bahadur Shah Gas Power Plant in the District Jhang on 07.07.2017 stated that "Awarding Contract without bidding is great offense and the person responsible should be put in Jail".
- **Rule 55(A) - Single Complying Proposal:** Conventionally in Pakistan, the word 'Bid' is used for goods and works and 'proposal' is used for submission of consultancy proposals. PEC also clearly identified these in their Act & Byelaws to this distinct terminology for consultancy, supply and works. However, the word bids and proposals are used throughout the PPRA Rules for procurement of goods, works and services. The Rule 55(A) is outside Chapter-VII which PPRA has designated as for procurement of consultancy services. Nevertheless, accepting single complying proposal, although has a benefit for speedy procurement; however, it is likely subjected to gross manipulation. It was observed in many cases that the Employer in order to accommodate award to a favorable contractor or consultant just disqualify the competitors to clear their ways for

award. In some cases, single compliance bids for award of the contract can be considered; however, if in the first attempt there remain only one competitor then it should have been re-advertised quickly or Rule 35(5) should be invoked that may ask the bid /proposals from all the bidders / applicants with some adjustments in the RFP. However, all this is dependent on the intension of the procuring agencies that whether they are eager to have a transparent award to the right party or not.

- **Rule 53 & 57: Negotiations**: This provision of negotiations should be read in conjunction with Rule-57 (Limitations on Negotiations).Rule-53 allows negotiating with the highest ranked bidder in case of consultancy; among others, the contract price also. Rule-57 refers to negotiation in case of procurement of goods. Therefore, this mix up is confusing as Rule-53 deals with the consultancy proposals but the Rule-57 deals with the procurement of goods.

  Nevertheless, this provision of 'negotiating on contract price' is against the Federal Laws including PEC regulations, funding agencies and International best practices. In a competitive bidding, across the board discount of contract price is prohibited. However, during the negotiation based on the likely adjustments in the RFP or scope of services, the contract price could be adjusted accordingly. Contrary to this above spirit of Negotiations; procuring agencies often used to disqualify the competent and deserving bidders and manipulate to award the contract either to its liked bidder or the low quality bidder. This provision is often used as a weapon to disqualify the competent bidder and allow a lower quality bidder to award the contract.

- **Rule 58:** This rule stipulates that the procuring agency shall keep all information regarding bid evaluation confidential until the time of the announcement of bid evaluation report. This rule implies that announcement of bid evaluation report is the end of the bidding process; as such, when the result of the both technical and financial bids will be announced and publicized, the process shall not be further cancelled until it is essential that the bidder could not negotiate and submit the post-bid prerequisites stipulated in the bidding documents. After the publication of the result, process of award must be completed as per stipulations in the bidding documents.

- **Rule 59 - Alternative method of procurements:** For petty purchases, alternative methods of procurement are logical; however, the ceiling for such purchases should have to be increased because of inflation.

- **Rule 59 (c) (iv):** This rule is used for Direct Contracting which is often mis-interpreted to prevent allowing Variation Order (VO) in case of construction contract or continuity of an existing consultancy contract which are permitted under Federal PPRA and PEC Regulations besides the International best practices. The intent of this rule is for procurement of goods particularly spare parts or Direct Procurement from other sources where the prices are fixed by the appropriate authority. This Rule is also used in case of 'emergency' for such procurement. A close review of the Clause 59(c) prima-fascia stipulates that the rule of 59(c) (iv) allows the repeat order not exceeding 15% of the original procurement.

- **Rule 59 (c) (v) – Emergency Procurement:** A procuring agency can procure goods, works or services directly by declaring an emergency from an 'authority'. Under Rule-2(e) - 'Authority' means the Punjab Procurement Regulatory Authority; and under Rule 2(j) 'competent authority' means the head of the procuring agency or any other officer authorized to act as competent authority; under Rule 2(q) 'emergency' means natural calamity, disaster, accident, war and operational emergency which may give rise to an

abnormal situation requiring prompt and immediate action to limit or avoid damage to person, property or the environment. Forgoing three definitions, if read in conjunctions then these provisions can be used only in case of a situation such as, force-majeure conditions. Declaring such emergency by an authority must have to be replaced with 'competent authority' as defined under Rule 2(j) to avoid confusions. Generally, a Chief Executive of a public sector organization i.e. Secretary of a Govt. department or Board of Directors of a corporation should delegate such powers to the Chief Engineer or Chief Executive respectively to make legal provisions complied with. Nevertheless, this rule is often misused to avoid competitive bidding for the works like directive of the Government Department for arrangement of inauguration of a project by the Government Department or corporation.

- **Rule 59 (d) – Negotiated Tendering:** This rule is randomly used to procure goods, works or services without prior publication of procurement notification by taking the self-interpreted plea that the procurement is for research or artistic reasons. Even these factors could be satisfied with the competitive bidding.
- **Rule 59 (d) (iii) – Extreme Urgency:** Urgency as defined under Rule 2(a)(d) means a limited timeline for the accomplishment of procurement which cannot be met through open and limited bidding method. PPRA Rules did not define the extreme urgency. The procuring agencies to avoid competitive bidding, manipulate this provision invoking this rule to make direct procurement. Although invoking this rule is subject to the approval of the Provincial Cabinet under Rule 59(d)(iv); however, either such approval is not at all referred to the provincial cabinet or even such approval is taken with retrospective effect after the decision of going ahead with direct procurement by the procuring agency. Such rule is not at all desirable for a procuring agency that is to make the independent regulations to all the public sectors including the Government.
- **Rule 60 – Unsolicited Proposal:** This rule is introduced on March 11, 2014 for the EPC procurement for the cost above 1,000 million. Such procurement under Rule 60(a)(b) prescribed for open competition; however, has exempted the 'initiator' of the unsolicited proposal from the bidding process. Under Rule 60(e), the Initiator of the proposal shall be given first right of refusal, if he does not emerge as a lowest bidder. In such right of refusal, the 'initiator' will be permitted 5% additional weight ages of the proposal. Such a rule is amusing and is not comprehensible for a transparent bidding when read Rule-4.It is also injustice to the participants in the open competition that they have very less chance to win the project and the Initiator will be awarded the project by giving preference of 5%; meaning thereby the other participants shall simply waste their time, money and efforts. Notwithstanding, above rule stipulates that the conceptual part of the project must have to understood and developed by the procuring agency on the basis of which a suitable mode of contracting could be invited not limiting to EPC mode. Without following such method, provisions in Rule-60 is simply manipulative and injustice to the bidders.
- **Rule 61(3) - Exemption**: Rule-61(3) stipulates that procuring agency while inviting competitive bidding for goods exceeding 1 million rupees; the public sector manufacturing unit participating in the bid can secure the contract award by giving matching price of the lowest bid within three working days of opening of the bids. This provision has been made mandatory by the introduction of the Rule-61(1) that nobody should 'relax' the application of these rules for procurement of services. Such provisions in the PPRA Rules is contradictory itself to its intent.

- **Rule-61 (2), (3) & (4)** are clear that such procurement is for goods including the equipment. However, reference to procurement of services under rule 61(1) is not understandable because service are not manufactured but rendered as an intellectual input.

## IMPLICATIONS OF AWARD WITHOUT COMPETITIVE BIDDING

From the above discussions, it could be concluded that misinterpretation, manipulation, deficient provisions, noncompliance with the federal regulations are synonymous to award of contract without competitive bidding; although theoretically such practices are followed.

- **Violates relevant country laws such as PPRA, PEC & CCP**

  As has been discussed in the foregoing paragraphs that provincial rules for procurement of the provincial Governments should not be conflicting with the similar Federal Laws and regulations such as, Federal PPRA, PEC & Competition Commission of Pakistan. In PEC regulations for consultancy under Section6 (2)(h), it is stated that "*selection of consulting engineers is done through open and fair competition and no preference is given to any consulting engineer for reason of his being in the public or private sector"*.

  Competition Commission Pakistan also strictly promotes competition. As such, few of the PPRA Rules as discussed above not only violate the similar Federal Laws but also become discriminatory and unjust for the competitors.

- **Genuine bidders refrain from participation**

  If the International or National genuine bidders observe such practices as stipulated in the PPRA Rules; they become reluctant in participation; as they tend to believe that the projects will be awarded to the targeted bidder.

- **Time line and quality are compromised**

  A contract awarded at right cost with equitable conditions is the guarantee to ensure quality and completion within stipulated time. Whereas, as discussed above under PPRA Rules, there are tendency to reduce the cost and imposing one sided conditions biased to the employer leading to delay in project and lesser quality of work and services.

- **Loss of investors' confidence.**

  Consequent to the various factors as discussed above within Punjab PPRA Rules; and also use of one sided bidding documents having no transparency, investors are reluctant to invest in this region.

- **Promote malpractices**

  It is universally true that a tougher rule is subject to manipulation by the procuring agencies which promote the malpractices. A simple rule understandable to all the parties in conformances to the International practices can be a guarantee to minimize the malpractices by the procuring agencies.

- **Increase in claims.**

  As discussed above, with contradictory and confusing rules, more claims are likely to be submitted by the contractor and the consultants.

- **Loss in national exchequer**

  Contractors do not pay from their own pocket but reduce the quality and increase the claims to compensate his loss which ultimately is the loss in National Exchequer.

- **Destroy moral character of the Nation**

  Procuring agencies by using the ambiguous rules, not only cause financial loss to the national exchequer but also degenerate the morality of the individual person dealing with such procurement and contract whether it may be the procuring agencies, contractors or consultants.

**Paper No. 786**

# IMPACT OF SOIL CHARACTERISTICS ON FAILURE OF FLOOD BUNDS- CASE STUDY FROM PUNJAB PAKISTAN

**Ghulam Zakir Hassan, Ghulam Shabir, Adnan Hassan, M. Shahzad Abdul Rahim**

# IMPACT OF SOIL CHARACTERISTICS ON FAILURE OF FLOOD BUNDS- CASE STUDY FROM PUNJAB PAKISTAN

By

**Ghulam Zakir Hassan[1], Ghulam Shabir[2], Adnan Hassan[3], M. Shahzad Abdul Rahim[3]**

## Abstract

Flood bunds are earthen embankments constructed along rivers to control the floods. Pakistan which lies in the Indus Basin has been facing severe threats and losses from the floods since histories. About 6807km length of flood embankment has been constructed to safeguard against the floods in the country. Punjab Province has been severely hit by heavy floods causing severe loss to human lives, infrastructure, crops and livestock etc. A study was carried out to evaluate the "health" of flood bunds by investigating their geotechnical, geometrical, geochemical and hydraulic properties to ascertain strength against breaching during floods. A length of about 731 km of Chenab River was selected and investigations were carried out on 78 sites of 38 flood bunds which were found critical during flood 2014.

Different field activities including density test, measurement of geometrical parameters, collection of undisturbed/disturbed soil samples, physical inspections and collection of ground water samples from bore holes were performed. Laboratory tests including soil texture, Atterberg's limits, compaction, unconfined compressive strength, permeability, Direct Shear etc. were performed in laboratories of Irrigation Research Institute (Irrigation Department Lahore, Punjab, Pakistan).

A River Embankment Breaching Vulnerability Index (REBVI) was derived to delineate the risk of vulnerability of the embankment. Cluster analysis was performed based on vulnerability index for each indicator.

On the basis of REBVI and Cluster analysis it was concluded that 11 out of 78 sites are representing high potential to prevent embankment breaching while 35 sites low potential to prevent embankment breaching point indicating that the consistence of the embankment is very high and high in 11 and 35 flood bund sites respectively. The consistence of the embankment are medium, low and very low at remaining 16, 12 and 4 flood bund sites due to moderate vulnerability, High probability of embankment breaching and very high vulnerability respectively.

## 1. INTRODUCTION

Flood bunds or levees are the earthen hydraulic structures which are constructed around the rivers to control the flood water to avoid the damages to the infrastructures and populations. Precisely flood bunds as defined "man-made structures, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control, or divert the flow of water so as to provide protection from flooding". These are most of the time man made construction, which have been developed from time to time.

Nature has blessed Pakistan with plenty of surface water which originates from Himalayans in the north and flows toward Arabian Sea through River Indus and its tributaries. A number of barrages and head works have constructed on different rivers, mostly during British rule in the

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Subcontinent. Major purpose of these structures is to divert the river water to the canals for irrigating the fertile lands of Indus plane. These structures on the rivers cause the hindrance to the natural regime of the rivers which lead to the heavy floods, especially during monsoon season owing to heavy rainfall particularly in the upper Indus basin. To control the floods and keep the river waters within limits earthen dykes have been constructed from time to time and again along the rivers in Pakistan. At present about 6803 km flood bunds are existing on different rivers in Pakistan out of which, 3332 km fall in Punjab. Summary of flood bunds in Punjab is given in **Table 1.** Unfortunately, less attention is paid to geo-technical, geo-hydrological, and other research and investigations during construction of these structures due to which, the failure of flood bund occurs causing heavy loss to infrastructure, crops, livestock and human lives.

The geotechnical inspection and assessment of flood defence embankments were studied by **Dyer et al** (2007), in which they discussed about the geotechnical characteristics and failure of flood embankments. The failure was due to seasonal wetting and drying with potential cracking on the crest and side slopes. This can accelerate the rate of weathering and softening of the embankment fill and creates the possibility that water-filled cracks will lower the embankment's ability to resist the flood.

Different causes of failure related to embankment earthwork are seepage, piping erosion and slope instability. Reason of failure is broadly divided into two modes depending on where the failure generates. First mode is founding strata (settlement, sliding, seepage and piping and uplift pressure) and second mode is embankment structure (slope instability, internal seepage, erosion of outward face and toe) **Morris et al (2007**).

Failures in embankments of Irrigation canals in Sindh were overtopping, internal erosion, structural defects and piping and those were found out during study conducted by **Bhanbhro et al (2014)**. Internal erosion in embankments can be originated by several factors such as backward erosion, leakage because of hydraulic fracturing, slope instability, high permeability region due to improper compaction, internal instability, hydraulic gradient, soil type and degree of saturation. Piping can be originated when hydraulic gradient of seepage is greater than hydraulic gradient of that soil. Burrowing animals like rats, earth worms burrow and dug the tunnels and holes which cause piping. Sand boils was phenomenon related to piping and developed at downstream side of the embankments, which lead to failures due to sudden unexpected water levels. Overtopping of the embankments was due to several reasons, like erosion of top surface of embankment and high flood levels in the canal. Frequent drying and wetting of embankment cause cracks in its soil which was due to the difference of water levels in canals.

Reasons of failures of levees along the lower Mississippi River were overtopping, scouring and under seepage **(Rogers J.D)**. Levees were overtopped where they experienced differential settlement, generally underlain by soft soil. Majorly three types of failure of levees/Embankments include force of water pushes out the weakened section of embankment, water overflows the banks, eroding rift into gaping hole and earth in levee become so saturated, water springs out of it like an artesian well.

(**Morris et al (2007)** prepared a report on management of flood embankment consisting of four separate parts, Part A: Function and management of flood embankments, Part B: Performance and characterization of flood embankments, Part C: Risk and risk management and Part D: Good practice references. In part B of the report, common hazards and causes of failure of flood embankment were described to include zones of weak and highly permeable material causing slipping or seepage, reduction of crest level, local seepage path at junctions with other structure, settlement due to consolidation of soft founding strata, localized overtopping, erosion

of outward face due to overtopping and inward face due to wave action, seepage and piping and shallow slope instability.

Characterization and geotechnical properties of Penang residual soils with emphasis on landslides were studied by **F. Ahmad et al (2006).** The attempt was made to characterize the larger granitic residual soil of Penang Island by discussing nature, structural features, engineering behavior and field properties of soil sample extracted from 8 sites. Similar tests like natural moisture content, liquid limit, plastic limit, specific gravity, maximum dry density and shear strength test were used for the characterization of soil of Penang Island.

**Jia Yafei** conducted a study to develop the numerical model for simulating overtopping and embankment breaching process. The key physical-empirical dam breaching mechanism of earthen embankment through WinDAM model was used to achieve the goal and homogeneous embankment constructed from cohesive soil materials of simple cross section was assumed with general flow conditions were utilized. Following process was considered:

- Frictional erosion on embankment top segment
- Head cut erosion and migration
- Channel widening

**Hanson et al. 2001** clarified that erosion to the embankment made of cohesive soil was related linearly to an erodibility parameter and the excessive shear stress.

**Hossain et al (2010)** carried out a study by recording the physical and mechanical properties, geotechnical characteristics and stability analysis of failed banks of Jumna and Padma Rivers in Bangladesh. Physical and geotechnical characteristics were evaluated by conducting particle size analysis, density, liquid limit, plastic limit, compaction, consolidation, permeability, direct shear test and unconfined compressive strength tests in accordance with Japanese industrial standards (JIS) and Japanese geotechnical standards (JGS). They also concluded that permeability of soil was high with other low strength properties and bank failure mechanism was associated with the formation of tension cracks behind the face.

**Pierce at al (2011)** conducted a research study to develop the South Carolina Department of Transportation Geotechnical material database for embankment design and construction**.** Geotechnical information was carried out by 197 borrow pits across the state of South Carolina and data was limited to soil description that often include USCS, AASHTO soil classification. Tests for physical properties included visual manual identification, moisture content, specific gravity, particle size distribution, liquid limit, plastic limit and classification. Tests for mechanical properties included standard proctor, compaction, direct shear and triaxial compression.

Major types of flood bunds include Homogeneous bunds, Zoned bunds and Diaphragm Type bunds. Bunds (also called tetras) are small barriers to runoff coming from external catchments (and possibly to a field where crops are to be grown). Bunds slow down water sheet flow on the ground surface and encourage infiltration (groundwater recharge) and soil moisture. Flood bunds are provided with a particular size and shape to enable them withstand the anticipated high flood water levels with a free board of 3 ft. to 7 ft. Some peak floods in Punjab in different rivers have been shown in **Figure 1.**

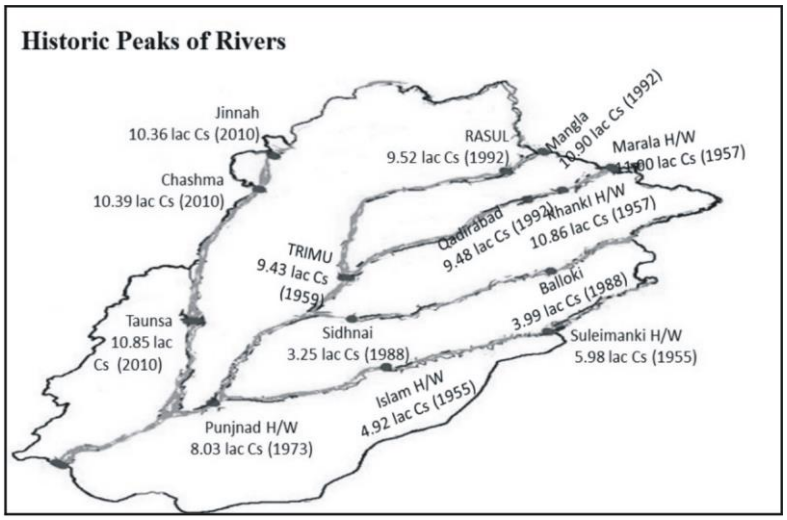



***Figure 1:*** Historic peak discharges at different irrigation structures of rivers. **PDRP (2014)**

***Table 1:*** Salient features of Rivers and flood bunds in Punjab

| River | Length of River (K.M.) | Length of Rivers (miles) | Catchment Area in the Mountains (Sq. mile) | Length of Bunds (miles) | No. of Spurs / Studs |
|---|---|---|---|---|---|
| Indus | 547 | 342 | 1,18,400 | 811 | 131 |
| Jhelum | 363 | 227 | 12,445 | 155 | 43 |
| Chenab | 731 | 457 | 11,399 | 1330 | 309 |
| Ravi | 694 | 434 | 3,562 | 630 | 127 |
| Sutlej | 515 | 322 | 30,550 | 406 | 30 |
| Total: | 2850 | 1782 | 1,76,356 | 3332 | 640 |

Source = Punjab Irrigation Department

## 2. MATERIALS AND METHODS

### 2.1 Study Area

A study on 38 flood bunds of River Chenab was carried out to evaluate the “health” of flood bunds by investigating their geotechnical, geometrical, geochemical and hydraulic properties to ascertain their strength against breaching during floods.

The Chenab River is a major river of India and Pakistan. It forms in the upper Himalayas in the Lahaul and Spiti district of Himachal Pradesh, India, and flows through the Jammu region of Jammu and Kashmir into the plains of the Punjab, Pakistan. The River Chenab was called Iskmati or Ashkini in Vedic times by Indians. The river is generally considered to be the second healthiest river of Pakistan after River Indus. It originates from Bara lacha 31°11′20″N 72°11′57″E which is high mountain connecting Lahaul district in Himachal Pradesh to Ladakh in Jammu and Kashmir and joined by River Jehlum at Trimmu and then by River Ravi, confluence with Sutlej to form Panjnad River in Bahawalpur district, Punjab, Pakistan 29°20′57″N 71°1′41″E. It then forms the Panjnad by joining River Sutlej near Uch Sharif before falling into the River Indus at Mithankot. Its total Length is 960 km (597ml). Average discharge is 800.6m3/s (28,273 ft3/s).This river forms at the confluence of streams Bhaga & Chandra which

join at a place called Tandi in Occupied Jammu & Kashmir state. Upper most part is snow covered and forms the North East part of Himachal Pradesh From Tandi to Akhnur the river traverses through high mountains.Its length is 770 miles/1,232 Km. its catchment area is 26,100 Sq.miles/41,760 Sq.km.The River enters Pakistan a little over Head Marala with very sharp changes in slope (130 ft/mile above Tandi reduced to 2 ft/mile close to Trimmu). Its tributary Rivers include twelve major tributaries (6 each in occupied Jammu & Kashmir and Pakistan). Doara, Dowara, Halsi, Bhimber, Palku and Budhi join close to Marala. There are mainly Marala, Khanki, Qadirabad, Trimmu, Punjnad barrage on this river. The flood bund sites are shown in Fig 2.

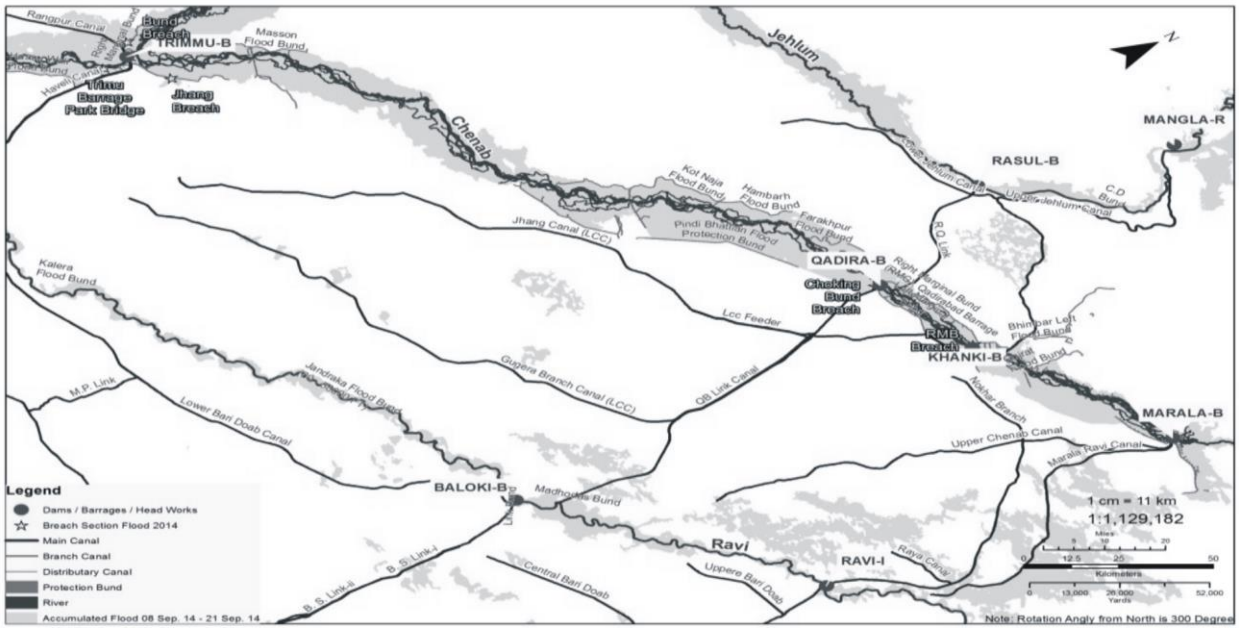



***Figure 2:*** Flood bund sites of River Chenab

## 2.2 Field Survey and investigations

Field surveys were carried out for observing the physical condition of the flood bunds. During the field investigation, disturbed and undisturbed soil samples were collected from the proposed sites of embankments for field and laboratory testing. In order to know the nature of bank materials, the soil samples were collected directly from the broken parts of the flood embankments.

A total of 78 sites were selected to investigate the geotechnical attributes of earthen embankment. However, the samples were collected from three borehole locations (top of bund, country side and river side) from each site. Laboratory testing was performed to determine the index properties of the soil samples in order to classify the soil materials. The laboratory tests were performed in laboratories of **Irrigation Research Institute** (**Irrigation Department Lahore, Punjab, Pakistan**). The testing measures/procedure were in accordance with **American Standard for Testing Materials** (**ASTM, 1961)** and **British Standard 1377( 1990)**.

The tests include Particle size analysis through Sieves, Bulk density, Liquid limit (%) and Plastic limit (%), Modified AASHTO compaction test and Permeability test. The collected soil samples were classified based on **ASTM D 421- 58(1965) & 422- 63 (1963)** and **Jumikis, A. R. (1962)**. More detailed investigations were undertaken including plasticity index (PI) followed by **ASTM D 423-61 (1961) and Burmister (1949),** Modified AASHTO compaction test to measure the degree of compaction of flood bund by **(ASTM D1557-70 / AASHTO T180** and **Qureshi M. S.**

**and Akbar A (1995),** Unconfined Compressive Strength (UCS) test by **ASTM D2166/ D2166M** and **(Manojit, Gouri and Kumar (2012) and Qureshi M. S. and Akbar A( 1995)**, Permeability test (to find out ease with which water flows through soil and/or rocks) by **ASTM D 2434** and **Jumikis, A. R. (1962) and** Direct shear test (to find out the shear strength parameters like C and ø) by using **ASTM D 3080** and **Qureshi M. S. and Akbar A (1995).** Detailed observations on embankment structures were measured by using different appropriate field techniques.

Geochemical analysis (to find out pH, Organic Matter, sulphates, and chlorides, Ec etc.) of soil and water samples collected from flood bunds were carried out using procedure followed by **BS 1377(1990).**

To investigate the geometry of embankments/flood bunds, detailed observations including bank top height, base width and bank slope were recorded. The design cross sections were collected from concerned Sub Divisions. Existing cross sections of flood bunds (top width, height and slope of bunds at River and Country side) were observed at site and then compared with the designed parameters.

## 2.3 Lab Testing

The samples collected in field were analyzed in lab and following tests were performed:

| Sr. No | Test/Investigation |
|---|---|
| 1 | Grain Size Distribution (GSD)/Sieve Analysis |
| 2 | Atterberg Limits (Liquid and Plastic Limits, Plasticity Index) |
| 3 | Modified AASHTO compaction |
| 4 | Direct Shear Test |
| 5 | Unconfined Compressive Strength |
| 6 | Permeability Test |
| 7 | Chemical Analysis of soil sample |
| 8 | Chemical Analysis of groundwater sample |
| 9 | Preparation of Bore logs and Gradation Curves |

## 3. RESULTS AND DISCUSSION

Field density test, measurement of geometrical parameters, collection of undisturbed/disturbed soil samples, physical inspections and ground water samples from bore holes were collected during field investigation. The results of particle size analysis (soil gradation/texture), bulk density, Atterberg's limits, Modified AASHTO compaction, unconfined compressive strength, permeability, angle of internal friction etc. were achieved after performing lab tests in laboratories of Irrigation Research Institute (Irrigation Department Lahore, Punjab, Pakistan). On the basis of results achieved during investigations from field and performing different tests in labs, A River Embankment Breaching Vulnerability Index was derived.

A River Embankment Breaching Vulnerability Index (REBVI) was derived based on weightings of bank material to delineate the risk of vulnerability of the embankment. Cluster analysis was performed based on vulnerability index for each indicator.

Transformation of absolute values into weighted values (W) for sites of flood bund is shown in Table2.

Methodological Approach of River Embankment Breaching Vulnerability Index is shown in **Fig 3** as below

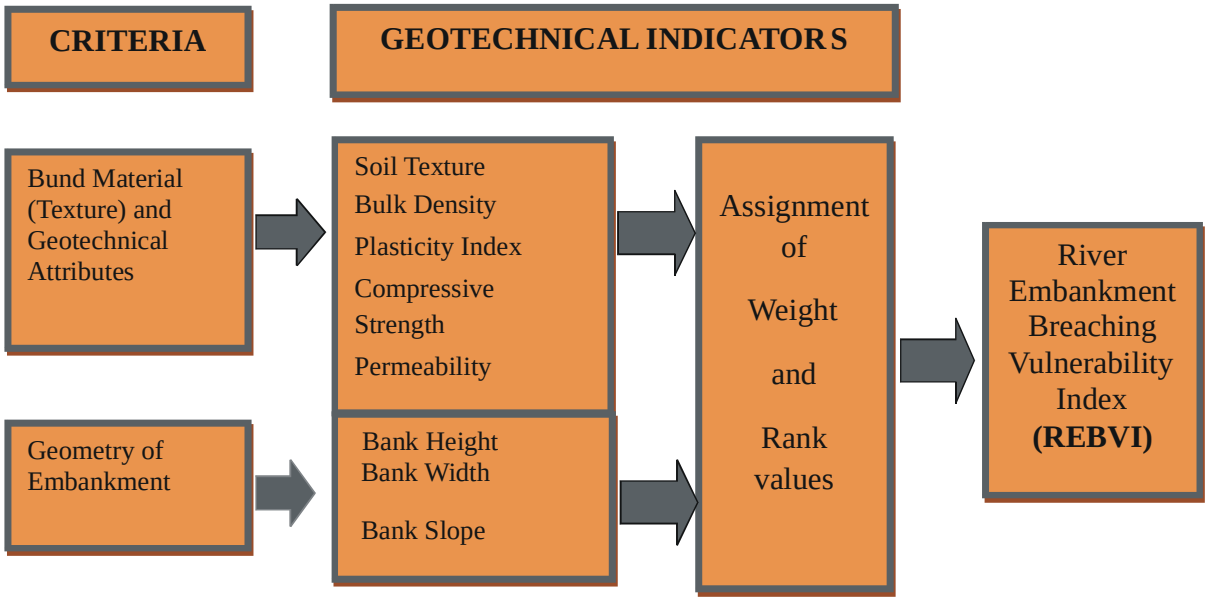



**Table 2:** Transformation of absolute values into weighted values (W)

| Weight Value (W) | Texture | Compaction (%) | Cross-sections | Plasticity Index | Permeability (cm/sec) | Unconfined Compressive Strength (Kg/cm$^2$) | Angle of Internal Friction (Degrees) |
|---|---|---|---|---|---|---|---|
| 0 | Silty Clay | High (>90) | According to Design | Medium Plastic (10-20) | Impervious <0.0000001 | Stiff (1-2) | Very Dense 40-43 |
| 1 | Clayey Silt | Medium (80-90) | Over Design | Low Plastic (5-10) | Very Low 0.00001-0.0000001 | Medium (0.5-1) | Dense 35-40 |
| 2 | Sandy Clay | Low (<80) | Under Design | Slightly Plastic (1-5) | Low 0.001 - 0.00001 | Soft (0.25-.5) | Medium 30-35 |
| 3 | Sandy Silt | | | High plastic (20-40) | Medium 0.1 - .001 | Very soft (0-0.25) | Loose 28-30 |
| 4 | Silty Sand | | | Very High Plastic (>40) | High >0.1 | | very Loose 25-28 |
| 5 | Clayey Sand | | | non plastic (0) | | | |
| 6 | Sand | | | | | | |
| Rank Value | 1 | 2 | 3 | 4 | 5 | 6 | 7 |

### 3.1 River Embankment Breaching Vulnerability Index (REBVI):

In this analysis of embankment breaching was delineated based on a multi-criteria analysis using bank materials, geotechnical and geometrical parameters. Variables such as texture, geotechnical parameters, and cross section of embankment either individually or in combination are associated with the embankment breaching.

i. Ranking of Different Parameters. (1-7):1 means highly ranked and 7 means lowest ranked

ii. Weighting of values within Parameter. (0-6):0 means very less vulnerable and 6 means very highly vulnerable.

After deriving the normal weights and ranks of all individual parameters were integrated with one another in a linear model in order to demarcate REBVI **Mondal et al (2012)** in the study area. The equation of REBVI is as follows:

**REBVI** = $(R_{ST} \times W_{ST}) + (R_C \times W_C) + (R_X \times W_X) + (R_{PI} \times W_{PI}) + (R_p \times W_p) + (R_{UCS} \times W_{UCS}) + (R_\Phi \times W_\Phi)$

Where R= Rank Value, W= Weight Value, ST = Soil Texture, C= Compaction, X = Cross Section, PI = Plasticity Index,

p = Permeability, UCS= Unconfined Compressive Strength, $\Phi$ = Angle of internal Friction

The result of the REBVI showed that the highest score is less potential to prevent from embankment breaching or bank erosion. Lowest score is more potential to prevent from embankment breaching or bank erosion.

The soil texture, compaction, physical parameters, plasticity index, unconfined compressive strength and angle of internal friction are most important geotechnical parameters of flood bunds, for its stability and to potential embankment breaching, which may aids to protect the flood. Geotechnical parameters are presented in **Table 3.** Soil texture of the flood bunds along River Chenab showed that most of the soil were Silty Clay and Sandy Silt. Range of Compaction was 57.2% to 95.3 % with average value of 88.6% and SD±6.195. Most of soil samples were low to medium plastic (1.0 to 15.9) with average value of 11.2 and SD±3.674. Unconfined compressive strength test results showed that the maximum compressive strength of flood bund was 1.81kg/cm$^2$ in Left Embankment along Nullah Deg RD 4-5, alternatively, the lowest was recorded from the Maddudas Flood Bund RD 43-44 (0.39 Kg /cm$^2$). Average value of all test results was 1.3kg/cm$^2$ and SD±.293. The permeability of soil samples collected from flood bunds were in the range from 0.0006 to 0.0033 cm/sec with average value of .00090 cm/s and SD±.001.

***Table 3:*** Flood bund material, structures, geotechnical attributes and River Embankment Breaching Vulnerability Index (REVBI)

| Name of Site / Bund | Location/ RD | Texture | Comp-action (%) | Cross Section | PI | Perme-ability (cm/sec) | Un-confined Compressive Strength (Kg/cm2) | Angle of Internal Friction (Degrees) | Vulner-ability Index |
|---|---|---|---|---|---|---|---|---|---|
| Pindi Bhattian F/B | RD 48-49 | Clayey sand | 87.7 | Under Design | 5.53 | 0.00257 | 0.98 | 30.60 | 52.00 |
| | RD 53-54 | Clayey silt | 82.8 | Under Design | 6.35 | 0.00301 | 0.90 | 29.80 | 55.00 |
| | RD 58-59 | Sandy silt | 88.6 | Under Design | 4.83 | 0.0032 | 0.89 | 31.00 | 54.00 |
| | RD 67-68 | Sandy silt | 82.9 | Under Design | 5.98 | 0.00292 | 0.98 | 28.80 | 57.00 |
| Buddhi Nullah | RD 3-4 | Sandy silt | 87.8 | Under Design | 11.28 | 0.00122 | 0.57 | 29.20 | 53.00 |
| Punjnad RMB | RD 33+600 | Sandy silt | 90.2 | Under Design | 5 | 0.00216 | 0.67 | 31.30 | 48.00 |
| | RD 36+000 | Sandy silt | 80.9 | Under Design | 2.93 | 0.00168 | 0.64 | 30.20 | 54.00 |
| | RD 39+100 | Sandy silt | 91.1 | Under Design | - | 0.00201 | 0.64 | 32.50 | 64.00 |
| Jhang F/B | RD 0+556 | Sandy silt | 92.6 | According to Design | 9.12 | 0.000943 | 0.56 | 28.20 | 44.00 |
| | RD 2+336 | Sandy silt | 91.7 | According to Design | - | 0.00308 | 0.00 | 29.40 | 77.00 |
| LMB Trimmu | RD 20+350 | Sandy silt | 93.9 | Under Design | 7.93 | 0.00151 | 0.67 | 29.00 | 55.00 |
| | RD 25+810 | Sandy silt | 84.1 | Under Design | 10.21 | 0.00137 | 0.54 | 31.00 | 46.00 |
| | RD 42-43 | Sandy silt | 80.7 | Under Design | 5.76 | 0.00223 | 0.79 | 30.30 | 50.00 |
| | RD 43-44 | Sandy silt | 83.2 | Under Design | - | 0.00137 | 0.18 | 31.50 | 78.00 |
| M/Garh F/B | RD 157+140 | Sand | 77.1 | According to Design | - | 0.00317 | 0.81 | 30.60 | 65.00 |
| | RD 148+400 | Silty sand | 77.7 | According to Design | 7.43 | 0.00178 | 0.58 | 31.10 | 47.00 |
| Chenab F/B | RD 87+100 | Sand | 81.8 | Under Design | 10.81 | 0.00332 | 0.69 | 32.00 | 49.00 |
| Escape channel F/B | RD 6+225 | Clayey silt | 80.4 | Under Design | 6.15 | 0.00184 | 0.66 | 31.00 | 48.00 |
| RMB Qadirabad | RD 52+800 | Clayey sand | 88.2 | Under Design | 9.44 | 0.00109 | 0.94 | 31.10 | 52.00 |
| | RD 54+450 | Silty clay | 88.5 | Under Design | 5.49 | 0.00116 | 0.84 | 31.20 | 47.00 |
| | RD 55-56 | Silty clay | 90.7 | Under Design | 4.14 | 0.00119 | 0.87 | 31.00 | 49.00 |
| | RD 59-60 | Sandy silt | 94.0 | Under Design | 6.49 | 0.0015 | 0.82 | 29.20 | 55.00 |
| Hassuwali F/B | RD 42+390 | Clayey silt | 86.92 | Over Design | 3.31 | 0.00113 | 0.89 | 30.70 | 49.00 |
| | RD 33+990 | Sandy silt | 90.12 | Over Design | - | 0.00148 | 0.72 | 31.40 | 61.00 |
| | RD 44+825 | Silty clay | 88.32 | According to Design | 9.42 | 0.00125 | 0.74 | 31.90 | 41.00 |
| | RD 57+240 | Clayey silt | 95.37 | Over Design | 6.33 | 0.00123 | 0.85 | 31.50 | 43.00 |
| Akbar F/B | RD 6+600 | Sandy silt | 87.5 | Under Design | 5.78 | 0.00144 | 0.62 | 31.20 | 50.00 |
| Shershah F/B | RD 18-19 | Sandy silt | 90.78. | Under Design | - | 0.00116 | 0.75 | 30.00 | 71.00 |

| Name of Site / Bund | Location/ RD | Texture | Comp-action (%) | Cross Section | PI | Perme-ability (cm/sec) | Un-confined Compressive Strength (Kg/cm2) | Angle of Internal Friction (Degrees) | Vulner-ability Index |
|---|---|---|---|---|---|---|---|---|---|
| Muzaffar Garh F/B | RD 82+750 | Silty clay | 87.0 | Under Design | 14.98 | 0.00112 | 1.10 | 32.00 | 37.00 |
| | RD 75+600 | Clayey silt | 80.4 | According to Design | 10.25 | 0.000583 | 1.01 | 31.00 | 27.00 |
| | RD 54+950 | Clayey silt | 82.5 | According to Design | - | 0.00199 | 1.10 | 32.00 | 52.00 |
| Khangarh F/B | RD 25+120 | Silty clay | 92.6 | Under Design | 14.97 | 0.00135 | 0.96 | 33.00 | 41.00 |
| | RD 29+990 | Sandy silt | 87.18 | According to Design | 6.75 | 0.00132 | 0.90 | 31.00 | 44.00 |
| | RD 43+100 | Clayey silt | 90.1 | According to Design | 7.21 | 0.00114 | 1.01 | 32.00 | 34.00 |
| | RD 77+700 | Clayey silt | 83.49 | According to Design | 11.81 | 0.000924 | 1.10 | 30.00 | 34.00 |
| | RD 84+400 | Silty clay | 89.66 | Under Design | 9.43 | 0.00147 | 0.81 | 31.00 | 47.00 |
| Shehr Sultan J Head Spur | RD 0-1 | Sandy silt | 82.27 | Under Design | - | 0.00135 | 1.05 | 30.00 | 67.00 |
| Chandar Bhan | RD 55+400 | Silty clay | 85.7 | Under Design | 14.53 | 0.00127 | 1.00 | 31.00 | 43.00 |
| | RD 54+300 | Clayey silt | 82.48 | Under Design | 6.11 | 0.00069 | 1.10 | 31.00 | 37.00 |
| Nagni F/B | RD 10+500 | Silty clay | 82.99 | Under Design | - | 0.00092 | 1.00 | 31.00 | 58.00 |
| | RD 15+500 | Clayey silt | 79.53 | Under Design | 7.75 | 0.00137 | 1.00 | 31.00 | 50.00 |
| Dhundoo F/B | RD 19+500 | Silty clay | 79.23 | Under Design | 5.49 | 0.00183 | 0.90 | 30.50 | 49.00 |
| | RD 21+000 | 0 | 79.89 | Under Design | 9.26 | 0.00119 | 0.71 | 30.00 | 62.00 |
| Gardez F/B | RD 5+500 | Silty sand | 80.51 | Under Design | 9.36 | 0.000751 | 1.10 | 31.00 | 40.00 |
| | RD 13+500 | Silty clay | 75.74 | Under Design | 5.16 | 0.00133 | 1.01 | 30.00 | 50.00 |
| | RD 15+500 | Sandy silt | 77.28 | Under Design | - | 0.00169 | 0.91 | 31.00 | 68.00 |
| Sat Burji F/B | RD 64+500 | Clayey silt | 81.34 | Over Design | 1.08 | 0.0015 | 0.91 | 30.00 | 56.00 |
| | RD 65+900 | Clayey silt | 85.69 | Over Design | - | 0.00141 | 1.11 | 31.10 | 55.00 |
| Jalalpur Khaki F/B | RD 6+100 | Sandy silt | 83.32 | Under Design | 9.5 | 0.00132 | 0.71 | 31.10 | 50.00 |
| | RD 7+500 | Sandy silt | 76.19 | Under Design | 7.22 | 0.000689 | 0.71 | 31.50 | 47.00 |
| | RD 45+500 | Sandy silt | 81.19 | According to Design | 1.66 | 0.000975 | 1.21 | 31.90 | 37.00 |
| Balochwah F/B | RD 3+500 | Silty sand | 74.73 | Over Design | 12.62 | 0.00142 | 0.86 | 32.00 | 46.00 |
| | RD 4+500 | Silty clay | 81.36 | Over Design | 4.52 | 0.00138 | 0.91 | 30.00 | 55.00 |
| | RD 16+500 | Silty clay | 84.68 | Over Design | - | 0.00138 | 0.82 | 30.00 | 67.00 |
| Chenab F/B | RD 23+000 | Sandy silt | 84.19 | Under Design | 9.96 | 0.000601 | 1.01 | 33.50 | 39.00 |
| | RD 6+500 | Silty clay | 81.99 | Under Design | 15.93 | 0.00103 | 0.81 | 31.00 | 43.00 |
| LMB Qadirabad | RD 17+000 | Clayey silt | 85.93 | Under Design | 11.28 | 0.000987 | 0.72 | 33.00 | 39.00 |
| | RD 21+000 | Sandy silt | 92.94 | Under Design | - | 0.00115 | 0.72 | 33.00 | 64.00 |
| Left Embankment Nullah Deg u/s Kingra Bridge | RD 9+700 | Silty sand | 87.1 | Under Design | 9.7 | 0.000778 | 0.00 | 31.20 | 58.00 |

| Name of Site / Bund | Location/ RD | Texture | Comp-action (%) | Cross Section | PI | Perme-ability (cm/sec) | Un-confined Compressive Strength (Kg/cm2) | Angle of Internal Friction (Degrees) | Vulner-ability Index |
|---|---|---|---|---|---|---|---|---|---|
| Dadal F/B | RD 16+456 | Clayey sand | 74.53 | Under Design | 2.53 | 0.000675 | 1.05 | 30.40 | 47.00 |
| | RD 22+500 | Silty clay | 83.5 | Under Design | 9.51 | 0.0006 | 1.00 | 29.70 | 49.00 |
| Kot Naja FB | RD 49-50 | Sandy silt | 89.7 | Under Design | 5.5 | 0.00163 | 0.86 | 31.00 | 50.00 |
| Massan F/B | RD 34-35 | Sandy silt | 88.9 | Under Design | - | 0.0017 | 0.95 | 32.00 | 66.00 |
| | RD 47-48 | Clayey silt | 57.26 | Under Design | 9.23 | 0.00178 | 1.05 | 32.00 | 44.00 |
| Maddu Das F/B | RD 43-44 | Silty clay | 89.06 | Under Design | - | 0.0011 | 0.39 | 31.00 | 69.00 |
| | RD 45-46 | Silty clay | 88.11 | According to Design | 11.62 | 0.00102 | 0.43 | 30.00 | 50.00 |
| | RD 48-49 | Silty clay | 88.18 | Under Design | 10.74 | 0.000961 | 0.47 | 31.20 | 44.00 |
| LMB Marala | RD 44-45 | Sandy silt | 87.39 | Under Design | 12.65 | 0.00116 | 0.74 | 33.30 | 46.00 |
| | RD 49-50 | Clayey silt | 84.69 | Over Design | 10.9 | 0.00101 | 0.81 | 33.10 | 41.00 |
| | RD 56+000 | Silty sand | 89.75 | Under Design | 9.64 | 0.00111 | 0.81 | 31.70 | 51.00 |
| | RD 59-60 | Sandy silt | 94.56 | Under Design | 10.6 | 0.00209 | 0.86 | 32.00 | 44.00 |
| Rangpur F/B | RD 279+500 | Silty sand | 76.59 | Under Design | - | 0.00169 | 0.90 | 31.10 | 69.00 |
| | RD 287+500 | Sandy silt | 80.4 | Under Design | 5.34 | 0.00249 | 1.05 | 30.90 | 44.00 |
| | RD 290+500 | Silty clay | 77.18 | Under Design | 4.5 | 0.00169 | 0.81 | 27.40 | 67.00 |
| | RD 303+500 | Sand | 80.26 | Under Design | - | 0.0027 | 0.00 | 31.20 | 81.00 |
| Embankment along left bank of Deg Nullah from Choor Bridge to Jaistywala | RD 4-5 | Silty sand | 90.76 | Under Design | 14.77 | 0.000638 | 1.81 | 31.00 | 34.00 |
| | RD 11-12 | Silty clay | 91.82 | Under Design | 6.2 | 0.000612 | 1.17 | 28.70 | 41.00 |
| | RD 13-14 | 0 | 95.34 | Under Design | 14.86 | 0.000561 | 1.72 | 32.00 | 36.00 |

## 3.2 River Embankment Breaching Vulnerability Index (REBVI):

The results of River Embankment Breaching Vulnerability Index (REBVI) are presented in **Table 3**. The REBVI value is ranging from 27 to 81 with average 42.1 and SD±11.052. The lowest value of REBVI was recorded for Muzaffar Garh F/B RD (75+600) and highest score REBVI value was recorded for Rangpur F/B (RD 303+500).

## 3.3 Cluster Analysis:

Cluster analysis was performed based on vulnerability index for each indicator of embankment breach (**Table 4**). Five clusters were considered for analysis

According to River Embankment Breaching Vulnerability Index (REBVI), Cluster-I (<40 REBVI value) represents high potential to prevent embankment breaching at 11 sites and II (REBVI value of 41 - 50) represents low potential to prevent embankment breaching point at 35 sites. In these areas of Cluster I and II, the consistence of the embankment is very high and high respectively. Cluster III (REBVI value of (51 – 60) represents moderate vulnerability at 16 sites. In Cluster IV (REBVI value of 61 – 70) represents High probability of embankment breaching at 12 sites and V (>70 REBVI value) very high vulnerability points were delineated at 4 sites. In

these areas of Cluster III, IV and V, the consistence of the embankment are medium, low and very low respectively.

**Table 4:** Probability of Flood Bund Breaching zoned in five clusters according to their River Embankment Breaching Vulnerability Index **(REBVI)**

| Cluster | Range of River Embankment Breaching Vulnerability Index (REBVI) | Consist-ency | Cluster Description of River Embankment Breaching | Location / Sites | No. of sites (%) |
|---|---|---|---|---|---|
| I | < 40 | Very High | High potential to prevent embankment breaching | Muzaffar Garh F/B (RD 82+750, 75+600)<br>Khangharh F/B (RD 43+100, 77+700)<br>Chandarbhan F/B ( RD 54+300)<br>Gardez (RD 5+500)<br>jalalpur Khaki (RD 45+500)<br>Chenab F/B (RD 23+000)<br>LMB Qadirabad (RD 17+000)<br>Embankment along Nullah Deg (RD 4-5, 13-14) | 11 (14.10) |
| II | 41-50 | High | Low embankment breaching | Punjnad RMB (RD 33+600)<br>Jhang F/B (RD 0+556)<br>LMB Trimmu (RD 25+810, 42-43)<br>Muzaffar Garg F/B (RD 148+400)<br>Chenab F/B (RD 87+100, 6+500)<br>Escape Channel F/B (RD 6+225)<br>RMB Qadirabad(RD 54+450, 55-56)<br>Hassuwali F/B (RD 42+390, 44+825, 57+240)<br>Akbar F/B (RD 6+600)<br>Khangarh F/B (RD 25+120, 29+990, 84+400)<br>Chandar Bhan /B (RD 55+400)<br>Nagni F/B (RD 15+500)<br>Gardez F/B (RD 13+500)<br>Jalalpur Khaki (RD 6+100, 7+500)<br>Balochwah (RD 3+500)<br>Dadal F/B (RD 16+456, 22+500)<br>Kot Naja F/B (RD 49-50)<br>Massan (RD 47-48)<br>Maddudas F/B (45-46,48-49)<br>LMB Marala (RD 44-45, 49-50, 59-60)<br>Rangpur F/B (RD 287+500)<br>Nullah Deg Left Embankment F/B (RD 11-12)<br>Dhundoo F/B (RD 19+500) | 35 (44.87) |
| III | 51-60 | Medium | Moderate potential of embankment breaching | Pindi Bhattian F/B (RD 48-49, 53-54, 58-59, 67-68)<br>LMB Marala (RD56+000)<br>Budhi Nullah (RD 3-4)<br>Punjnad (RD 36+000)<br>LMB Trimmu (RD 20+350)<br>RMB Qadirabad F/B (RD 52+800, 59-60)<br>MuzaffarGarhF/B (RD 54+950)<br>Nagni F/B (D 10+500)<br>Sat Burji F/B (RD 64+500, 65+900)<br>Balochwah F/B (RD 4+500)<br>Left Embankment Nukah Deg Kingra Bride (RD 9+700) | 16 (20.51) |
| IV | 61-70 | Low | High probability of embankment breaching | Punjnad RMB (RD 39+100)<br>Muzaffar Garh F/B (RD 157+140)<br>Hassuwali F/B (RD 33+990)<br>Shehar Sultan F/B ( RD 0-1)<br>Dhundoo F/B (RD 21+000)<br>Gardez F/B ( RD 15+500)<br>Balochwah F/B (RD 16+500)<br>LMB Qadirabad F/B ( RD 21+000)<br>Massan F/B (RD 34-35)<br>Maddudas F/B (RD 43-44)<br>Rangpur F/B (RD 279+500, 290+500) | 12 (15.38) |
| V | > 70 | Very Low | Very High probability of embankment breaching | Rangpur F/B (RD303+500)<br>Shershah (RD 18-19)<br>Jhang F/B (RD 2+336)<br>LMB Trimmu (RD 43-44) | 4 (5.13) |

# 4 CONCLUSIONS AND RECOMMENDATIONS

## 4.1 Conclusions:

1. The results of analysis from test sites of Flood Bunds along River Chenab showed that Geotechnical and Geometrical properties of embankment are significantly responsible for embankment breaching. Though the areas are suffering from high breaching, however Rangpur F/B (RD 303+500), Shershah (RD 18-19) Jhang F/B (RD 2+336) and LMB Trimmu (RD 43-44) are most vulnerable sites. Embankment Vulnerability point selection is very important for ensuring the success of any geotechnical solution for better management of embankment breaching and its stability.
2. Grain Size Analysis of soil samples collected from 78 sites of Flood Bunds indicates that 18 % sites are of very uniform soil, 64 % of medium uniform while 17% of very non uniform.
3. Plasticity Index of soil samples collected from identified sites of flood bunds were calculated after finding out liquid and plastic limits of soil samples collected through boreholes. The results indicate that 23% soil samples are medium plastic, 12% slightly plastic, 43% low plastic and 22% non-plastic.
4. Relative Compaction of soil samples indicate that the relative compaction has higher value at 22% of flood bund sites, 61% medium compacted and 17% are low compacted.
5. Unconfined Compressive Test shows that soil at 5% of flood bund sites is very soft, 4% soft, 68% medium and 23% stiff.
6. Permeability Results indicate that soil at 0% of flood bund sites is impervious, 23% low and 77% medium.
7. Direct Shear Test indicates the angle of internal friction of the soil samples which shows that 1% soil samples are very loose, 23% loose, 76% medium and 0% dense.
8. Geometry of the flood bunds was observed and it was found that 4% of the sites were according to the design, 12% were over design and 84% sites were under design.
9. Geochemical Analysis of soil samples collected from flood bunds reveal that TDS at 4% of sites is above limit while 96% sites are showing their results within limits, Organic Matter of all soil samples collected from different flood bunds is within limit and sulphate content in soil samples are 90 % within limits and 10% above limits. The results of analysis of water samples collected from bore holes at different sites indicate that chloride values are within permissible limit and the result of analysis of TDS show that all water samples are within limit.
10. Cluster Analysis based on River Embankment Breaching Vulnerability Index (REBVI) was divided in five clusters. Cluster-I (<40 REBVI value) represents high potential to prevent embankment breaching at 11(14 %) sites and II (REBVI value of 41 - 50) represents low potential of embankment breaching point at 35 (45%) sites. Cluster III (REBVI value of 51 – 60) represents moderate potential of embankment breaching at 16 (21%) sites. In Cluster IV (REBVI value of 61 – 70) represents high probability of embankment breaching at 12 (15%) sites and V (>70 REBVI value) represents very high probability of embankment breaching at 4 (5%) sites.
11. Human Intervention in the form of Irrigation system for crop cultivation leads to the break of embankment which also weakening the embankment.
12. Rodent attacks and Rat holes also weakening the River Embankment.

## 4.2 Recommendations:

1. Cross-sections of flood bunds need to be maintained as per original design.
2. The geotechnical properties of embankment materials need to be improved by using materials like clay, silty clay and clayey silt.
3. Additional pushta to be provided on country slope where HGL is exposed.
4. Periodic monitoring of the flood bunds is recommended before and after flood season.
5. Geophysical survey to be carried out using latest techniques and tools.
6. Activation of wetting channels may be ensured.
7. Future construction must be ensured as per standard specifications and material (silt from 18 to 50%, sand less than 52% and clay from 7 to 17% and 95% compaction of all layers).
8. Slope stability analysis to keep the phreatic line protected shall be carried out at critical sites.
9. Design optimization should be ensured keeping in view the flood frequency analysis.
10. Site specific corrective measures may be adapted to safeguard the flood bunds against heavy floods.
11. Geosynthetics materials can be used to strengthen the existing flood bunds.
12. Clay core and sand core can also be used as a measure to strengthen the existing flood bunds.

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# HISTORY OF TOLERABLE DEFORMATION OF FOUNDATIONS

**Sohail Kibria & Sajid Iqbal**

# HISTORY OF TOLERABLE DEFORMATION OF FOUNDATIONS

By

**Sohail Kibria[1] & Sajid Iqbal[2]**

## Abstract

The issue of prediction of deformation of foundations has been attracting the attention of geotechnical as well as structural engineers for ensuring sound performance of structures. Quite often, a realistic prediction of actual deformation is quite difficult because structures are too complicated to be modeled, actual material properties are not well known, as-built is not the same as designed, rate and stages of construction and post-construction change in geo-environmental factors are not readily known etc.

The deformation based limit states viz. architectural and serviceability limits states, are directly related to the deformations and need to be established separately. In practice, some settlements do take place safely before finishes are placed and therefore may not cause any distress to the final product.

The prediction of deformation at different stages of construction and their threshold values for distress-free performance of structures have been handled by various researchers including Terzaghi and Peck (1948), Skempton and MacDonald (1956), Meyerhof (1956), Polshin and Toker (1957), Sowers (1962), Bjerrum (1963), Burland and Wroth (1974), Grant et al. (1974), Wahls (1981) and Moulton et al. (1985), etc. Whereas various standard field and laboratory testing and analyses techniques are in vogue for prediction of deformations, empirical and observational procedures have been adopted by various researchers as well as by building codes, for defining safe threshold of foundation deformation.

Current study investigates the credible work done in this regard since 1930s to date.

Engineers should recognize potential problems associated with structures on variable ground conditions individually. The intention of this paper is to provide awareness to engineering community on need to consider allowable movement of structures with regard to the type of structure and foundation and underlying subsurface soil conditions.

**Keywords:** Limit states, Differential settlement, Deformation prediction

## 1. Introduction

There has been tremendous growth of construction industry in Pakistan since last decade. Every structure needs a foundation system to transfer its load to underlying soil. Foundations of structures are now increasingly being built on weaker ground. There is a need for engineers to consider the allowable settlements of the foundations in their design and understand the basic soil-structure interaction. Engineers have to estimate or predict the amount of deformation and that whether the structure can tolerate it. Settlement problems are not confined to high-rise buildings and important structures alone and indeed a number of single storey houses have experienced excessive differential settlements, which resulted in very expensive repair. There are certainly cases where structures have settled as much as a foot without significant damage to the structure. In Mexico City, many structures have settled on the order of seven feet and still

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remained operable. The Palace of Fine Art Building, built in early 1900s, has settled more than 3.5 m in clay soil but is still in use (Lambe and Whitman, 1969; Das and Sivakugan, 2010). Design engineers should consider both the total and differential settlements but it is the differential settlement that is really of concern.

The settlement-induced building damage is critical for building new structures. The study of foundation behavior is one of the most important links between structural and geotechnical engineering: the reason for possible foundation deformations which can lead to damage as regards the serviceability and safety of structures is often of geotechnical nature. A realistic prediction of deformation is often challenging that is not limited to but includes complexity of structure, type and size of structures, heterogeneous nature of subsurface soils, as-built is not the same as designed, rate and stages of construction, rate and uniformity of settlements and post-construction changes in geo-environmental factors etc.

Differential settlement is the difference in settlement between two foundations or the difference in settlement between two points on a single foundation. The differential settlement of various parts of a building can lead to damage of a superstructure. Hence, it is important to quantify the total and differential settlements and develop limiting values for these parameters for desired safe performance of structures.

No matter how accurate a settlement prediction analysis may be, it is of limited practical value if the designer is not aware of the amount of settlement which can be allowed for a particular structure under consideration. The amount of allowable settlement depends on many factors and due to its complexity most researchers have relied on observations of actual settlements and damages in actual structures to develop empirical methods to determine the magnitude of settlement allowed. Usually, a qualitative assessment of level of damage is correlated with some measure of differential settlement. Then, an attempt is made to relate the differential settlement with maximum total settlement. This paper deals with review of design codes and credible work done by several researchers since 1930s to date on allowable movement of structures with regard to type of structure and foundation and underlying subsurface soil conditions and provides a guideline for design engineers.

## 2. Limit States in Foundation Design

Loading soil and rock with a structure always leads to deformation. If the deformation becomes severe enough that the structure ceases to function in an acceptable manner, then the structure has reached the limit state. We are concerned mostly with three goals in foundation design: (1) avoid collapse, which is associated with ultimate limit states, (2) preserve the serviceability of structure (a serviceability loss is serviceability limit state), (3) prevent damage to the appearance of structures, which is associated the architectural limit states. Differential settlements can damage the appearance of structures, disrupt the functionality of structure such as door and window jamming, uneven floors, utility breeches etc. and be so large and uneven that parts of structures become so overstressed that they may fail. In an extreme, such failures may lead to partial or complete collapse of a structure.

Tilt may be a problem with tall structures. In this case, one side of structure either is more heavily loaded or is supported by more compressible material. The excessive tilt may lead to instability and collapse. Tilt, even if not threatening the stability, leads to awkward if not outright unserviceable conditions. The most famous case of foundation tilt is the Leaning Tower of Pisa [See Fig. 1(a)]. Cracking in a structure as shown in Fig. 1(b) can be related to both ultimate and serviceability states. It is therefore useful to understand how cracks form in response to foundation movement and how these crack differ from those that develop due to shrinkage (of plaster, for example, upon drying) and other such local effects (Salgado, 2008).



**Fig. 1: (a)** Leaning Tower of Pisa (Coduto, 2001)



**(b)** distortion of building due to excessive differential settlement (Min, 2005)

To determine whether a foundation has settled or moved excessively, it is essential to quantify the amount of foundation deformation using a suitable parameter. The value of this parameter could then be compared with the maximum tolerable value that the parameter can assume before a limit state is reached. Differential settlement, angular distortion and deflection ratio have been adopted as defining limits by several researchers and as per codes. Allowable maximum total settlement is then correlated with these parameters.

Referring to Fig. 2, the differential settlement $\delta$ for any two columns of a frame building is the difference between the total settlements $S_1$ and $S_2$ of the two columns. If L is the span or distance between the centre of two foundations/columns, the angular distortion $\alpha$ is defined as,

$$\alpha = \frac{\delta}{L} \qquad \text{(Eq. 1)}$$

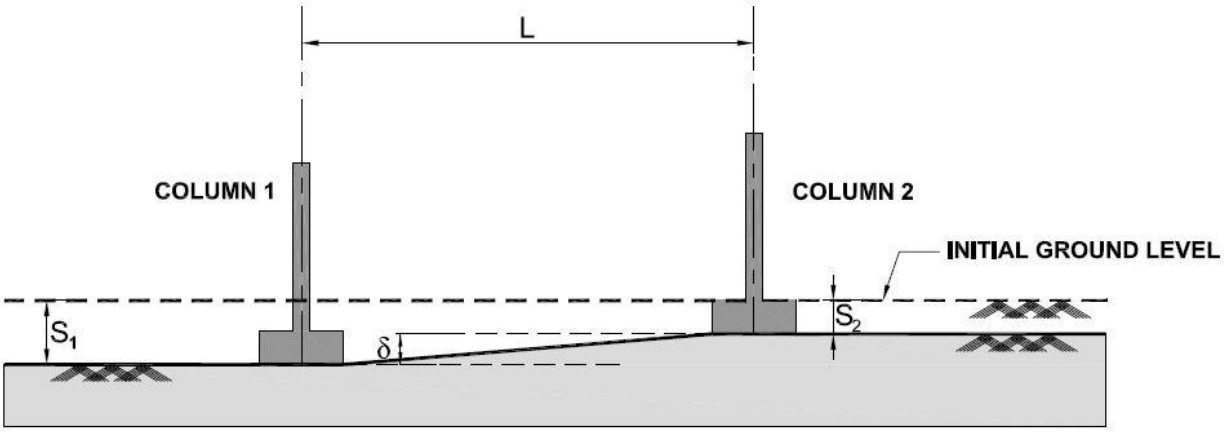



**Figure 2:** Definition of angular distortion due to differential settlement (modified after Salgado, 2008)

Fig. 3 shows that the differential settlement alone is not sufficient for assessing tolerable settlement. The serviceability and ultimate states result from the distortion of the superstructure caused by foundation deflection. It is clear from the figure that the same differential settlement causes different degree of distortion for different spans.

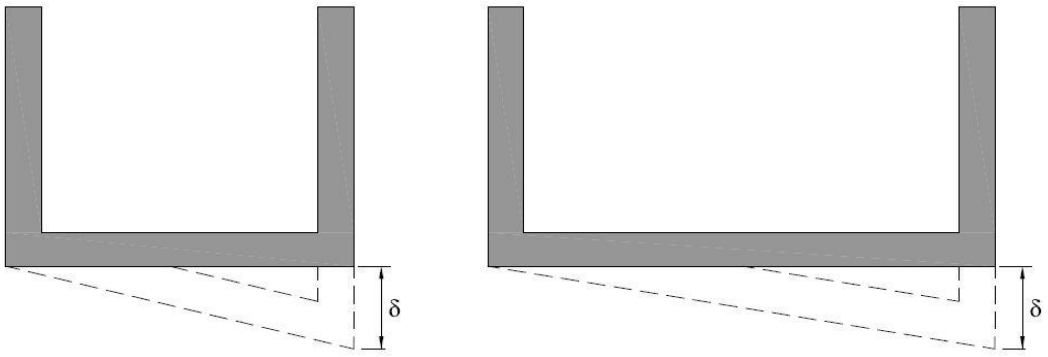



**Figure 3:** The same differential settlement causes more distortion in case (a) than in case (b) because of shorter span (modified after Salgado, 2008)

The relative deflection $\Delta$ is the vertical deflection measured from a reference line. The reference line can be generally defined as the line joining two points of foundation in such a way that the line lies always below (for hogging) or above (for sagging) all other foundation points. It tends to appear when excavation or tunneling activities are underway near the building and can therefore cause greater foundation movement near the edges of the buildings. Soil swelling can also generate hogging but swelling tends to be uneven and it is difficult to predict with accuracy.

The maximum relative deflection $\Delta_{max}$ is the maximum value of the relative deflection $\Delta$, that is, the maximum vertical distance between the deformed foundations and reference line. The maximum deflection ratio ($\Delta$/L) is the ratio of $\Delta_{max}$ to the horizontal projection L of the reference line, which coincides with the width of building when whole foundation is either in the sagging mode or hogging mode. For a symmetric building under symmetric loading in uniform soil, the entire building would generally be either in a sagging mode or hogging mode, and $\Delta_{max}$ would be observed exactly at mid span. For the more complex situation as shown in Fig 4(b),

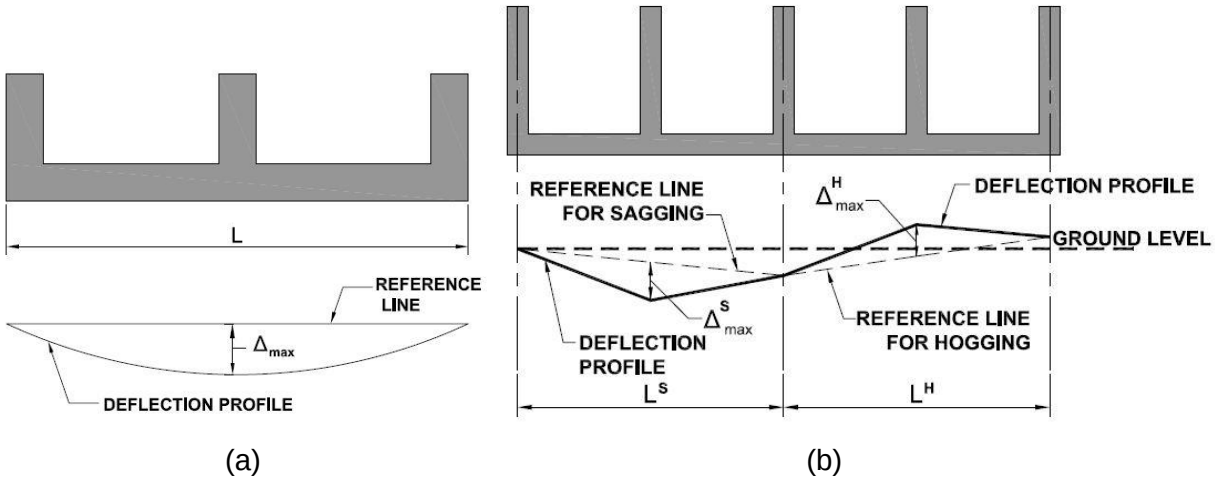



**Figure 4:** Foundation configurations and resulting maximum relative deflection $\Delta_{max}$ (a) symmetric sagging and (b) complex structure both in sagging and hogging modes (modified after Salgado, 2008)

$\Delta^S_{max}$ for sagging is associated with appropriate span $L^S$, while $\Delta^H_{max}$ for hogging is associated with $L^H$ (Salgado, 2008).

## 3. Historical Development of Limit States

Foundation deformation needs to be limited to allowable levels to ensure adequate serviceability of the structures and preserve their appearance. Since the 1930s, attempts have been made by various researchers and standard codes to recommend allowable values for limits states. A brief description of the pioneering work carried out by researchers on development of limit states is reported chronologically in the following, along with the summary of limits states presented in Table 1 to 3.

### 3.1 Buildings, Special Structures and Allied Facilities

**3.1.1 Terzaghi (1938):** Terzaghi studied the performance of many buildings and concluded that most ordinary structures can tolerate a differential settlement of 20 mm between two adjacent columns. Terzaghi and Peck (1948) also suggested that the differential settlements of footings on sands are unlikely to exceed 75% of total settlement, which means that total settlement, the frame can withstand is 25 mm. For clays, however, differential settlement may be greater than this value and may even approach the total settlement (NAVFAC, 1982). This may be the reason for designing most of the building foundations for the total settlements of 25 mm.

**3.1.2 Skempton and MacDonald (1956):** Skempton and MacDonald were among the first to propose a rational way to assess tolerable settlement for frame building by using the concept of angular distortion $\alpha$. They developed a database of 98 buildings, both frame buildings and masonry buildings with load-bearing walls, in area of London, England. For each building, they had sufficient information on foundation settlement and on whether any cracking developed as a result of the settlement. Skempton and MacDonald gave the angular distortion limits for cracking of walls or panels in conventional buildings or load bearing walls as 1/300 and for structural damage to columns and beams as 1/150.

According to their observations, as long as the angular distortion is capped at 1/500, foundation settlement will not lead to either structural or architectural damage. This value already includes a reduction factor to account for uncertainties in both the observations and calculated settlements; therefore, design loads need not to be magnified by load factors for settlement estimation. Skempton and Macdonald also attempted to establish correlations between angular distortion, maximum total settlement and maximum differential settlement. Their correlations depend on the type of soil and foundation. These criteria were later confirmed in works of Grant et al. (1974) and Wahls (1981).

Skempton and Macdonald's evaluation did not consider the influence of building's length to height ratio or other measures of relative shear and bending distortion. A number of authors including Polshin and Toker (1957), Bjerrum (1963), Burland and Wroth (1974), Grant et al. (1974) and Wahls (1981) have either reassured angular distortion criteria or proposed similar empirical approaches.

**3.1.3 Meyerhof (1956):** Meyerhof also treated loads bearing walls and frame panels separately and recommended angular distortion of 1/250 for open frames, 1/500 for infilled frames and 1/1000 for load bearing walls or continuous brick cladding. It should be noted that angular distortion of 1/1000 corresponds to deflection ratio of about 1/2500 (Burland and Wroth, 1974). Therefore Meyerhof's recommendations are closely in line with those of Polshin and Toker (1957).

**3.1.4 Polshin and Toker (1957):** Polshin and Toker presented the Soviet experiences and included allowable settlement criteria from the U.S.S.R Building Code (1955). Polshin and Toker adopted the concept of length to height ratio of a building and limiting tensile strain before cracking as the basis of their analyses.

**3.1.5 Ohsaki (1960):** Ohsaki compared computed and observed settlements of actual buildings for the purpose of making contribution towards development of limit states. Buildings selected include seventeen apartment buildings, five schools buildings and two hospital buildings, all of which were located in Hiroshima city, Japan. Ohsaki concluded that angular distortion of 1/1000 to 1/2000 is more appropriate for reinforced concrete structures and are slightly conservative than values of angular distortion recommended by Skempton and MacDonald (1956).

**3.1.6 Sowers (1962):** Sowers indicated that structures that are more flexible, such as simple steel frame buildings, or have more rigid foundations, such as mat foundations can sustain larger values of total and differential settlement. Allowable values of angular distortion and maximum settlement are listed in the summary Table 1 and 2.

**3.1.7 Bozozuk (1962):** A particularly extensive study was conducted by Bozozuk on 574 civil monotype buildings in Ottawa, Canada. These buildings were all old two storey load bearing wall structures with superficial foundations. Bozozuk reported that buildings experiencing angular distortion of 1/180 and settlement up to 50 mm remained undamaged.

**3.1.8 Bjerrum (1963):** Bjerrum supplemented the recommendations made by Skempton and MacDonald (1956) and presented a figure relating angular distortion to building performance based on additional data combined with those of Skempton and MacDonald study and is widely referenced (e.g. USACE (1990), NAVFAC (1982), etc.)

Figure 5 provides limiting values for buildings containing sensitive machinery or overhead cranes. In addition, Bjerrum also reported the allowable total and differential settlement for isolated and raft footings on clays to avoid any damage to buildings.

Foundations on sands seldom exhibit large settlement due to monotonic increase in stress; typically, settlement will be less than 75 mm (Bjerrum, 1963; Terzaghi, 1956). Most problems with the foundations on sands are due to vibration or other forms of cyclic loading, or due to groundwater seepage and erosion. In the areas of the world that are prone to seismic activity, earthquake activity can cause large-scale instability and highly damaging movement of foundations on sands.

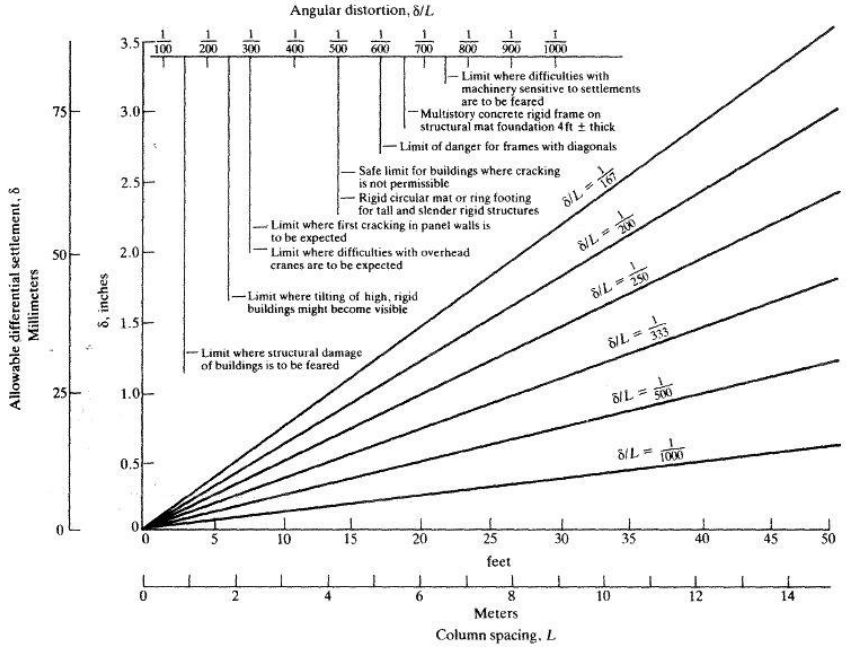



**Figure 5:** Tolerable settlement for buildings (Bjerrum, 1963; NAVFAC, 1982; USACE, 1990)

**3.1.9 Feld (1965):** Feld cited a rather large number of specific structures observed in United States, UK, Europe, Soviet Union, Egypt and Turkey with a given settlement and structural response, which might be of interest in considering a specific problem. However Feld concluded that no answer can be expected to the question that how much settlement a structure can tolerate, until much more data is available to indicate the limits of tolerance for structures of various rigidities and for settlements at different time rates.

**3.1.10 Burland and Wroth (1974):** Burland and Wroth concluded that while the allowable limit of angular distortion of 1/500 proposed by Skempton and MacDonald (1956) is satisfactory for frame buildings, it is unsafe for load bearing walls undergoing sagging, and criteria proposed by Polshin and Tokar (1957) is more appropriate. However criteria proposed by Polshin and Toker is not adequate for load bearing walls undergoing hogging since cracking has been observed to occur at value of deflection ratio which are half those recommended by Polshin and Toker.

Burland and Wroth also proposed appropriate allowable limits of angular distortion on the basis of his observation of load bearing walls undergoing hogging. The problem to allowable settlement should be based on assessing global stiffness of a structure and then estimating the deflections ratio that will result from its interaction with ground. The likely deflection ratios can then be compared with damage criteria.

**3.1.11 Grant et al. (1974):** Study of Skempton and MacDonald (1956) was substantiated by Grant et al. from a study of 95 additional buildings of more recent construction and the allowable limits and correlations were reevaluated with respect to amount of damage. Their recommendations are summarized in Table 2 and 3, and are in line with Skempton and MacDonald (1956) with slight improvement in the allowable values. The angular distortion values associated with a given settlement decreases with increasing rigidity of the raft so according correlation would be valid only for flexible raft and for situations where it is economically justifiable to be very conservative, e.g. in the design of high-precision sensitive machinery foundations.

It was observed by Grant and his co-workers that large settlement generally associated with deep clay deposits cause relatively smaller differential settlement than do those associated with shallow clay layers. It is popular premise that buildings undergoing slow differential settlement are damages less than buildings sustaining equal but rapid settlements. To evaluate this hypothesis, Grant et al. obtained information on the rate of settlement for 66 buildings and concluded that based on limited data available, the values of maximum angular distortion corresponding to building damage to be essentially the same for cases involving slow and fast settlements. Consideration of the rate of settlement is only important for the extreme conditions of either very slow or fast settlements.

**3.1.12 Wahls (1981):** Wahls generalized the approach of Skempton and MacDonald, homogenizing with the theories of Polshin and Tokar (1957) and Burland and Wroth (1974). His conclusions are in line with the recommendation of Burland and Wroth (1974).

**3.1.13 Ricceri and Soranzo (1985):** Ricceri and Soranzo examined the allowable settlement of structures built in Italy, the result of which had been published in past 25 years. Sixty nine (69) structures of various types including steel, reinforced concrete and load bearing brick walls with both shallow (raft, anular beams) and deep foundations (wooden piles, diaphragm walls) in cohesive, granular and layered soils. It should be noted that no structure with isolated footings were examined. Range of structure stiffness is large ranging from flexible structures (steel oil tanks), intermediate stiffness office and industrial buildings, to high stiffness structures (reinforced concrete cellular silos). Ricceri and Soranzo suggested a total settlement limit of 80 mm for no damage and 200 mm to avoid intolerable structural damage. It is notable that some

buildings founded on relatively rigid rafts have tolerated maximum settlement in excess of 400 mm, without suffering significant damage.

**3.1.14 Bowles (1996):** Bowles endorsed the findings of Skempton and MacDonald however recommended that if total and differential settlements are kept within slightly conservative values listed in parenthesis in Table 2, statistically the structure should adequately resist the foundation.

**3.1.15 Coduto (2001):** The generic $\delta_{max}/S_{max}$ ratios in Table 2 may be used to predict differential settlement. These values are based on Bjerrum (1963)'s data and Coduto (2001)'s professional judgment and are probably conservative.

**3.1.16 Zhang and Ng (2005, 2007):** Zhang and Ng collected displacement information on over 300 buildings and compiled into a database. Approximately 200 buildings are located overseas, reported by Skempton and MacDonald (1956) and Grant et al. (1974), and over 100 buildings are located in Hong Kong and Mainland China, reported by Lumb (1964) and other researchers. Twenty tall buildings in Hong Kong and Mainland China, which were constructed in the past decade but experienced considerable settlement, were included for study. In terms of usage, the buildings in the database can be classified into office types, such as office blocks, flats, hotels and hospitals, and mill types, such as ware houses and factories. Both steel and reinforced concrete structures and structures with load bearing walls were included in database. The buildings were founded in three major soil conditions: sand, clay and alluvium. The foundation types included pile foundations and shallow foundations (i.e. footings, raft and box foundations). Nearly all the tall buildings in the database were on pile foundations, except a few cases that adopt box foundations.

### 3.1.16.1 Probabilistic Based Allowable Limits:

Although the safe tolerable settlements and angular distortion limits recommended in the past by all researchers and design codes are deterministic, Zhang and Ng studies showed that, owing to the presence of uncertainties, the tolerable limits of a particular type of structure is a random variable rather than a fixed value. Zhang and Ng presented a probability based approach for determining the distribution of limiting tolerable displacements and observed intolerable displacements for all categories of buildings. Mean values of limiting tolerable vertical displacement and angular distortion were obtained after performing probability distributions for a particular type of structure.

**3.1.16.1(a) Characteristic Value Approach:** A characteristic tolerable displacement at a specified probability of exceedance $\alpha$ was determined based on the probability distribution of the limiting tolerable displacement. A value $\alpha$=0.05 was selected by Zhang and Ng considering the reliability and uncertainty of observed foundation displacements.

**3.1.16.1(b) Safety Factor Approach:** The allowable deterministic displacement can also be calculated for conventional design by applying a safety factor to the mean limiting tolerable displacement (Bjerrum, 1963; Wahls, 1994). Zhang and Ng calculated deterministic tolerable displacement for conventional design by applying a safety factor of 1.5, as reported by Wahls (1994), to the mean limiting tolerable displacement. The allowable vertical displacements and angular distortions based on safety factor approach and characteristic values approach are presented in Table 2.

### 3.1.16.2 Comparison

It can be observed that the characteristic values are generally smaller than those determined by applying safety factor of 1.5 to mean limiting values. Zhang and Ng stated that the allowable vertical displacement values and angular distortion used in practice appear to have been

determined more by a safety factor approach. Yet, the use of characteristic value approach is preferred since this considers both the magnitude and variability of limiting displacement, and hence can lead to consistent level of confidence.

Referring to Table 2, it can also be observed that allowable vertical displacement (total settlement) for frame buildings and load bearing wall structures calculated by Zhang and Ng using safety factor approach are smaller than those suggested by Polshin and Toker (1957) and Chinese Code for Design of Building Foundations (MOC, 2002). For buildings located on sands and clays, Zhang and Ng calculated allowable vertical displacements using safety factor approach fall into the ranges suggested by Skempton and MacDonald (1956).

## 3.2 Settlement Behavior of Foundations on Sands and Clays

Buildings on clay are found to have higher tolerable vertical displacements compared to those on sands. This may be caused by three factors. First, vertical displacements of a foundation on clay usually occur more slowly than on sands. Second, particularly applicable to shallow foundation, the same amount of displacement of foundation in clay may cause smaller internal forces in the superstructure since clay is usually clay is less stiff hence the soil reaction on structural elements is likely to be smaller. Third, as most common construction materials are subject to long term deformation, large strains can be accommodated without cracking when deformation occurs slowly (Zhang and Ng, 2007).

Buildings on sands represent a different problem than clay. In first case, because settlement occurs rapidly, more of deformation is developed before installation of nonstructural finishes, which are more sensitive to distortion. So it is unusual to find cases of buildings supported on sands with significant damages (Polshin and Tokar, 1957; Bozozuk, 1962). Total settlements on clay are generally greater than those on sands owing to consolidation process. But differential settlements which are more dangerous for buildings, tends to be greater on sands because sands deposit are generally he heterogeneous (Burland et al., 1977). Beside, structure on sands cannot profit by slow rate of deformation (creep), which allows greater strains because of viscosity of construction materials, leading to greater allowable differential settlement.

## 3.3 Bridges

**3.3.1 Moulton et al. (1985):** Moulton and his co-workers carried out a comprehensive study of tolerable movement of 314 highway bridges in the United States and Canada for the Federal Highway Association (FHWA) of the United States. The bridges types include steel, concrete and composite, and were simply supported or continuous span built on a range of soils (clays, sands) and rocks. The primary causes were excessive movement of approach embankments, inadequate lateral resistance and large foundation displacements. The acceptable limiting values of tolerable angular distortion for single span and continuous span brides are 1/200 and 1/250, respectively, of both steel and concrete bridges. Moulton et al. also recommended limiting the horizontal movement of abutment to 38 mm.

There seems to be general agreement among investigators who have studied the performance of bridges that horizontal movements cause more severe and widespread problems than do settlements (Bozozuk, 1978; Walkinshaw, 1978; Moulton et al., 1985; Wahls, 1990).

**3.3.2 Barker et al. (1991):** Based on the re-examination of the data compiled by Moulton et al. (1985), Barker et al. (1991) worked for National Cooperation for Highway Research Program (NCHRP) and revised the recommendation made by Moulton and his co-workers and recommended to adopt the angular distortion values of 1/125 and 1/250 for single span and continuous span bridges, respectively. Barker et al. (1991) noted that vertical settlement is more damaging when accompanied by horizontal movements.

Various investigators (Bozozuk, 1978; Walkinshaw, 1978; Grover, 1978; Moulton et al., 1985; Wahl, 1990) have suggested the allowable limits of settlement and horizontal movement and those are summarized in Table 3.

**3.3.3 Zhang and Ng (2005):** Zhang and Ng also determined tolerable horizontal movement of bridge abutment and angular distortions for bridges using probability based approach and displacement information on bridges reported by Moulton et al. (1985) and have been presented in Table 3. Zhang and Ng focused on 171 bridges that have experienced appreciable bridge abutments movements and 204 bridges that have experienced certain angular distortion. Limiting tolerable

## 4. Threshold of Deformation as per Codes

Several credible codes have established recommendations for designing the structures according to limit states. Tables 1 to 3 summarize the recommended values of the deformation limits.

### 4.1 Buildings, Special Structures and Allied Facilities

**4.1.1 U.S.S.R Building Code (1955):** The principal of design according to limit states had been adopted in the U.S.S.R Building Code for all type of engineering structures (steel, reinforced concrete, masonary and timber). The limiting values of angular distortion, deflection ratio, and allowable settlement adopted by U.S.S.R, which were based on more than 25 years of settlement observations, are summarized in Table 2 and 3.

**4.1.2 NAVFAC (1982):** Limiting values of settlement and angular distortion as recommended by Bjerrum (1963) and Polshin and Toker (1957) were reported by Naval Facilities Engineering Manual (NAVFAC, 1982) 7.01 of U.S. Navy.

**4.1.3 United State Army Corps of Engineers (1990):** USACE stated that many structures can tolerate substantial downward movement without cracking as reported by Polshin and Toker (1957) and Bjerrum (1963). However it is recommended that total settlement should not exceed 50 mm for most facilities and differential settlement in buildings should be limited to 12.5 mm to avoid cracking and structural damage. A typical specification of total settlement for commercial buildings is 25 mm (Lambe and Whitman, 1969).

**4.1.4 American Society of Civil Engineers (1993):** ASCE recommends allowable settlement of 25 mm and angular distortion of 1/500.

**4.1.5 Eurocode (1994):** As regards the serviceability and ultimate limits states, Eurocode offered limiting values are shown in Table 1 and 2. It is interesting to note that the guidelines proposed by Eurocode are similar to those developed by Skempton and MacDonald (1956) more than 30 years later. However, as indicated by Eurocode, recommended limiting values are only simple guidelines and not rigid rules.

**4.1.6 Chinese Code for Design of Building Foundations (2002):** The tolerable vertical displacements for frame buildings and load bearing walls can be limited to 120-200 mm as per Chinese Code for Design of Building Foundations (Ministry of Construction, Beijing, 2002). The Code also stipulated tolerable angular distortion values for buildings with heights ranged up to 100 m and are reported in Table 1 and 2.

**4.1.7 Canadian Foundation Engineering Manual (2006):** Tolerable limits for foundations deformation as listed in Table 1 and 2, according to Canadian Foundation Engineering Manual (2006) may be used for low risk projects and an initial guide for higher risk projects and provides a realistic estimate of tolerable movements. Communication between structural and

geotechnical engineers is encouraged to address adequately appropriate serviceability limit states criteria.

**4.1.8 Institution of Civil Engineers (ICE)' Manual of Geotechnical Engineering (2012):** ICE Manual of Geotechnical Engineering recommended routine limits for allowable settlement and angular distortion for a range of structures are summarized in Table 1 and 2 following a thorough review of work done by Ricceri and Soranzo (2005) and Zhang and Ng (2005).

ICE Manual of Geotechnical Engineering also reported deterministic deformation limiting values on the basis of studies carried out by Zhang and Ng (2005). Most buildings with settlement of less than 100 mm are undamaged and there were no report of damage when settlement is less than 100 mm. However, when the settlements exceed 200 mm intolerable damage becomes common. It also reported that values of angular distortion which are more severe than 1 in 400 are usually intolerable.

## 4.2 Bridges

**4.2.1 ICE Manual of Geotechnical Engineering (2012):** ICE Manual of Geotechnical Engineering recommended values of 60 mm, 40 mm and 1/250 to 1/500 for maximum settlement, horizontal displacement of abutment and angular distortion, respectively, for the serviceability of bridge structures.

ICE Manual of Geotechnical Engineering also reported, based on comprehensive study on tolerable movements of bridges by Zhang and Ng (2005), that movements are usually tolerable if horizontal movement of abutment and angular distortion are less than 50 mm and 1 in 500, respectively.

**4.2.2 AASHTO (2012):** AASHTO adopted the allowable values of angular distortion for bridges, as recommended by Barker et al. (1991). Recommended limits states by ICE Manual of Geotechnical Engineering (2012) for bridges are summarized in Table 3.

Table 1: Summary of Credible Literature on Allowable Limit States (Angular Distortion, $\Delta/L$ and Deflection Ratio, $\Delta/L$)

| Description of Structure | U.S.S.R (1955) and Polshin and Tokar (1957); Reported by USACE (1990) | | Shampton and MacDonald (1990) | Meyerhof (1956, 1979) | Ohsaki (1969) | Bowles (1982) | Bozozuk (1982) | Bjerrum (1963) | Grant et al. (1974) | Burland and Wroth (1974) | NAVFAC (1982) | Riccati and Serranzo (1985) | USACE (1990) | Eurocode (1994) | ASCE (1993) | Chinese Building Code (BSOC, 2002) | Zhang and Ng (2004); See Note 1 | Canadian Foundation Eng. Manual (2006) | ICE Geotech Manual (2012) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | On sand or hard clay | On plastic clay | On sand or hard clay | On plastic clay | | | | | | | | | | | | | | | |
| | $\Delta/L$ (See Note 2) | | $\Delta/L$ | | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ | $\Delta/L$ |
| Ore and industrial Building Columns Foundations | | | | | | | | | | | | | | | | | | | |
| Steel frame structures | | 1/500 | 1/500 | | | | | | | | | 1/500 | 1/500-1/300 | | | | | | |
| Reinforced concrete frames structures | | 1/450 | 1/300 | 1/300 | 1/1000-1/2000 | 1/250-1/400 | | | 1/300 | | | 1/500 | 1/333-1/300 | | 1/500 | | 1/400 | | 1/500-1/900 |
| End row of columns with brick cladding | | 1/150 | 1/1000 | | | | | | | | | | | | | | | | |
| Structures where auxiliary arms does not arise during non-uniform settlement of foundations | | 1/300 | 1/400 | | | | | | | | | | | | | | | | |
| Tilts of smoke stacks, water tower, silos etc. | | 1/250 | 1/250 | | 1/250 | | | | | | | | 1/400 | | | | | 1/2000 | |
| Crane rails and tracks for bridge crane trucks | | 1/333 | 1/333 | | | | | | | | | | | | | | | | |
| Plain brick walls: | | | | | | | | | | | | | | | | | | | |
| For multistorey dwellings and civil buildings | | | | | | | | | | | | | | | | | | | |
| at L/H ≥ 5 | 1/5000 | 1/2500 | 1/1500 | 1/1000 | | | | | | | | | 1/1250 | | | | | | |
| at L/H ≥ 5 | 1/2500 | 1/1500 | 1/600 | 1/500 | | | | | | | | | 1/2500 | | | | | | |
| Load bearing wall structures: Serviceability limit | | | | 1/2500 | 1/1000 | | | | 1/500 | | | 1/500 | | | | | | | 1/500-1/900 |
| Ultimate limit | | | | 1/150 | | | | | | | | 1/150-1/300 | | | | | | | 1/150-1/250 |
| Simple storey mill buildings, wall cracking | 1/1000 | 1/1000 | 1/400 | | | 1/500-1/1000 | | | | | | | | | | | | | |
| Simply supported steel frames | | | | | | 1/300 | | | | | | | | | | | | | |
| Continuous steel frames | | | | | | 1/500 | | | | | | | | | | | | | |
| Open frames | | | | 1/150 | 1/250 | | | | | | | | | | | | | | |
| High continuous brick walls | | | | | | 1/1000-1/2000 | | | | | | | | | | | | | |
| Plaster cracking (gypsum) | | | | | | 1/1000 | | | | | | | | | | | | | |
| Reinforced concrete building curtain walls | | | | | | 1/333 | | | | | | | | | | | | | |
| Machine operation – cotton loom | | | | | | 1/333 | | | | | | | | | | | | | |
| Machine operation – turbo generator | | | | | | 1/5000 | | | | | | | | | | | | | |
| Overhead cranes | | | | | | | | 1/500 | | | 1/300 | | 1/500 | | | | | | 1/500 |
| Chimneys | | | | | | 1/333 | | | | | | | | | | | | | |
| Cracking of floor slabs | | | | | 1/150 | 1/150 | | | | | | | | | | | | | 1/150-1/150 |
| Two storey brick houses | | | | | | | 1/150 | | | | | | | | | | | | |
| Machinery sensitive to Settlement | | | | | | | | 1/750 | | | 1/750 | | 1/750 | | | | | | 1/250-1/500 |
| Frames with diagonals | | | | | | | | 1/250 | | | 1/600 | | 1/600 | | | | | | |
| Cracking of brick walls | | | | | | | | 1/150 | | | 1/150 | | 1/150 | | | | | | |
| Cracking of panel walls | | | | | | | | 1/150-1/300 | | | 1/150-1/300 | | 1/300 | | | | | | |
| Cracking in walls and partition | | | | | | | | | | | | | | | | | | 1/500 | |
| Cracking in walls and partition - end bays | | | | | | | | | | | | | | | | | | 1/1000-1/1400 | |
| Visible tilting of high rise buildings | | | | | | | | 1/250 | | | 1/250 | | 1/250 | | | | | 1/300 | |
| General Buildings | | | | | | | | | | | | | | | | | | | |
| Safe limit for no cracking | | | 1/250 | | | 1/500 | 1/500 | | | | 1/500 | | 1/500 | 1/500 | | | | | 1/500-1/900 |
| Danger of structural damage | | | 1/150 | | | 1/150 | | | | | 1/150 | | 1/150 | 1/150 | | | | 1/150-1/250 | 1/150-1/250 |
| Reinforced-concrete-framed brick wall panels: | | | | | | | | | | | | | | | | | | | |
| Structural damage | | | | | | | | | | | | 1/500 | | | | | | | |
| No serious problem (serviceability limit) | | | | | | | | | | | | 1/150-1/200 | | | | | | | |
| Brick cladding to steel columns | | | | 1/1000 | | | | | | | | | | | | | | | |
| Unreinforced load bearing walls: | | | | | | | | | | | | | | | | | | | |
| Safe limit for cracking | | | | 1/1000 | | | | | | | 1/1000 (L/H=3) | | 1/1000 | | | | | 1/1000 (L/H=1) | 1/1000 (L/H=1) |
| | | | | | | | | | | | 1/500 (L/H=5) | | | | | | | 1/500 (L/H=5) | 1/500 (L/H=5) |
| Safe limit for toppling | | | | 1/2000 | | | | | | | 1/3000 (L/H=1) | | 1/3000 | | | | | 1/3000 (L/H=1) | 1/3000 (L/H=1) |
| | | | | | | | | | | | 1/2500 (L/H=5) | | | | | | | 1/2500 (L/H=5) | 1/2500 (L/H=5) |
| Steel petroleum or fluid storage tanks | | | | | | | | | | | 1/300 | | | | | | | | 1/300-1/900 |
| Circular silos tank: Flexible base/Rocking top | | | | | | | | | | | | | 1/150 | | | | | | |
| Flexible base/fixed top | | | | | | | | | | | | | 1/125 | | | | | | |
| Jointed rigid concrete pressure conduits | | | | | | | | | | | 1/50 | | | | | | | | |
| Tower like buildings: Serviceability limit | | | | | | | | | | | | | | 1/1200-1/2000 | | | | | |
| Ultimate limit (collapse) | | | | | | | | | | | | | | 1/250 | | | | | |
| Building with Height (H): H ≤ 24 m | | | | | | | | | | | | | | | 1/250 | | | | |
| 24 m < H ≤ 60 m | | | | | | | | | | | | | | | 1/333 | | | | |
| 60 m < H ≤ 100 m | | | | | | | | | | | | | | | 1/500 | | | | |
| H > 100 m | | | | | | | | | | | | | | | 1/1000 | | | | |

Note 1: ICE Geotech Manual (2012) reported deterministic values of angular distortion on the basis of studies carried out by Zhang and Ng (2005). Note 2: $\Delta/L = 1/300$ is approx. equal to $\Delta/L = 3/900$ (Burland and Wroth, 1974)

**Table 2: Summary of Credible Literature on Allowable Total Settlement & Differential Settlement, Angular Distortion and Correlations (See Note 2)**

| Based on Structure Type | | Based on Foundation Type and Subsurface Soils | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| U.S.S.R (1955) and Polshin and Tokar (1957), Reported by USACE (1990) | Average $S_{max}$ (mm) | Skempton & MacDonald (1956), Values in brackets recommended by Bowles (1996) | Building Foundation Type | Allowable Max. Differential Settlement, $\delta_{max}$ (mm), when $\delta L/L=1/300$ | | Allowable Max. Total Settlement, $S_{max}$ (mm), when $\delta L/L=1/300$ | | $R = (\delta L/L)/S_{max}$ ($in^{-1}$) | | $(\delta L/L)_{max}$ where B is raft width | Max. Differential Settlement, $\delta_{max}$ (in) | |
| | | | | Sands | Clays | Sands | Clays | Sands | Clays | Clays | Sands | Clays |
| Building with plain brick walls on continuous and separate foundations | | | | | | | | | | | | |
| L/H ≤ 1.5 | 80 | | Isolated | 32 (25) | 46 (35) | 50 | 78 | 1/600 | 1/1000 | - | 350 (L/L) | 550 (L/L) |
| L/H ≥ 2.5 | 100 | | Raft | 32 (25) | 46 (35) | 50-78 (35-65) | 78-127 (65-100) | 1/750 | 1/1250 | - | 350 (L/L) | 550 (L/L) |
| Buildings with brick walls, reinforced with reinforced concrete or reinforced brick belts (independent of L/H ratio) | 150 | Grant et al. (1974) | Isolated or Piles | 25 | 50 | 50 | 100 | 1/600 | 1/1200 | - | 350 (L/L) | 550 (L/L) |
| Frame Buildings | 100 | | Raft | 30 | 56 | 90 | 60 (B=15) 175 (B=45 m) | 1/750 | 1/1250 | 1.25 ($S_{max}$/18.5 B) $S_{max}$ (in), B (ft) | 350 (L/L) | 550 (L/L) |
| Solid reinforced concrete foundations of blast furnaces, smoke stacks, silos, water towers, etc. | 300 | | Strip | Allowable max. total & differential settlement and R values lie in between values for isolated and raft foundations as rigidity of strip footing lies in between isolated/raft. | | | | | | - | 350 (L/L) | 550 (L/L) |
| Sowers (1962) | Allowable $S_{max}$ (mm) | ICE Geotech Manual (2012) | Foundation Type | Allowable Max. Differential Settlement, $\delta_{max}$ (mm) | | Allowable Max. Total Settlement, $S_{max}$ (mm) | | Notes: 1. These limits are intended for low risk solutions and simple structures. 2. Pad/strip footings (generally flexible) may be rigid depending on their thickness compared with size. 3. ($\delta_{L/H}$ for isolated, strip, pad footings): (a) limit is applicable when clay is immediately below the foundation. When a stiff layer (dense sand or gravel stratum) is between foundation and clay, then differential settlement can be substantially reduced. (b) Assume linear interpolation to assess $\delta_{L/H}$ when $S_{max}$ is between 50 mm and 200 mm. 4. $\delta_{max}$ for raft on clay can be reduced up to <0.15 $S_{max}$ as its relative bending stiffness increases. | | | | |
| Drainage/Connection to channelization | 150-300 | | | Sands | Clays | Sands | Clays | | | | | |
| Access | 300-600 | | | | | | | | | | | |
| Masonary load bearing walls | 25-50 | | Isolated, Pad, Strip | < 0.75 $S_{max}$ | <0.66 $S_{max}$ for $S_{max}$<50 mm & <0.5 $S_{max}$ for $S_{max}$>200 mm | 40 | 65 | | | | | |
| Frame structures | 50-100 | | | | | | | | | | | |
| Smoke stacks, silos, mats | 50-300 | | Raft | - | < 0.33 $S_{max}$ | 65 | 100 | | | | | |
| Canadian Foundation Engineering Manual (2006) | Allowable $S_{max}$ (mm) | Bjerrum (1963), See Note 3 | Isolated | - | 50 | - | 150 | Coduto (2001) | Design Values of $\delta_{max}/S_{max}$ for Spread Footings (Square/Rectangular) | | | |
| Avoid damage to services connections Sands | 50-75 | | Raft | - | 125 | - | 250 | Predominant soils below footings | | | Flexible | Rigid |
| Avoid damage to services connections Clays | 50-125 | | | | | | | Sandy | | | | |
| ASCE (1993) | Allowable $S_{max}$, mm | Eurocode 7 (2004) | Settlement Type | Serviceability Limit | | Max. Acceptable Movement for Isolated Footing | | Natural Soils | | | 0.9 | 0.7 |
| | | | | Isolated | Raft | | | Compacted fill of uniform thickness underlain by stiff natural soils | | | 0.5 | 0.4 |
| Avoid damage to utilities services connections in structures (serviceability) | 25 | | $S_{max}$ (mm) | 25 | 50 | 50 | | Clayey | | | | |
| | | | $\delta_{max}$ (mm) | - | - | 20 | | Natural Soils | | | 0.8 | 0.5 |
| Terzaghi (1939) & Terzaghi and Peck (1948) | | Zhang & Ng (2005), See Note 1 | Foundation Type | sand, clay and alluvium soils strata | | | | Compacted fill of uniform thickness underlain by stiff natural soils | | | 0.4 | 0.3 |
| For ordinary structures footings | | | | $S_{max}$ for no damage | | $S_{max}$ for intolerable damage | | Burland et al. (1977) | | | | |
| Allowable $S_{max}$ (mm) | 20 | | Isolated | 50-100 mm | | 200 mm | | Frame buildings on isolated footings | | | $\delta_{max}$ (mm) | $S_{max}$ (mm) |
| Allowable $S_{max}$ ($S_{max}$≤0.75) on sands | 25 | | | | | | | Settlement limits to avoid damage | | | 50 | 150 |
| Bozozuk (1962) | | Rodriguez-Ortiz & Scarnato (1986) | Piles, Raft, Annular Beam, etc. | granular, cohesive and layered soils | | | | | | | | |
| Allowable $S_{max}$ (mm) for two storey brick load bearing walls structures | 50 | | | $S_{max}$ for no damage | | $S_{max}$ for intolerable damage | | | | | | |
| | | | | 80 mm | | 200 mm | | | | | | |

| Zhang and Ng (2005, 2007) | | | | | | | | | | Chinese Code for Design of Building Foundations (MOC, 2002) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Approach | Limit Type | Usage Building | | Structural Type | | Foundation Type | | Soil Type | | | |
| | | Mill | Office | Frame Building | With load bearing walls | Shallow | Deep | Sand/Fill | Clay | | |
| Allowable value, mean limiting tolerable value/FS (Safety factor=1.5) | Vertical Settlement/Total Settlement (mm) | 141 | 81 | 92 | 60 | 145 | 71 | 57 | 113 | Tolerable settlement for frame buildings and buildings with load bearing walls (as reported in Zhang and Ng, 2007) | 120-200 mm |
| | Angular Distortion, $\delta L$ | 1/285 | 1/435 | 1/320 | 1/415 | 1/245 | 1/770 | - | 1/333 | United States Army Corps of Engineers, USACE (1990) | |
| Characteristics value, α | Vertical Settlement/Total Settlement (mm) | 53 | 47 | 29 | 22 | 49 | 42 | 27 | 44 | $S_{max}$ for most facilities | $\delta_{max}$ for buildings |
| (α, probability of exceedance=0.05) | Angular Distortion, $\delta L$ | 1/525 | 1/833 | 1/1000 | 1/1111 | 1/833 | 1/3333 | - | 1/1667 | 50 mm | 12.5 mm |

Note 1: Institution of Civil Engineers' Geotech Manual (2012) reported deterministic values of total settlement on the basis of studies carried out by Zhang and Ng (2005).

Note 2: Values of angular distortions reported only based on probabilistic approach based studies carried out by Zhang and Ng (2005, 2007). Refer to Table 1 for summary of credible literature on allowable angular distortion.

Note 3: Also refer to Figure 5 for determining allowable differential settlement based on allowable angular distortion and spacing between centres of foundations/columns.

Table 3: Summary of Credible Literature on Allowable Movements of Bridges

| Reference | | Angular Distortion, Δ/L | | Allowable Horizontal Movement of Bridge Abutment (mm) | Allowable Bridge Pier Settlement (mm) | Basis for Recommendations |
|---|---|---|---|---|---|---|
| | | Single span bridges | Continuous span bridges | | | |
| Moulton et al., (1985), FHWA | | 1/200 | 1/250 | 38 | - | - |
| AASHTOO (2012) | | 1/125 | 1/250 | - | - | - |
| ICE Geotech Manual (2012) | | 1/250-1/500 | | 40 | 60 | - |
| Deterministic values by ICE (2012) using Zhang and Ng (2006) work | | 1/500 | | 60 | - | Tolerable |
| Barker et al., (1991), NCHRP | | 1/125 | 1/250 | - | - | - |
| Bozozuk (1978) | Reported by Barker et al. (1991), NCHRP | | | 25 | 51 | Not harmful |
| | | - | - | 51 | 102 | Harmful but tolerable |
| Wahlstrom (1978) | | - | - | | 63 | Ride Quality |
| | | | | 51 | > 63 | Structural Distresses |
| Grover (1978) | | - | - | | 102 | Ride quality and structural distresses |
| Wahls (1990) | | - | - | 51 | > 102 | Usually intolerable |
| Reference | | Zhang and Ng (2005) | | | | |
| Approach | | Limit Type | | All Bridges | Steel Bridges | Concrete Bridges |
| Allowable value, mean limiting tolerable value/FS (Safety factor=1.5) | | Horizontal movement of bridge abutment | | 90 mm | 89 mm | 88 mm |
| | | Angular Distortion, Δ/L | | 1/180 | 1/238 | 1/192 |
| Characteristics value, c* | | Horizontal movement of bridge abutment | | 42 mm | 51 mm | 47 mm |
| (α=0.05) | | Angular Distortion, Δ/L | | 1/588 | 1/538 | 1/345 |

* probability of exceedance

## 5. Prediction of Deformations

### 5.1 Factors affecting Prediction of Deformations

Prediction of deformations can be affected by multiple factors that are not limited to but include the following:

**(a). Geotechnical investigations limitations:** Accurate settlement analyses require detailed and high quality geotechnical investigations in good quantity to determine subsurface soil profile and engineering properties of soils. Limited and poor quality geotechnical investigations programme may lead to uncertainties in settlement predictions.

**(b). Variation in structural loads**: Ratio of actual loads to design loads may not be the same for all foundations leading to irregular settlement pattern.

**(c). Variability in soil properties**: Part of structure might be underlain by stiff natural soil, and a part by loose sand or soft clay. Such a structure may experience excessive differential settlement.

**(d). Time rate and uniformity of settlement**: Structure can be profited by the slow rate of deformation (creep) which allows greater strains before cracking, leading to greater allowable differential settlement. Negligible differential settlement is usually induced due to uniform total settlement.

**(e). Construction discrepancies**: Field construction methods may be significant in settlement outcome. For example, weak soil underlying the proposed depth of base of footing requires some compaction or replacement with select fill. Depending on the compactor and amount of compaction, the soil state can be changed significantly (increase in density, apparent overconsolidation, stiffness, etc.) leading to substantial reduction in settlement.

### 5.2 Common Methods for Deformation Predictions

As a means of estimating both the amount and time of consolidation and resulting settlement in clays, consolidation tests are run in a laboratory. Terzaghi theory of consolidation is widely used to calculate settlement due to the consolidation till date.

Settlement on sand is not amenable to solution based on laboratory consolidation tests. Indeed, settlement on sand is generally calculated by empirical means. Geotechnical literature is full of methods for predicting settlement of shallow foundations on sands. The methods for calculating settlement on sand, commonly used in practice, include: Steinbrenner and Fox method, Timoshenko and Goodier method, Alpan method, Teng method, Meyerhof method, Schmertmann method, Burland and Burbidge method and Mayne and Poulos method.

There is no convenient way to estimate the horizontal movement at abutments. Designer must estimate the possible amount of horizontal movement that may occur at a bridge abutment using experience and judgment or they must employ the modern finite element method based geotechnical computational tools. Terzaghi et al. (1996) recommended limiting the embankment surcharge to less than one-half of the bearing capacity of the subgrade soils to minimize the risk of progressive lateral movement. No systematic study has been made that confirms that this criterion is reliable under all possible situations.

United States Army Corps of Engineers (1990) reported that maximum angular distortion of footings due to soil heave and non uniform soil wetting may be estimated as per following guidelines:

i) When potential for soil heave and non-uniform soil wetting exist, the maximum angular distortion may be sum of the maximum settlements without soil wetting

and maximum potential heave of wetted soil divided by minimum distance between the respective settlement locations. Maximum potential heave may occur under lightly loaded part of foundations such at midpoint between diagonal columns and will normally only be the immediate elastic settlement; consolidation is not expected in soils with potential for heave in-situ.

ii) When potential for soil heave and uniform wetting occurs, the maximum angular distortion will be the difference between the maximum and minimum soil heave divided by distance between these locations.

## 6. Factors for Allowable Settlements

On the basis of observations by researchers, it reveals that the allowable settlement and angular distortion depends on several factors, including following:

**(a). The type of structure**: For example, wood frame buildings would be much more tolerable than unreinforced brick buildings.

**(b). Type of foundation**: For example, raft foundation can tolerate greater amount of differential settlements in highly variable grounds than isolated foundations.

**(c). Use of the structure**: Even a small crack in a house might be considered acceptable; whereas larger cracks in industrial building might not even be noticed.

**(d). The rigidity of structure**: Rigidity of structure has significant influence on differential settlement. A rigid structure will redirect some of its load to surrounding columns foundations, and thus provides more resistance to differential settlement.

**(e). Type of soil**: Foundations placed over different soil types would have different tolerable settlements. Footing on clay can tolerate higher settlement as compared to sand.

When assessing the severity of damage for an existing structure, damage criteria should be based on multiple factors including crack width, differential settlement and angular distortion. Severity of cracking damages versus approximate crack width and typical values of angular distortion of the foundations as summarized in Table 4 can be adopted as a guide for the performance assessment of building (Burland et al., 1977; Day, 2010).

**Table 4: Severity of Cracking Damage in Buildings (Burland et al., 1977; Day, 2010)**

| Damage category | Typical damage description | Approx crack width | Angular Distortion, δ/L |
|---|---|---|---|
| Negligible | Hairline cracks | < 0.1 mm | < 1/300 |
| Very slight | Very slight damage includes fine cracks that can be easily treated during normal decoration, perhaps an isolated slight fracture in building, and cracks in external brickwork visible on close inspection | 1 mm | 1/300-1/240 |

**Table 4: Severity of Cracking Damage in Buildings (Burland et al., 1977; Day, 2010)**

| Damage category | Typical damage description | Approx crack width | Angular Distortion, δ/L |
|---|---|---|---|
| Slight | Slight damage includes that can easily be filled and redecoration would probably be required; several slight fractures may appear showing on the inside of the building; cracks that are visible externally and some repointing may be required; doors and windows may stick | 3 mm | 1/240-1/175 |
| Moderate | Moderate damage includes cracks that require some opening up and can be patched by a mason; recurrent cracks that can be masked by suitable linings; repointing of external brickwork and possibly a small amount of brickwork replacement may be required; doors and windows stick; service pipes may fracture; weather-tightness is often impaired | 5-15 mm or number of cracks > 3 mm | 1/175-1/120 |
| Severe | Severe damage includes large cracks requiring extensive repair work involving breaking out and replacing section of walls (especially over doors and windows); distorted windows and door frames; noticeably sloping floors; leaning or bulging walls; some loss of bearing in beams; and disrupted service pipes | 15-25 mm but also depends on number of cracks | 1/120-1/70 |
| Very Severe | Very severe damage often requires a major repair job involving partial or complete rebuilding; beams lose bearing; walls lean and require shoring; windows are broken with distortion; and there is danger of structural instability | Usually > 25 mm but also depends on number of cracks | >1/70 |

Foundations subjected to settlement can be damaged by a combination of both vertical and horizontal settlements. A common situation where both vertical and horizontal foundation displacement occurs at cut-fill transition, such as shown in Figure 6 where lateral movement is the secondary result of the primary vertical movement due to settlement of foundation. In cases where lateral movement is the most predominant or critical mode of foundation, Table 4 can underestimate the severity of cracking damages for value of angular distortion (Day, 2010).

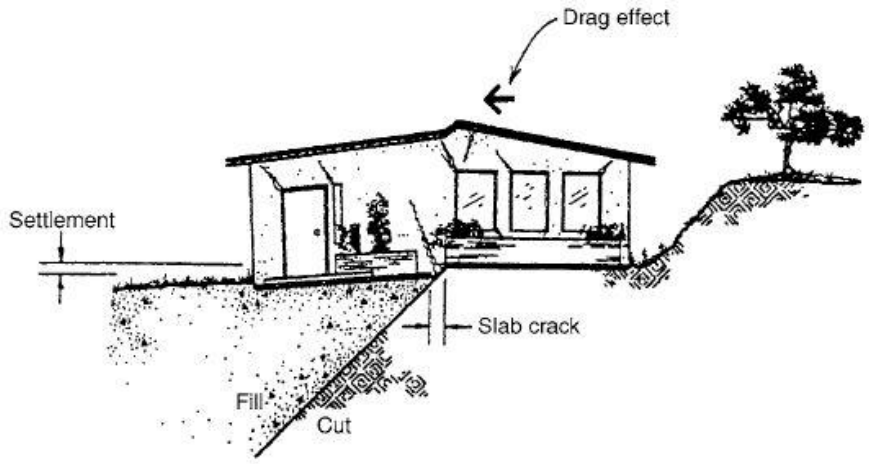



**Figure 6:** Cut-fill transition

## 7. Recommendations

There must be some buried guidelines in the file of every engineer and every contractor which, if properly collated, would indicate limit of tolerance in every type of structure located on every type of soil and foundation. The design engineer should be aware of possible expected deformation and the threshold values for safety. With the knowledge of the allowable deformation, it may be possible for engineers to choose proper foundation system for the proposed structure and given subsurface soil conditions.

At the start of project, the geotechnical engineer should consult with the engineer of concerned disciplines or architect about the anticipated loading conditions, allowable settlement of structures, stages of construction, activity schedule and preliminary thoughts on the type of structures.

When the entire structure settles equally at every foundation points, the magnitude of total settlement is of little concern. It is the differential settlement that should be a concern.

The values of deformation limits states in table 1 to 3 provide a good guideline for design engineers with reference to type of structure and foundation and underlying soil type for normal works in Pakistan. It is observed that the recommended values of allowable limits by Skempton and MacDonald (1956), Grant et al. (1974), Zhang and Ng (2005, 2007) using safety factor approach and ICE Manual of Geotechnical Engineering (2012) lie in close proximity.

Besides, buildings on clays have higher tolerance to vertical displacements compared to those on sands. Sands can cause more severe problems to overlying structure, owing quick settlement response.

After a careful review of the credible literature, standard design codes and personal experiences in local soil conditions and considering factors affecting allowable limits and deformation predictions, the following general recommendations have been framed for the design of foundations of building structures in Pakistan.

**Table 5:** General Recommended Values for Foundation Design of Ordinary Building Structures in Pakistan

| Foundation Type | Allowable Max. Differential Settlement, $\delta_{max}$ (mm) | | Allowable Max. Total Settlement, $S_{max}$ (mm) | | Allowable Angular Distortion, δ/L |
|---|---|---|---|---|---|
| | Sands | Clays | Sands | Clays | |
| Isolated/Strip | 25 | 35 | 35 | 45 | 1/400 |
| Raft | 30 | 40 | 75 | 100 | 1/500 |

However selection of allowable limits by design engineer should be followed with good engineering judgment, consideration factors for allowable limits, local construction constraints and on the basis of quality of information about the ground conditions.

Engineering judgment should be used in deriving the final settlement using available methods, keeping in mind the factors affecting settlement and comparisons between predicted and observed settlement for various methods. However our experience indicates that prediction of settlement using Teng and Meyerhof methods have been quite reliable and lead to consistent level of confidence.

Recommendations for the design of bridge foundations have been formulated in Table 6 followed by a thorough review of pioneering work done by researched and design codes. Settlement limits are stated for bridge pier without regard to the type of foundation system.

**Table 6:** Recommended Values for Bridges in Pakistan

| Angular Distortion, δ/L | | Allowable Horizontal Movement of Bridge Abutment (mm) | Allowable Bridge Pier Settlement (mm) | | Limit for Recommendations |
|---|---|---|---|---|---|
| Single span bridges | Continuous span bridges | | Single span bridges | Continuous span bridges | |
| 1/125 | 1/250 | 50 | 50 | 35 | Tolerable |

Horizontal movements of abutment cause more severe damage to bridge structures than vertical settlements of bridge foundations do. There is no convenient way to estimate the horizontal movement at abutments. Designer must employ the modern numerical modeling based computational tools to estimate the possible amount of vertical and horizontal movement that may occur at a bridge abutment.

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**Paper No. 788**

# FINANCIAL AND ECONOMIC ANALYSIS OF PROJECT OF UP-GRADATION / REHABILITATION OF MAIN LINE-2 OF PAKISTAN RAILWAYS

**Abdul Rehman Chaudhry & Muhammad Inam**

# FINANCIAL AND ECONOMIC ANALYSIS OF PROJECT OF UP-GRADATION / REHABILITATION OF MAIN LINE-2 OF PAKISTAN RAILWAYS

By:

**Abdul Rehman Chaudhry[1] and Muhammad Inam[2]**

Main Line-2 (ML-2) route of Pakistan Railway's network comprises its following five existing sections:

| | | |
|---|---|---|
| 1. | Kotri – Dadu – Habib Kot | = 346 km |
| 2. | Habib Kot – Jacobabad – Kashmor | = 177 km |
| 3. | Kashmor – D.G. Khan – Kot Adu | = 306 km |
| 4. | Kot Adu – Bhakkar – Kundian | = 231 km |
| 5. | Kundian – Basal – Attock City | = 194 km |
| | Total: | = 1,254 km |

The above mentioned ML-2 route joins main line ML-1 route (Karachi-Lahore-Rawalpindi-Peshawar) at Kotri in the south and at Attock City in the north.

The existing infrastructure on the above mentioned sections has mostly completed its useful life and is in depleted condition. The up-gradation/rehabilitation of the infrastructure on these sections is essentially required to meet not only the anticipated traffic after establishment of China-Pakistan Economic Corridor (CPEC), but also to meet with the additional transportation demands of imported coal for coal fired power plants to be built in Sindh & Punjab provinces.

The importance of ML-2 route will increase manifold after the proposed construction of new railway line between Gwadar and Jacobabad as it will provide an additional route for the transportation of freight traffic to up country and Afghanistan, not only from Gwadar Port but also from Karachi/Bin Qasim Ports.

The proposed up-gradation/rehabilitation of ML-2 route shall also be part of CPEC in addition to the up-gradation of ML-1 (Karachi-Lahore-Peshawar) route of Pakistan Railway. The construction of the new rail link from Gwadar to Jacobabad (located on ML-2), is planned for construction under CPEC by 2030.

Pakistan Railways accordingly decided to carry out feasibility study of existing Main Line-2 (ML-2) from Kotri to Attock City stations with a view to ascertain the existing condition of its infrastructure and the improvements required to achieve the speeds up to 160 Km/h for passenger trains and up to 120 km/h for freight traffic.

The Contract for carrying out the above referred feasibility study was awarded in February, 2016 to the JV comprising ILF Consulting Engineers ZT GmbH, Rum/Innsbruck, Austria, and ILF Pakistan (Pvt) Limited, Lahore.

The feasibility study has been completed and draft feasibility study report has been submitted to the Client. The salient features of the financial and economic analysis of project included in the feasibility study report, have been described in this paper.

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# FINANCIAL AND ECONOMIC ANALYSIS OF PROJECT OF UP-GRADATION / REHABILITATION OF MAIN LINE-2 OF PAKISTAN RAILWAYS

## 1. INTRODUCTION

### 1.1 Project Background

Main Line-2 (ML-2) route of Pakistan Railway's network comprises its following five existing sections:

| | | |
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| | Total: | = 1,254 km |

The above mentioned ML-2 route joins main line ML-1 route (Karachi-Lahore-Rawalpindi-Peshawar) at Kotri in the south and at Attock City in the north, as shown in the Pakistan Railways Network Plan in Figure 1-1, on the next page.

According to the available data almost all the above referred five sections comprising ML-2, except Jacobabad-Kashmor and Kashmor - D.G. Khan - Kot Adu sections, were constructed between 1878 and 1899 i.e. more than 100 years ago. Jacobabad-Kashmor section was converted from Narrow Gauge to Broad Gauge and opened to traffic in 1956. The section from Kashmor to D. G. Khan and Kot Adu was opened to traffic between 1969 and 1973.

The existing infrastructure on most of the above mentioned sections comprising ML-2 has mostly completed its useful life and is in depleted condition. The up-gradation/rehabilitation of the infrastructure on these sections is essentially required to meet not only the anticipated traffic on these sections after establishment of China-Pakistan Economic Corridor (CPEC), but also to meet with the additional transportation demands of imported coal for coal fired thermal power plants to be built in Sindh & Punjab provinces.

The importance of ML-2 route will also increase manifold after the proposed construction of new railway line between Gwadar and Jacobabad as it will provide an additional route for the transportation of freight traffic to up country and Afghanistan, not only from Gwadar Port but also from Karachi/Bin Qasim Ports.

The proposed up-gradation/rehabilitation of ML-2 route shall also be part of CPEC in addition to the up-gradation of ML-1 (Karachi-Lahore-Peshawar) route of Pakistan Railways. The construction of the new rail link from Gwadar to Jacobabad (located on ML-2), is planned for construction under CPEC by 2030.

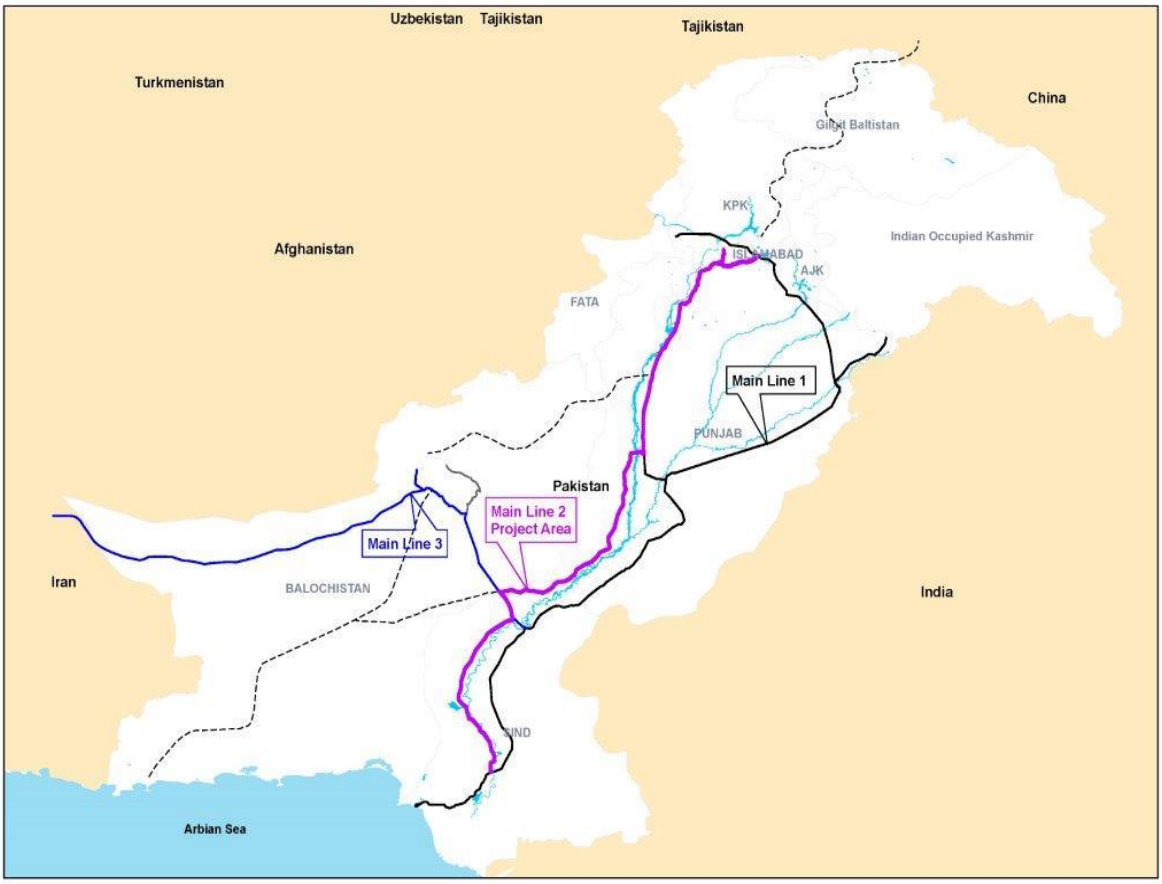



***Figure 1:*** Pakistan Railways Map for the Project Area

Pakistan Railways accordingly decided to carry out feasibility study of existing Main Line-2 (ML-2) from Kotri to Attock City stations with a view to ascertain the existing condition of its infrastructure and the improvements required to achieve the speeds up to 160 Km/h for passenger trains and up to 120 km/h for freight traffic on this route.

The Contract for conducting the above referred feasibility study was awarded in February, 2016 to the JV comprising ILF Consulting Engineers ZT GmbH, Rum/Innsbruck, Austria, and ILF Pakistan (Pvt) Limited, Lahore.

## 1.2 Feasibility Study for the Up-gradation/Rehabilitation of Main Line (ML-2)

The scope of services for the feasibility study included condition survey of the existing infrastructure, topographic survey, hydrology studies, geological & geotechnical investigations along the existing alignment, study for assessment of passenger and freight traffic including future transport demand forecast, redesigning of the existing alignment for the increased speed, improvements to the existing signalling and telecommunication systems, cost estimates and financial and economic analysis of the proposed upgradation/rehabilitation of ML-2 route.

The feasibility study of the project has recently been completed and draft feasibility study report submitted to the Client. The salient features, the financial and economic analysis which were an essential component of the study have been described in the next chapter.

# 2. FINANCIAL AND ECONOMIC ANALYSIS

## 2.1 Methodology

### 2.1.1 General approach

Cost Benefit Analysis (CBA) is an analytical tool which is used to estimate the socio-economic impact (in term of benefits and costs) related to the implementation of certain policy actions and/or projects. The impact must be assessed against predetermined objectives and the analysis is usually made taking into account the sum of all individuals directly and indirectly concerned by the action.

The objective of CBA is to identify and monetize (i.e. attach a monetary value to) all possible impacts of the action or project under scrutiny, in order to determine the related costs and benefits. In principle, all impacts should be assessed: financial, economic, social, environmental, etc. Traditionally, costs and benefits are evaluated by considering the difference between a scenario with the project and an alternative scenario without the project (the so-called “incremental approach”).

Then the results are aggregated to identify net benefits and to draw conclusions on whether the project is desirable and worth implementing. To that extent, the CBA could be used as a decision-making tool for assessing investment to be financed by public resources.

More specifically, within the framework of preparation and appraisal of potential financing institutions project, they require a CBA to:

a) To assess whether a project is worth co-financing. This assessment is performed using an Economic Analysis.

b) To assess whether a project needs co-financing. This assessment is performed using a Financial Analysis.

The proposed sequence for the CBA in the framework of project preparation is the following:

- Strategic approach and definition of objectives and methodology
- Financial Analysis and Economic Analysis
- Sensitivity analysis and Risk Assessment when it is required
- Reporting conclusions.




***Figure 2:*** General methodology for CBA

The different methodologies for each stage of CBA are described in the sub-sections below.

2.1.2 Methodology for Financial Analysis:

The purpose of the financial analysis is to assess the financial performance of the proposed project over the period under consideration, with the view to establish the most suitable financing structure for the project. This is based on the extent of its financial self-sufficiency and long term sustainability, its financial performance indicators, as well as the justification for the amount of necessary assistance being sought.

More specifically, the financial analysis has to cover the following steps:

- to estimate the project revenues and costs and their implications in terms of cash-flow
- to determine the funding gap of the selected option and subsequently calculate the eligible expenditure that can be co-financed by the Funds
- to define the project financing structure and its financial profitability
- to verify the sufficiency of the projected cash flow to ensure the adequate operation of the project and meet all investment and debt service obligations

2.1.3 Methodology for Economic Analysis

The purpose of the economic analysis is to prove that the project has a positive net contribution to society and is therefore, worth being co-financed by government funds. For the selected alternative, the project benefits should exceed the project costs and, more specifically, the present value of the project economic benefits should exceed the present value of the project economic costs.

In practical terms, this is expressed as a positive ENPV, a Benefit/Cost (B/C) ratio higher than 1, or a project EIRR exceeding the discount rate used for calculating the ENPV.

2.1.4 Methodology for Sensitivity Analysis

The purpose of the sensitivity analysis is to assess the robustness of the result of the project financial and economic indicators. To achieve this, the first part of the analysis (sensitivity analysis) aims at identifying the key variables and their potential impact in terms of changes in the profitability indicators.

The relevant profitability indicators to be considered for the sensitivity analysis are as a minimum FIRR/C and corresponding FNPV are calculated. The same is applied for economic indicators ENPV and EIRR.

The sensitivity analysis consists of three steps, with the result of each one of them having to be reflected in the application for funding:

**Stage 1 - Identification of key variables:** This step implies the identification of those variables that are considered critical for the sustainability of the project outcomes. It is done by calculating the values of the profitability indicators after variations of +/- 1 % in a set of project variables.

**Stage 2 - Calculation of switching values for the key variables:** Given the results of step 1, any project variable for which a variation of 1 % results in a variation of more than 5 % in the value of the base case FNPV or ENPV will be considered a key variable. These key variables require the calculation of the so-called *switching value*, which is the maximum variation (in percentage) in the key variable that is needed to make the FNPV or ENPV (whichever is relevant for that specific key variable) turns zero.

**Stage 3 - Estimation of probability distribution for the profitability indicators:** This step implies a qualitative assessment of the relevant factors that may affect the values of the key variables as well as the mitigation measures already included in the project to reduce the impact of those factors.

## 2.2 Financial Analysis Assumptions

2.2.1 General Assumptions

In the Financial Analysis preparation the following main assumptions are used:

- Reference periods of 30 years operational period and 5 years construction period from 2016.
- Discounting rate of 12%, recommended by the Client, calculated for the needs of this analysis;
- VAT of 17,5 %, according to Pakistan legal framework;
- Macroeconomic indicators (inflation, GDP, etc.), prognosed in the Study for Assessment of Passenger and Freight Traffic;
- Financing of the National co-financing by Government budget;
- Margin between income and O&M costs will be subsidized by the Government budget for stable railway company;
- Depreciations and contingencies are not used as a cash flow.

2.2.2 Investment Costs Breakdown Assumptions

The total investment costs are estimated on the basis of the technical experts' calculation for the construction works and needed equipment.

Exchange rate of 1.0890 USD to EUR is used - .

Exchange rate of 104.71 PKR to USD is used for transfer from USD in PKR.

2.2.3 Operation and Maintenance (O&M) Costs

The methodology used for determination of O&M costs in the project included the next steps and parameters:

- Structure and amount of the current O&M cost for operation and maintenance of the Pakistan Railways for the whole railway network, cost classification, including Rolling Stock and Infrastructure activities.

  These analyses are based on historical data by P&L Statement and Yearbook of Pakistan Railways for 2012-2013, 2013-2014 and 2014/2015. From these analyses fixed costs and variable costs are determined;
- Structure and amount of the current O&M cost for operation and maintenance of the Pakistan Railways for the whole railway network Infrastructure are quantified. These analyses are necessary to forecast the costs of infrastructure by sections and for the whole line option;
- Structure and amount of the current O&M cost for operation and maintenance of the Pakistan Railways for the whole Rolling Stock. These analyses are necessary to forecast the costs of Rolling Stock by sections and for the whole line;
- Analysis and determination of structure and amount of the current cost for operation and maintenance of Pakistan Railways for the different sections and the whole line Kotri – Attock City;

2.2.4 Residual Value Assumptions

To determine the full value of revenues in the estimates for FIRR, it was necessary to define the amount of the Residual Value of the assets or the present value of expected net cash flow during the years of economic life outside the reference period.

As already mentioned the forecast period is 35 years, out of which 30 years being the effective time of use of the assets for the network infrastructure and equipment.

2.2.5 Revenues Assumptions

The income covered cost for the both main activities – transport services and railway infrastructure management.

Analyses of the Operation revenue by activities show that the main group of income is from passenger transport services – more than 69%. Freight transport services take only 15% in the total income.

2.2.6 Calculations for the Financial Analysis Assumptions

The financial net present value, FNPV and the financial rate of return, FIRR on the total investment cost, measure the performance of the investment independently of the sources or methods of financing.

The FNPV is expressed in monetary terms (USD), and depends on the scale of the project. The second indicator is a pure number, and is scale-invariant. The preferred indicator should usually be the net present value because the rate of return may be somewhat misleading and contains no useful information about the 'value' of a project.

A discount rate of 12% was applied to calculate FNPV. For FNPV estimations, the incremental revenue and operating costs are used.

## 2.3 Economic Analysis Assumptions

2.3.1 General Assumptions

Main general assumptions for Economic analyses are as follows:

- Load factor, important for transferring of benefits from traffic forecasts
- Type of engine, which is very important for externalities determination – noise, air pollution etc.
- Traffic structure by mode of transport and zone of moving, which is used in VoC, noise and climate changes determinations is shown in the table below:

2.3.2 Benefits Assumptions

2.3.2.1 Value of Time (VoT) Assumptions

The key inputs for the derivation of the value of work time are shown in the table below. These have been estimated based on data freely available from the different official information sources:

***Table 1:*** Key indicators for VoT calculation

| Key indicators | Value |
|---|---|
| Average working time | 46.37h per week |
| GNI per capita | 1,250 $ |

| Key indicators | Value |
|---|---|
| GDP per capita | 1,252 $ |
| Unemployment rate, | 6.5% |
| Correction for the poverty headcount ratio | 12.4% |
| Average monthly salary | 63,067 PKR or 624.63 $ |

2.3.2.2 Vehicle Operating Cost (VOC) Assumptions

The benefits of reduced costs for consumers are cumulated by the following effects:

- Reduced distance with the construction of the railway line;
- Saved costs for cars consumptions for diverted traffic;
- Saved costs for car maintenance for diverted traffic from road.

2.3.2.3 Accident Costs Assumptions

Unit value of marginal costs for accidents of EU members countries are transferred to Pakistan through PPP, exchange rate and GDP growth. After the calculations the unit value per passenger km was determined:

***Table 2:*** Unit values of accidents per passenger km (pkm), prices to 2015

| Transport mode | USD/100pkm | | | |
|---|---|---|---|---|
| | **Motorways** | **Rural** | **Urban** | **Average** |
| Cars | 1,2192 | 1,36 | 4,5421 | 3,5000 |
| Buses | 0,9811 | 0,73 | 4,1825 | 3,2600 |
| Rail | | | | 0,1434 |
| **Differences – incremental** | | | | |
| Cars | 1,0758 | 1,22 | 4,3986 | 3,3566 |
| Buses | 0,8377 | 0,59 | 4,0391 | 3,1166 |

Source: Own calculations

2.3.2.4 Other External Benefits Assumptions

In the other benefits, named externalities are included air pollution, noise, soil and water pollution etc.

The benefits of reducing **air pollution** are accumulated from the effects:

- Increased speed with the possibility to maintain optimum and ecological speed of travel;
- Diverting passengers from road to railway;
- Increased traffic has an adverse impact, which cannot be compensated in some cases.

2.3.3 Benefits from Job creation

Every major investment initiative creates jobs in two main directions:

- during construction, by involving workers who have not been hired because construction companies concentrate its capacity for ongoing projects, but now they require additional people. The permanent staff of construction companies is not an additional benefit from this project because they will use staff anyways, if not this project, then in another, but the newly-hired workers are within the scope of the study
- jobs to operate the newly created assets– since the line is already operational and the project does not envisage any new rolling stock purchases, there will be no new jobs created for the operation of the newly created assets

2.3.4 Economic Investment Costs of selected option Assumptions

For fiscal correction of investments and O&M costs conversion factors are used:

***Table 3:*** Standard conversion Factor

| Fiscal corrections- investments costs | Value |
|---|---|
| Labour | 11,76% |
| Materials | 50,63% |
| Equipment | 10,45% |
| Other costs | 5,81% |
| Conversion factor | 63,65% |

Source: Own calculation and structure of P&L statement of Pakistan Railways

2.3.5 Economic O&M Costs of selected option Assumptions

The same approach as for Economic investment costs of fiscal correction was applied for O&M costs and **SCF =0.6365** was used.

2.3.6 Economic Indicators Assumptions

The economic net present value (ENPV) is the main indicator used to assess the economic viability of the project. It gives information about the difference between the discounted total benefits and costs of a particular project.

## 2.4 Sensitivity Analysis Assumptions

2.4.1 Sensitivity Analysis of the Financial Indicators

Sensitivity Analysis evaluates the degree of changes which take place in the individual financial indicators, in event of a change of any of the main parameters with regard to those selected in the project. This influence is investigated for the financial net present value of the investments and FIRR of the investments.

In the specific project evaluates the effect from the various variations investigating their influence over FNPV of the investments and the capital, and of the FIRR of the investments and the capital in events of a change of the following parameters:

- Investment costs;

- Operation and Maintenance costs;
- Revenues

The critical variables were additionally determined at which a change of the parameters with ±1, may result in a changes of the financial indicators values with more than 5% for FNPV and 1% for FIRR.

2.4.2 Sensitivity Analysis of the Economic indicators

All variables, which influence on the indicators ENPV and EIRR of the investment project shall be investigated for sensitivity. Sensitivity Analysis was also made for the benefit/cost coefficient.

In this project the investigated variables are the following:

- Economic Investment costs
- Operation and Maintenance Economic costs
- Benefits from Value of Time (VoT)
- Benefits from Vehicle Operating Costs (VoC)
- Benefits from reduction of railway accidents
- External costs for the Environment.
- Job creation

## 2.5 Financial and Economic Analysis for Main line -2

2.5.1 Investment Costs

The total length of Main Line 2 is 1254 km and includes all 5 sections of the line, listed in sub-paragraph 1.1 above. Total investments exceed 2.7 billion USD (232,047milion PKR), incl. costs for Rolling stock. The detailed investments by group of assets are presented in the table below:

***Table 4:*** Total investment costs of Main Line 2 in USD and million PKR

| Phases | Investments by stages and scenarios | Main Line 2 | Main Line 2 |
|---|---|---|---|
| | | Kotri – Attock city (USD) | Kotri – Attock city (mln PKR) |
| **1.** | **Land acquisitions** | **52,049,000** | **5,030.98** |
| **2.** | **FS and Detailed Design** | **57,733,440** | **4,961.42** |
| **3.** | **Construction works** | **1,924,448,000** | **325,551.38** |
| 3.1. | Equipment, tunnels and bridges | 284,262,000 | 165,380.65 |
| 3.2. | Infrastructure | 1,239,222,000 | 25,001.71 |
| 3.3. | Buildings | 62,936,000 | 107,098.20 |
| 3.4. | Signalling | 336,660,000 | 27,957.22 |
| 3.5. | Telecommunication | | |

| Phases | Investments by stages and scenarios | Main Line 2 | Main Line 2 |
|---|---|---|---|
| | | Kotri – Attock city (USD) | Kotri – Attock city (mln PKR) |
| 3.6. | Power Supply | 1,368,000 | 113.61 |
| 4.1. | Contingencies | 395,299,400 | 34,082.33 |
| 4.2. | Supervision, Documentation & Conservation of Evidence | 96,222,400 | 8,269.03 |
| **4.** | **Other costs, incl. contingencies, supervision, design, FS etc.** | **601,304,240** | **52,343.76** |
| **5.** | **Rolling stock** | **192,665,457** | **17,753.18** |
| **6.** | **TOTAL COSTS** | **2,718,417,697** | **232,047.74** |

This amount for investments was used in all calculations in the CBA Report.

2.5.2 Residual Value

Residual value obtained for the project is presented in the following tables:

***Table 5:*** Financial Residual value for Main Line 2

| Assets by type of life cycle | Useful life in years | % | Using period according to forecast period | Annual amount for deprecations | Residual value |
|---|---|---|---|---|---|
| Equipment, tunnels and bridges | 100 | 1.0% | 30 | 2,842,620 | 198,983,400 |
| Infrastructure | 50 | 2.0% | 30 | 24,784,440 | 495,688,800 |
| Buildings | 50 | 2.0% | 30 | 1,258,720 | 25,174,400 |
| Signalling | 15 | 6.7% | 30 | 22,444,000 | 0 |
| Telecommunication | 20 | 5.0% | 30 | 0 | 0 |
| Power Supply | 20 | 5.0% | 30 | 68,400 | 615,600 |
| Rolling stock | 50 | 2.0% | 30 | 3,853,309 | 77,066,183 |
| Contingencies | 50 | 2.0% | 30 | 7,905,988 | 158,119,760 |
| Other costs | 44 | 2.3% | 30 | 13,550,518 | 194,788,697 |
| Total Financial residual value | | | | | 1,150,436,840 |
| Discounted residual value | | | | | 67,672,499 |

2.5.3 Operation and Maintenance (O&M) Costs

O&M costs for this section are calculated according to described methodology in sub-paragraph above, taking into account:

- Traffic forecast for all sections along Main Line 2 in train-km for Do minimum and With project scenario and incremental forecast for the section
- Unit variable and fixed costs determined in Chapter 7.3.3 per train-km
- Length of Main Line 2 is 1254 km

2.5.4 Revenues

Based on the described methodology and analysis of revenue is determined the unit cost of pax. This unit value multiplied by the traffic on Main Line 2 results in the revenue per year.

The cost of traveling on the designated section is the result of the average unit cost and the distance that a passenger must travel. Cumulative repetition of destination has been taken in consideration and corrected.

Results for Main Line 2 are:

***Table 6:*** Ticket prices of Main Line 2

| Destination | Unit income | Ticket price | |
|---|---|---|---|
| | USD/pkm | in USD | In PKR |
| Section 1 | 0.0141 | 4.89 | 512.38 |
| Section 2 | 0.0146 | 2.58 | 269.92 |
| Section 3 | 0.0160 | 4.89 | 512.38 |
| Section 4 | 0.0112 | 2.58 | 269.92 |
| Section 5 | 0.0133 | 2.58 | 269.92 |
| Total line | 0.0076 | 9.54 | 998.98 |

The detailed forecast is shown is Appendix A-2, but the summarized figures are the next:

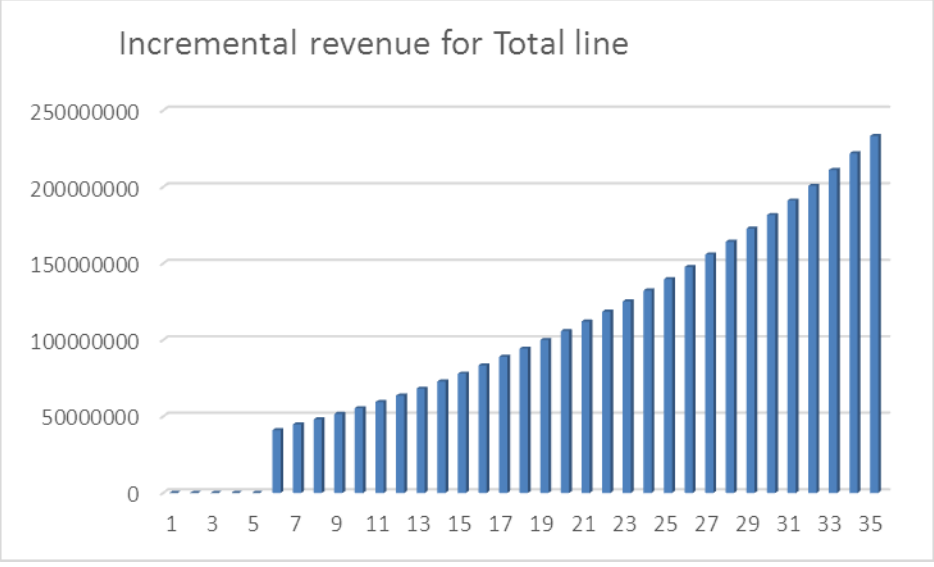



***Figure 3:*** Revenue by years of Main Line 2

Revenue for Main Line 2 over the reference period is 118.77 million USD average per year and the coefficient of coverage of the costs by the revenues 127.20%, indicating that the Total Line could cover its operating costs with its revenues. This is a very good coverage ratio for a railway project.

2.5.5 Financial Indicators and Financial Plan

2.5.5.1 Financial Indicators

The FNPV and FIRR indicators show the possibilities of the Net Revenue to compensate the Investment costs, without considering the sources of financing.

***Table 7:*** Financial Indicators of Main Line 2

| Financial indicators | Measure | Main Line 2 | Measure | Main Line 2 |
|---|---|---|---|---|
| | mln USD | -1,106.89 | mln PKR | -240,872 |
| mln USD | (%) | -2.11% | (%) | -2.11% |

The financial indicators show that the project insofar as Main Line 2 cannot recover the cost of investment over the period and is in need of additional funding.

This result is expected for railway projects, where the financial return is relatively slow – such projects are usually implemented not for their financial but for their economic benefits to society.

2.5.5.2 Financial Plan and Loan conditions

The necessary amount for the financing by IFIs is determined by using the Funding Gap approach. This is made for the whole line and transferred to each section as a 65% of total investments costs.

The main assumptions for the loan conditions are resumed in the next table:

***Table 8:*** Loan conditions for Main Line 2

| Item | Value |
|---|---|
| Total amount for investments in USD: | 2,718,417,697 |
| Loan amount required in USD: | 1,766,971,503 |
| Interests in %: | 0.88% |
| Grace period in years: | 5 years |
| Commitment fee 0.25% on the loan amount: | 441,743 |

Interests are determined according to last published conditions of World Bank and last data for value of LIBOR. After application of the conditions for funding sources, financial plan for the Main Line 2 is as follows:

***Table 9:*** Financial plan for Main Line 2

| Main Line 2 | 2016 | 2017 | 2018 | 2019 | 2020 | 2021 | Total |
|---|---|---|---|---|---|---|---|
| **Financial plan** | **10,978,244** | **282,820,014** | **705,742,869** | **815,525,309** | **903,351,261** | **0** | **2,718,417,697** |
| National co-financing | 10,978,244 | 91,851,146 | 247,010,004 | 285,433,858 | 316,172,941 | 0 | 951,446,194 |
| IFIs loan | 0 | 190,968,868 | 458,732,865 | 530,091,451 | 587,178,320 | 0 | 1,766,971,503 |
| Inflation | 0.00% | 2.85% | 3.03% | 3.23% | 3.44% | 4.46% | |
| Adjusted price | | 69,353,644 | 101,739,249 | 26,337,951 | 31,030,542 | 0 | 2,946,879,082 |

### 2.5.6 Economic Benefits

#### 2.5.6.1 Value of Time (VoT)

Time savings for Main Line 2 are defined in the traffic forecasts for the transferred traffic of buses and cars, as well as existing rail traffic.

***Table 10:*** Saving time in minutes for Main Line 2

| Mode of transport | Section 1 Kotri – Habib Kot | Section 2 Habib Kot – Kashmor | Section 3 Kashmor – Kot Adu | Section 4 Kot Adu – Kundian | Section 5 Kundian – Attock city | Main Line 2 |
|---|---|---|---|---|---|---|
| Bus/Coach | 98.00 | 61.50 | 119.00 | 133.50 | 167.00 | 439.00 |
| Car | 98.00 | 61.50 | 119.00 | 133.50 | 167.00 | 439.00 |
| Rail-existing pass | 464.69 | 224.37 | 157.35 | 124.61 | 177.17 | 1039.79 |
| Rail-existing freight | 712.19 | 339.59 | 190.45 | 162.06 | 242.24 | 1410.61 |

When applying a unit values calculated in sub-section 2.3.2.1 above and the rule of half of the projected traffic on the line, the benefits of reducing travel time are:

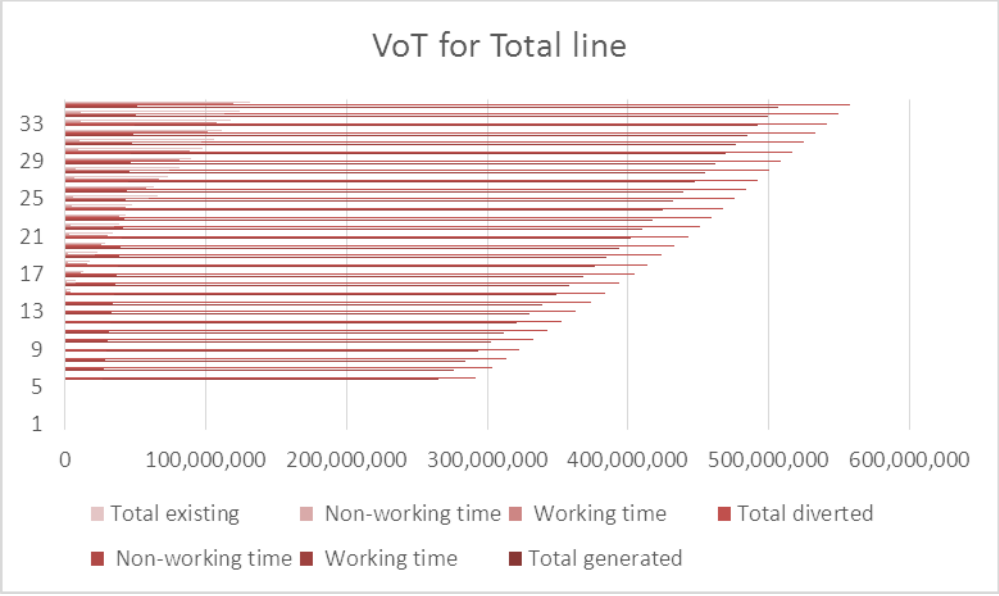



***Figure 4:*** VoT for Main Line 2

The benefits from time savings are mainly from traffic diverted from the road to the railway.

The largest benefit of this Main Line 2 is the value of time benefits, which amount to 45% of the total economic benefits.

2.5.6.2 Vehicle Operating Cost (VOC)

In order to determine the VoC the consultant has used the HDM RED model. The model's input parameters include data from each of the sections, which consists of the following:

- Length of the road section
- Condition of the road section
- Desired maximum speed
- Economic unit costs such as vehicle costs, fuel and lubricant costs
- Projected vehicle utilization in hours and km driven
- Vehicle service life
- etc.

Summarized results are shown in the following figure:

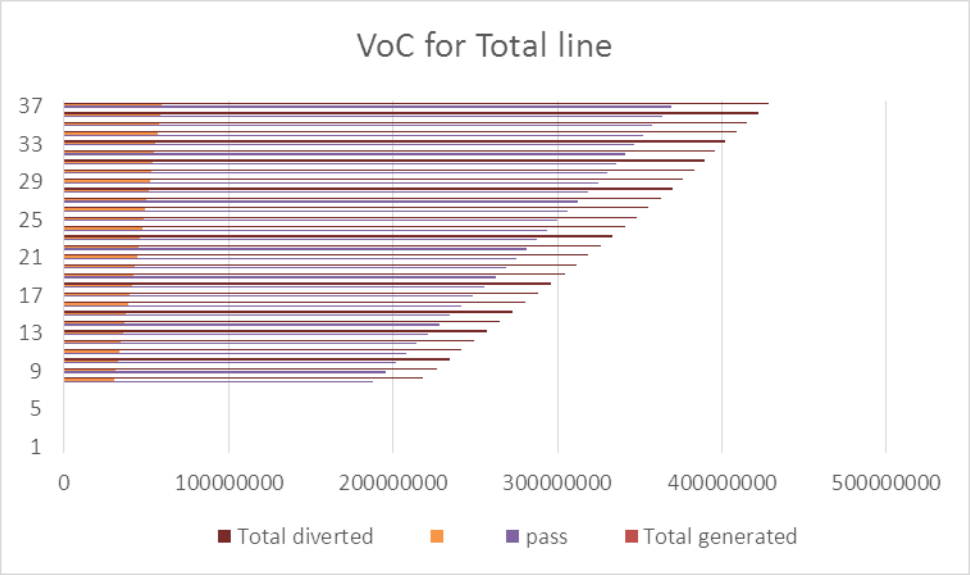



***Figure 5:*** VoC for Main Line 2

It is apparent from the graph that the main benefits are a result from road traffic transferred to the railway – there is no generated traffic.

Benefits from vehicle operating costs rank the second-highest of the benefits for this Main Line 2 with 33% of all economic benefits.

2.5.6.3 Accident Costs

The benefits of reducing the number of accidents in Main Line 2 are a result of transferred road traffic. As described above and the traffic forecasts, accident savings over time are presented in the following graph:

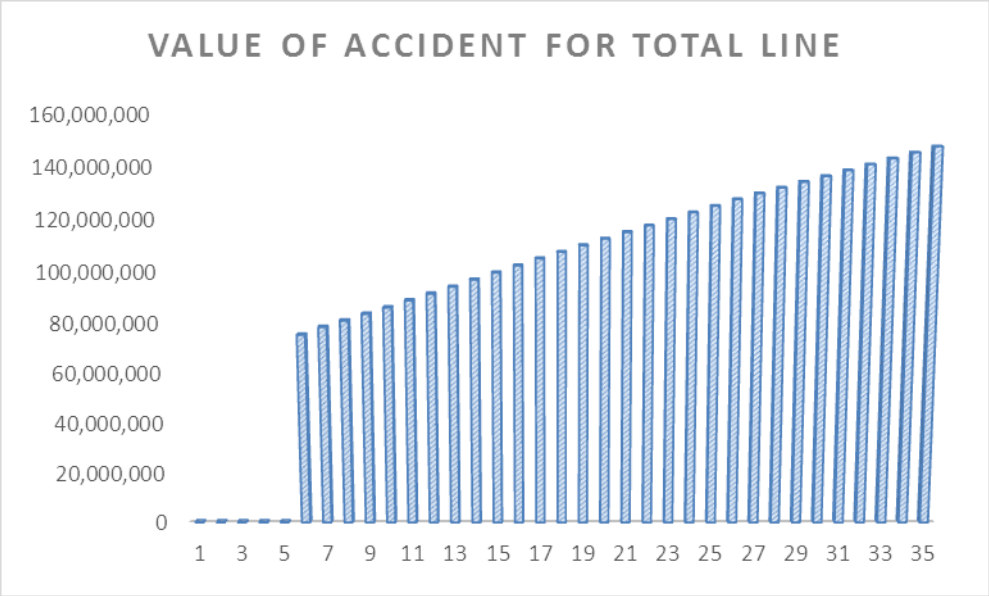



***Figure 6:*** Value of accidents for Main Line 2

Main Line 2 includes all sections, therefore, the benefits of reducing the number of accidents in this area are a sum of all the benefits from reduction of accidents along Sections 1, 2, 3, 4 and 5.

2.5.6.4 Environmental Benefits

Similar to benefits from accidents, the benefits of reducing environmental impact in the area are a sum of the benefits from Sections 1, 2, 3, 4 and 5.

Summarized results of the VoE benefits on Main Line 2 are shown in the following figure:

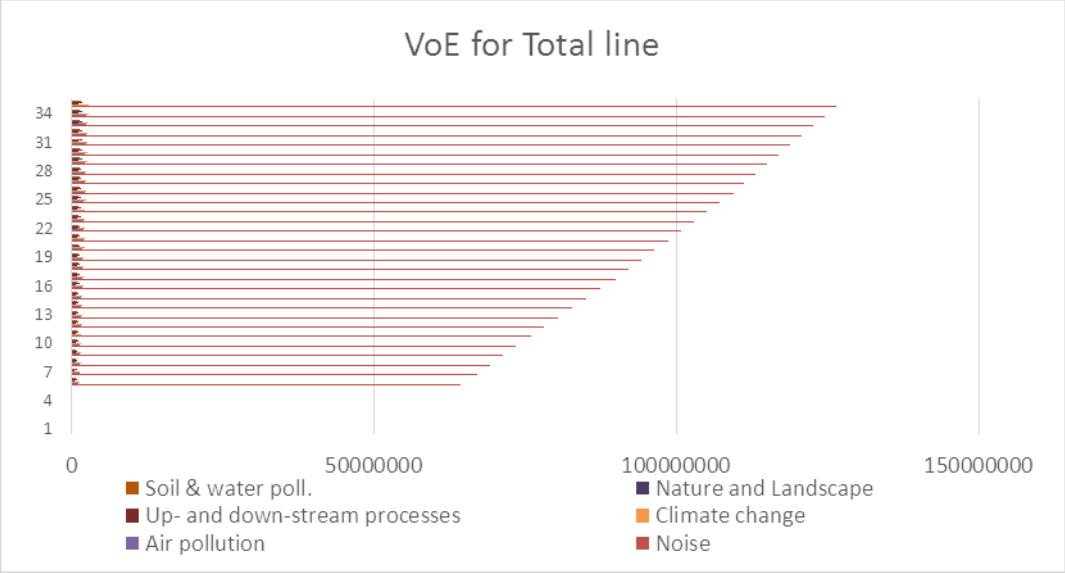



***Figure 7:*** Environmental benefits for Main Line 2

Reduction of noise has the greatest contribution – it forms over 90% of the total VoE benefits.

2.5.6.4 Benefits from Job Creation

In Main Line 2 the benefits of creating new jobs are associated solely with the construction period because the line is currently operational. The methodology approach derives the number of jobs created from the total investment – 51.33 million USD on average per year – the highest, since Main Line 2 includes all Sections and cumulates the benefits of each. Results from the estimated number of new jobs in this section are shown in the following table:

***Table 11:*** Benefits of Job Creation for Main Line 2

| Main Line 2 | 2016 | 2017 | 2018 | 2019 | 2020 | 2021 |
|---|---|---|---|---|---|---|
| Value of Investments | 10,978,244 | 282,820,014 | 705,742,869 | 815,525,309 | 903,351,261 | 0 |
| number of employed | 1,323 | 34,075 | 85,029 | 98,256 | 108,838 | 0 |
| Average salary in PKR per month | 42,567 | 44,653 | 46,885 | 49,230 | 51,691 | 54,276 |
| Average salary in USD per month | 407 | 416 | 426 | 436 | 448 | 460 |
| Average salary in USD per year | 4,878 | 4,997 | 5,118 | 5,235 | 5,374 | 5,522 |
| **Benefit - 15% of salary** | **967,904** | **25,541,775** | **65,272,842** | **77,149,549** | **87,740,984** | **0** |
| GDP Growth | 4.80% | 4.90% | 5.00% | 5.00% | 5.00% | 5.00% |
| Exchange rate | 104.71 | 107.23 | 109.94 | 112.86 | 115.42 | 117.94 |

During the period of construction it is expected that 6185 new jobs will be created for this section.

2.5.6.5 Economic Investment Costs and Residual value

Economic investment costs for Main Line 2 are obtained by applying the standard conversion factor (SCF) defined in the general assumptions for the project as a whole. The breakdown of the investment costs by elements is as follows:

***Table12:*** Economic Investments costs for Main Line 2

| Year | Labour | Materials | Equipment | Other costs | Total investments costs |
|---|---|---|---|---|---|
| 2016 | 1,894,353 | 3,607,725 | 1,079,592 | 405,983 | 6,987,652 |
| 2017 | 48,802,050 | 92,941,713 | 27,812,308 | 10,458,868 | 180,014,939 |
| 2018 | 121,779,567 | 231,924,715 | 69,402,224 | 26,098,830 | 449,205,336 |
| 2019 | 140,723,092 | 268,001,964 | 80,198,147 | 30,158,656 | 519,081,859 |
| 2020 | 155,877,912 | 296,863,763 | 88,834,886 | 33,406,517 | 574,983,078 |

According to Article 18 (Residual value of the investment) of Commission Delegated Regulation (EU) No 480/2014, for project assets with economic lifetimes in excess of reference period, their residual value shall be determined by 'computing' the net present value of cash flows in the remaining life years of the operation. This approach is applied for Economic residual value calculations.

Economic life of new assets is 50 years, but prognosis period is 30 and their net present value for the rest 20 years is used.

Results are shown in the next table:

***Table 13:*** Economic Residual value for Main Line 2

| Residual value -Economic value for Main Line 2 | Financial Residual value | Economic Residual value |
|---|---|---|
| Equipment, tunnels and bridges | 198,983,400 | 135,252,741 |
| Infrastructure | 495,688,800 | 336,928,955 |
| Buildings | 25,174,400 | 17,111,511 |
| Signalling | 0 | 0 |
| Telecommunication | 0 | 0 |
| Power Supply | 615,600 | 418,435 |
| Contingencies | 158,119,760 | 107,476,960 |
| Other costs | 194,788,697 | 132,401,523 |
| Total residual value | 1,150,436,840 | 781,973,452 |
| Economic Residual value | | 781,973,452 |

2.5.6.6 Economic O&M Costs

The same approach and SCF as for the economic investment costs is applied in determining the economic O&M costs.

2.5.6.7 Economic Indicators

The economic indicators for this section are presented in the following table:

***Table 14:*** Economic indicators of Main Line 2

| **Indicators** | **Main Line 2** |
|---|---|
| **ENPV in USD** | 2,678,634,255 |
| **EIRR in %** | 32.549% |
| **B/C ratio** | 2.94 |

The values of these indicators show that the project is viable for this section because:

- ENPV is positive;
- EIRR is > 12%, the used discount rate;
- The B/C coefficient is > 1.0

2.5.7 Sensitivity Analysis of the Financial indicators

Sensitivity Analysis for Main Line 2 of Financial indicators was prepared according to the methodology described in section 2.1.4 above and the results of the financial indicators are shown in the table below:

***Table 15:*** Values of tested variables changes for Main Line 2

| Tested variables | FNPV | FIRR | % deviation of NPV | % deviation of IRR |
|---|---|---|---|---|
| Base scenario | -1,106,793,237 | -2.11% | 0.00% | 0.00% |
| Increase of Investments with 1% | -1,113,411,881 | -2.12% | -0.60% | -0.60% |
| Decrease of Investments with 1% | -1,100,174,593 | -2.10% | 0.60% | 0.60% |
| Increase of Revenue with 1% | -1,103,122,128 | -2.01% | 0.33% | 4.64% |
| Decrease of Revenue with 1% | -1,110,464,345 | -2.21% | -0.33% | -4.70% |
| Increase of O&M costs with 1% | -1,110,478,114 | -2.16% | -0.33% | -2.66% |
| Decrease of O&M costs with 1% | -1,103,108,360 | -2.05% | 0.33% | 2.66% |

The critical variables for IRR are Revenue and O&M costs. There are no critical variables for NPV. The definition for a “critical variable” is a 1% change, which leads to more than 1% change in NPV and IRR. Switching values for the critical variables is performed below:

***Table 16:*** Switching values of critical variables for Main Line 2

| Variables | Switching value indicators | Value |
|---|---|---|
| **Investments** | Maximum increase before NPV<0 | already negative |
| | Maximum increase before IRR<12% | already<12% |
| | Minimum decrease before NPV>0 | no critical |
| | Minimum decrease before IRR>12% | no critical |
| **Revenue** | Maximum increase before NPV>0 | 302% |
| | Maximum increase before IRR>12% | 302% |
| | Minimum decrease before NPV<0 | already negative |
| | Minimum decrease before IRR<12% | already<12% |
| **O&M costs** | Maximum increase before NPV<0 | already negative |
| | Maximum increase before IRR<12% | already<12% |
| | Minimum decrease before NPV>0 | -301% |
| | Minimum decrease before IRR>12% | -301% |

Changes of financial indicators, concerning variables’ changes with deviation ±10% and ±20% are prepared. From the table below it is clear that revenues have most impact on financial profitability of the project, followed closely by investments and O&M costs:

***Table 17:*** Changes of FNPV and FIRR for Main Line 2

| Variables | FNPV | FIRR |
|---|---|---|
| Investment costs -20% | -974,420,363 | -1.84% |
| Investment costs -10% | -1,040,606,800 | -1.98% |
| Investment costs 0% | -1,106,793,237 | -2.11% |
| Investment costs +10% | -1,172,979,674 | -2.23% |
| Investment costs +20% | -1,239,166,110 | -2.35% |
| O&M Costs –20% | -1,033,095,696 | -1.01% |
| O&M Costs –10% | -1,069,944,467 | -1.55% |
| O&M Costs 0% | -1,106,793,237 | -2.11% |
| O&M Costs +10% | -1,143,642,007 | -2.68% |
| O&M Costs +20% | -1,180,490,777 | -3.26% |
| Revenue -20% | -1,180,215,405 | -4.39% |
| Revenue -10% | -1,143,504,321 | -3.16% |
| Revenue 0% | -1,106,793,237 | -2.11% |
| Revenue +10% | -1,070,082,153 | -1.18% |
| Revenue +20% | -1,033,371,068 | -0.35% |

2.5.8 Sensitivity Analysis of the Economic indicators

The results of the Sensitivity Analysis of Economic indicators are presented in the tables shown below:

***Table 18:*** Tested Economic variables for Main Line 2

| Tested variables | ENPV | EIRR | % deviation of NPV | % deviation of IRR |
|---|---|---|---|---|
| Base scenario | 2,678,634,255 | 32.55% | 0.00% | 0.00% |
| Increase of Investments with 1% | 2,667,377,975 | 32.29% | 0.42% | 0.80% |
| Decrease of Investments with 1% | 2,689,890,535 | 32.81% | -0.42% | -0.81% |
| Increase of O&M costs with 1% | 2,676,129,575 | 32.53% | 0.09% | 0.05% |
| Decrease of O&M costs with 1% | 2,681,138,935 | 32.56% | -0.09% | -0.05% |
| Increase of VoE with 1% | 2,682,632,936 | 32.57% | -0.15% | -0.08% |
| Decrease of VoE with 1% | 2,674,635,574 | 32.52% | 0.15% | 0.08% |
| Increase of VoA with 1% | 2,682,958,050 | 32.58% | -0.16% | -0.08% |
| Decrease of VoA with 1% | 2,674,310,460 | 32.52% | 0.16% | 0.08% |
| Increase of VoC with 1% | 2,691,718,790 | 32.63% | -0.49% | -0.25% |
| Decrease of VoC with 1% | 2,665,549,720 | 32.47% | 0.49% | 0.25% |
| Increase of VoT with 1% | 2,695,891,916 | 32.65% | -0.64% | -0.32% |

| Tested variables | ENPV | EIRR | % deviation of NPV | % deviation of IRR |
|---|---|---|---|---|
| Decrease of VoT with 1% | 2,661,376,593 | 32.44% | 0.64% | 0.32% |
| Increase of Job created with 1% | 2,676,107,667 | 32.51% | 0.09% | 0.12% |
| Decrease of Job created with 1% | 2,672,722,990 | 32.43% | 0.22% | 0.36% |

There are no critical variables for ENPV or EIRR. The definition for a "critical variable" is the same as for the Financial Sensitivity Analyses - 1% change, which leads to more than 1% change in ENPV and EIRR. Switching values for the critical variables is performed below:

***Table 19:*** Switching value of the critical variables of ENPV for Main Line 2

| Variables | Switching value indicators | Value |
|---|---|---|
| **Investments** | Maximum increase before NPV<0 | no critical |
| | Maximum increase before IRR<12% | no critical |
| | Minimum decrease before NPV>0 | already positive |
| | Minimum decrease before IRR>12% | already >12% |
| **O&M costs** | Maximum increase before NPV>0 | already positive |
| | Maximum increase before IRR>12% | already >12% |
| | Minimum decrease before NPV<0 | no critical |
| | Minimum decrease before IRR<12% | no critical |
| **VoE** | Maximum increase before NPV<0 | no critical |
| | Maximum increase before IRR<12% | no critical |
| | Minimum decrease before NPV>0 | already positive |
| | Minimum decrease before IRR>12% | already >12% |
| **VoA** | Maximum increase before NPV<0 | no critical |
| | Maximum increase before IRR<12% | no critical |
| | Minimum decrease before NPV>0 | already positive |
| | Minimum decrease before IRR>12% | already >12% |
| **VoC** | Maximum increase before NPV<0 | no critical |
| | Maximum increase before IRR<12% | no critical |
| | Minimum decrease before NPV>0 | already positive |
| | Minimum decrease before IRR>12% | already >12% |
| **VoT** | Maximum increase before NPV<0 | no critical |
| | Maximum increase before IRR<12% | no critical |
| | Minimum decrease before NPV>0 | already positive |
| | Minimum decrease before IRR>12% | already >12% |

| Variables | Switching value indicators | Value |
|---|---|---|
| **New job created** | Maximum increase before NPV<0 | no critical |
| | Maximum increase before IRR<12% | no critical |
| | Minimum decrease before NPV>0 | already positive |
| | Minimum decrease before IRR>12% | already >12% |

To emphasize the influence of each variable additional calculations have been made by changing each variable by ± 10% and ± 20%. From the table below it is clear that investments have most impact on economic viability of the project.

***Table 20:*** Deviation of all variables for Main Line 2

| Variables | ENPV | EIRR |
|---|---|---|
| Investment costs -20% | 2,903,759,857 | 38.99% |
| Investment costs -10% | 2,791,197,056 | 35.44% |
| Investment costs 0% | 2,678,634,255 | 32.55% |
| Investment costs +10% | 2,566,071,453 | 30.15% |
| Investment costs +20% | 2,453,508,652 | 28.11% |
| O&M Costs -20% | 2,728,727,852 | 32.86% |
| O&M Costs -10% | 2,703,681,054 | 32.71% |
| O&M Costs 0% | 2,678,634,255 | 32.55% |
| O&M Costs +10% | 2,653,587,456 | 32.39% |
| O&M Costs +20% | 2,628,540,657 | 32.24% |
| VoE -20% | 2,598,660,639 | 32.05% |
| VoE -10% | 2,638,647,447 | 32.30% |
| VoE 0% | 2,678,634,255 | 32.55% |
| VoE +10% | 2,718,621,063 | 32.80% |
| VoE +20% | 2,758,607,871 | 33.05% |
| VoA -20% | 2,592,158,353 | 32.01% |
| VoA -10% | 2,635,396,304 | 32.28% |
| VoA 0% | 2,678,634,255 | 32.55% |
| VoA +10% | 2,721,872,206 | 32.82% |

| Variables | ENPV | EIRR |
|---|---|---|
| VoA +20% | 2,765,110,157 | 33.09% |
| VoC -20% | 2,416,943,555 | 30.90% |
| VoC -10% | 2,547,788,905 | 31.73% |
| VoC 0% | 2,678,634,255 | 32.55% |
| VoC +10% | 2,809,479,605 | 33.36% |
| VoC +20% | 2,940,324,955 | 34.15% |
| VoT -20% | 2,333,481,025 | 30.40% |
| VoT -10% | 2,506,057,640 | 31.49% |
| VoT 0% | 2,678,634,255 | 32.55% |
| VoT +10% | 2,851,210,870 | 33.59% |
| VoT +20% | 3,023,787,485 | 34.61% |
| Job creation -20% | 2,640,568,553 | 31.72% |
| Job creation -10% | 2,657,491,941 | 32.09% |
| Job creation 0% | 2,678,634,255 | 32.55% |
| Job creation +10% | 2,691,338,716 | 32.86% |
| Job creation +20% | 2,708,262,104 | 33.27% |

2.5.9 Conclusions for Main Line 2

The revenues of Main Line 2 can fully cover its O&M costs. With a coefficient of coverage of 127%, the line will require no government subsidy to maintain its operations once the project has been realized.

The line is not financially profitable due to large investment amount required to implement the project. If the Pakistani government ensures that the project's loans are properly serviced, the project is financially sustainable.

The line is economically feasible with its highest economic benefits being the time savings from reduction of travel time from diverted and existing traffic on the railway. It is important to note that the time savings benefits are cumulated when all sections are implemented, therefore the total benefits of the line greatly exceed the sum of the benefits of each individual section, should the sections be implemented individually.

The financial risk of this section is low – it would require a 302% reduction in revenues or 301% increase in O&M costs for the project to become financially unsustainable.

The economic risk is very low – none of the variables are critical, which means that the project is not sensitive to changes in any variable.

## 2.6 Conclusions

The Project "Upgradation/Rehabilitation of Main Line-2 from Kotri to Attock City" includes upgrade/rehabilitation of five sections of the Pakistan railway network, indicated in sub-section 1.1 above. All sections can be upgraded/rehabilitated individually, in combination with each other and consecutively.

Main results from all scenario are presented in the table below:

***Table 21:*** Resume of main results from Financial and Economic analyses

| Name of indicators | Value of indicators | | | | | |
|---|---|---|---|---|---|---|
| | **Section 1** | **Section 2** | **Section 3** | **Section 4** | **Section 5** | **Total Line** |
| FNPV C | -277,033,651 | -148,276,593 | -256,552,202 | -184,203,579 | -196,046,513 | -1,106,793,237 |
| FIRR C | -2.90% | -3.29% | -2.23% | -1.75% | -0.59% | -2.11% |
| ENPV | 205,647,556 | 103,595,003 | 176,449,515 | 400,048,302 | 525,018,774 | 2,678,634,255 |
| EIRR | 18.85% | 19.18% | 19.07% | 30.75% | 33.68% | 32.55% |
| B/C | 1.53 | 1.58 | 1.55 | 2.69 | 2.93 | 2.94 |
| Financial sustainability | not, without State aid | | | | | |
| Financial Gap rate | not applicable | not applicable | not applicable | not applicable | not applicable | not applicable |
| Costs coverage ratio by Revenue | 112.06% | 106.37% | 132.31% | 133.71% | 144.64% | 127.20% |

*Source:* Own calculation

The results of the Financial and Economic analyses demonstrate that if a choice for upgrade/rehabilitation of individual sections is made, all sections are economically viable for individual upgradation/rehabilitation – Sections Kot Adu - Kundian and Kundian – Attock city have the highest economic viability.

All sections and the entire line have negative financial indicators with negative FNPV and FIRR lower than the used discount rate of 12%. This is the basis for the conclusion that sources for financing the project must be sought and the revenue generated cannot sustain the project without the aid of the Pakistan Government.

The necessary external financial resources that should be provided in the form of grant or IFIs loan represents 65% of the total investments, as determined by funding gap method.

Projected ticket prices are presented in the table below. These prices are weighted average between the different classes and are comparable to the current ticket prices.

| Length | km | Ticket price in USD with project | Ticket price in USD without project |
|---|---|---|---|
| Section 1 | 346,00 | 4,89 | 4,28 |
| Section 2 | 177,00 | 2,58 | 2,25 |
| Section 3 | 306,00 | 4,39 | 4,28 |
| Section 4 | 231,00 | 2,78 | 2,25 |
| Section 5 | 194,00 | 1,96 | 2,25 |
| Total line | 1 254,00 | 9,54 | 8,34 |

Given the above assumption about ticket prices and revenues thereof, the Income/O&M costs coverage ratio for the whole line is 120.59%. The O&M costs coverage ratios for each section are presented in the table below:

| Indicators | section 1 | section 2 | section 3 | section 4 | section 5 | Total line |
|---|---|---|---|---|---|---|
| Coverage ratio Income / O&M costs | 112.06% | 106.37% | 132.31% | 133.71% | 144.64% | 127.20% |
| Average Revenue per year | 31,865,410 | 15,770,874 | 29,604,989 | 27,100,207 | 40,852,819 | 145,194,298 |
| Average O&M costs per year | 28,435,164 | 14,826,335 | 22,375,630 | 20,267,554 | 28,245,038 | 114,149,721 |

The most significant contribution is economic benefits from mitigation of environmental adverse effects and on the vehicle operating costs. The modal shift to rail will sharply reduce noise and harmful gas emissions by large trucks on the road as well as the cost of road maintenance and air pollution.

The project for the entire line will lead to significant benefits for the society and consumers of railway transport, which greatly exceed the cost of upgrade/rehabilitation of the railway line in subject.

The savings include over 1.3 billion USD in costs for fuels, lubricants, spare parts and others for cars and buses from the transferred traffic, and over 1.7 billion USD from time savings as a result of the increased speeds on the railway line as a whole for the forecasted period.

The sensitivity analysis examined the project's sensitivity to changes in the discount rate. The results of the analysis demonstrate that the project will remain economically feasible even if the discount rate reaches 30%. Such high discount rates are very rarely applied and are an indication of a serious economic crisis in the country and significant risks for the investment.

Based on the results of the financial, economic and sensitivity analysis, the following are the main conclusions for this project:

- The railway line is economically feasible and will bear significant benefits for society – an enormous benefit from travel time savings, vehicle operating costs and road maintenance costs reduction, environmental benefits and benefits from reduction of noise, air pollution and others. Additionally, during construction the project will generate benefits from job creations that amount to $55 million.
- The project is capable of generating revenues that cover the O&M costs of all sections individually, and for the whole line the coverage ratio is over 127%.
- Provided that the Pakistani government assists Pakistan Railways with the repayment of the IFIs loan, the project is financially sustainable and bears acceptably low levels of risk.

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**Paper No. 789**

# INTERPRETATION OF PROBLEMATIC SONOGRAPHS IN SONIC INTEGRITY TESTING OF PILES

**Sohail Kibria, Asim Masud & M. Tayyab Javed**

# INTERPRETATION OF PROBLEMATIC SONOGRAPHS IN SONIC INTEGRITY TESTING OF PILES

By:

**Sohail Kibria[1], Asim Masud[2] & M. Tayyab Javed[3]**

## ABSTRACT:

Low strain impact integrity testing / Sonic Integrity Testing (S.I.T.) is finding its increasing use for the integrity evaluation of cast-in-place concrete piles in Pakistan. S.I.T. is considered very useful in assessing the quality/integrity of bored piles. State-of-the-art S.I.T. equipment with latest software provisions is being used for evaluating the integrity of piles in Pakistan.

Occasionally, the sonographs resulting from S.I.T. in the field and after adjustment by signal processing techniques are not easily comprehensible to delineate soundness/unsoundness of the tested piles. Our experience of testing more than 10,000 piles of various projects in Pakistan over a span of about five years, by team of experienced engineers, has indicated some serious difficulties/limitations in proper interpretation of some of the sonographs. Attempts have been made to correlate the signal of such sonographs with the type of possible defects, L/D ratios, pile head preparation, soil type, method of construction etc., keeping in view the pile construction records.

This paper describes ten (10) cases of selected sonographs, where serious difficulties were encountered in interpretation and were subsequently resolved based on our vast experience and using various techniques. The discussion in this paper and its findings are considered to provide useful guidelines for interpretation of S.I.T. signals, in similar testing and ground conditions.

**Key Works:** S.I.T., Interpretation, Sonographs, Integrity, Piles

## INTRODUCTION

Bored pile foundations are finding extensive applications in present-day civil engineering works in Pakistan. Cast-in-place concrete piles are often preferred as a deep foundation substitute where challenging/poor soil conditions exist. Amongst many available techniques, two types of construction methods of cast-in-place concrete piles, i.e., dry method and wet method and considered better. Mostly auger bucket is being used for dry method and reverse rotary with bentonite slurry is being used for wet method. It is very difficult or impossible to confirm the integrity of drilled shafts during construction by conventional methods.

Low strain impact integrity testing / Sonic Integrity Testing (S.I.T.) is finding its increasing use for the integrity evaluation of cast in-place concrete piles in Pakistan.

Low strain S.I.T. is a non-destructive technique. It involves applying low strain to a foundation element using light hammer impacts and evaluating the collected velocity records to deduce the qualitative and quantitative information for the foundation element. The S.I.T. equipment is very compact and readily portable, consisting of sensor/accelerometer, impact device/hammer, cable, putty, USB cable and S.I.T. recorder. The components of S.I.T. equipment are shown in Figure-1.

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During the last five (05) years, thousands of cast-in-situ concrete piles at different projects of Pakistan were tested by S.I.T. In this span, many problematic sonographs were also recorded.

This paper presents discussions on the principles, signal processing, and limitations of S.I.T. as well as case histories of problematic sonographs in the S.I.T. of piles tested all over Pakistan.

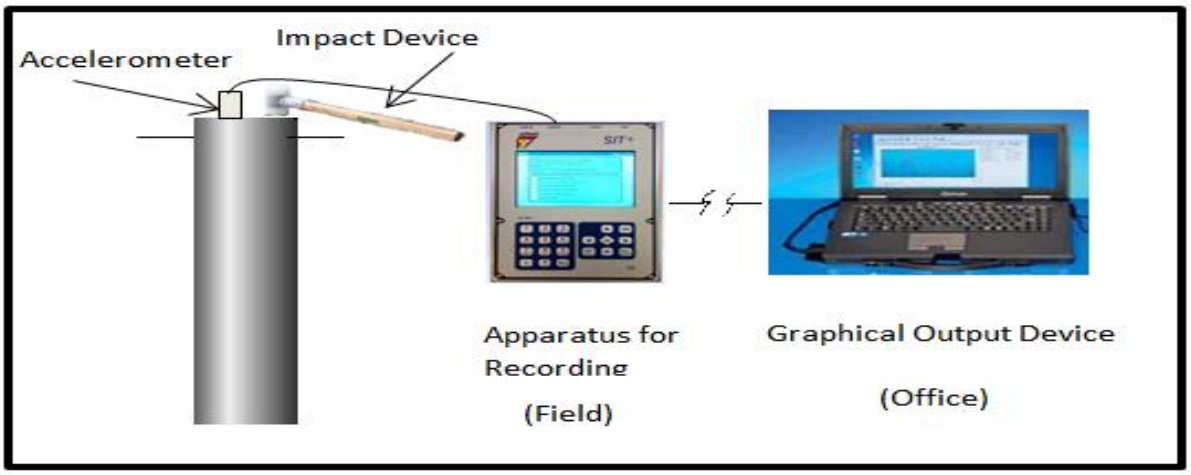
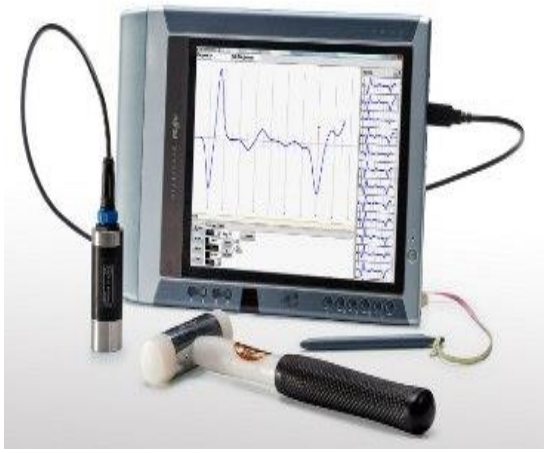



**Figure-1:** Components of S.I.T. Equipment

There are several types of S.I.T. equipments with the latest software that are being commonly used in Pakistan for the evaluation of pile integrity of these, two equipments viz. Profound (Netherlands) as well as PET Pro USB (U.K.) as shown in Figure-2, have been used for the present study.



Profound S.I.T. Equipment, Netherland



PET Pro USB S.I.T. Equipment, U.K.

**Figure-2:** S.I.T. Equipments

## PRINCIPLE OF SONIC INTEGRITY TESTING

The test method is based on the time domain, one dimensional wave theory as under:

$$\frac{\partial^2 u}{\partial x^2} - \frac{1}{c^2}\frac{\partial^2 u}{\partial t^2} = 0 \qquad (1)$$

where

u = displacement (m)

x = coordinate (m)

t = time (s)

c = wave propagation velocity

$$c = \sqrt{\frac{E}{\rho}} \tag{2}$$

Where,

E = Young's modulus of concrete (Pa)

ρ = density of concrete (kg/m$^3$)

The compressive wave is transmitted axially by the blow of hand-held hammer on to the pile head. The wave travels down along the pile shaft and reflects upward at the pile toe. In case where pile impedance variations occur due to change in pile cross sectional area or properties of pile materials, or presence of discontinuities, part or whole of downward travelling wave reflects at the impedance variation and returns to the pile top before the first reflection from the pile toe. The reflected wave is detected by an accelerometer placed on the pile head. Time velocity curve of signal is generally recorded. The reflected signal is also generally influenced by shaft friction contributed by the surrounding soil layers.

Following is the formula commonly used in analysis of integrity:

$$L = c.\frac{T}{2} \tag{3}$$

Where L is the pile length, c is the stress wave velocity and T is the travel time of the wave from pile top to toe and reflecting back to the top.

Impedance of pile z is given by:

$$z = \frac{EA}{c} \tag{4}$$

Where

A = Pile cross-sectional area (m$^2$)

The velocity of wave can be estimated from the strength of pile concrete or from the reference pile of the known length. Equation (4) shows that the change in pile impedance could be due to change in pile cross-sectional area.

## SIGNAL PROCESSING AND LIMITATION

In performing and interpreting the S.I.T., one must recognize the capabilities and limitations of this method. The quality of S.I.T. signal is also based on the operator's familiarity with the system and experience with design and construction of pile foundations, e.g., factors such as pile head preparation for the attachment of sensor and signal processing etc. which can affect the S.I.T. results.

Based on our experience, L/D ratio not exceeding 40 can produce the significant signals. S.I.T. does not yield any information on pile capacity. S.I.T. is capable of finding the presence of major defects such as voids or breaks and their size.

In order to obtain the optimum measurement signal of the pile, signal processing techniques were used. These can be applied while using the SIT in the field or later on, while analyzing measurement signals afterwards using SIT PC software.

**Filter** is used to eliminate the high frequency noise and to obtain a smooth signal. **Gain/Amplification** ratio used to obtain a legible reflectogram; it has to be compensated for damping, usually exponentially. In a well-balanced reflectogram, the impact from the top and reflection from the toe should have approximately the same amplitude. **Sharp** is used to emphasize significant features of the reflectogram and it down tones the smaller ones. **Fast Fourier Transform (FFT) Curve** is a frequency-domain analysis of the time-domain signal. PET converts frequencies to length (using the current wave speed). The FFT curve is usually smooth and can also help quantify the location of shallow anomalies. **Pile Averaging** means average of all the signals recorded for one pile. **Signal Stacking** technique is used to heap the entire signals for showing the average of these signals. Signal processing techniques of sharp, FFT, averaging and signal stacking are only valid for PET Pro USB S.I.T. equipment.

## CASE HISTORIES:

Ten case histories of problematic sonographs of piles on different projects in Pakistan are presented below.

### Case History 1: (Major joint/inclusion in pile at LOLMTP)

A large number of drilled piles were cast for Lahore Orange Line Metro Train Project (LOLMTP). The total length of this project was 27.0 km. Of this, the elevated length of 25.0 km was supported on pile foundation. At this project, shorter piles having diameter of 1.2 m and length ranging from 16.0 to 22.0 m were constructed with dry rotary auger bucket and longer piles having diameter of 1.2 m and length ranging from 23.0 to 52.0 m were constructed with reverse rotary method. This project is in under-construction now but piling work is almost complete.

Based on the geotechnical investigation, the subsurface conditions at this site consisted of a thin cover of firm to stiff lean / silty clay underlain by medium-dense to dense silty sand.

Sonic Integrity Testing (S.I.T.) was performed on all the piles of this project in which one of the problematic sonographs of pile is showing a major joint. This S.I.T. signal of pile having length of 16.5m cast by dry auger bucket method is shown in Figure-3.

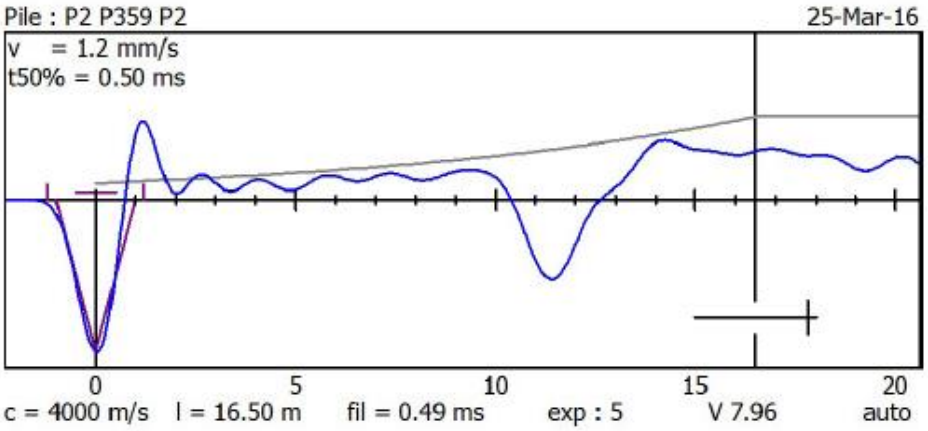



**Figure-3:** Major joint at 11.5 m

Interpretation of S.I.T signal with the aid of signal processing technique available in S.I.T. software was carried out. It was revealed that there was possibility of major joint/inclusion at 11.5 m below pile cut off level/reduction in impedance from 10.0 to 12.5 m below pile cutoff level. Toe reflex was also not evident at the required depth.

After interpretation of S.I.T. signal of this pile, coring in the pile was also recommended on this pile for better clarification. After coring, it was revealed that length of pile was short i.e., 11.5 m instead of 16.5 m. The results of S.I.T. signal and coring led to the same conclusion.



**Figure-4:** Coring of pile in progress

Therefore, by considering the results of S.I.T. and coring, strengthening measures in terms of supplementary piles were recommended.

## Case History 2: (Poorly Prepared Pile Head at LOLMTP)

The sonograph related to the case of poorly prepared pile head was collected from LOLMTP. This pile having diameter of 1.2 m and length of 16.0 m was also caste by dry auger bucket method. The sonograph of this case is shown in Figure-5.

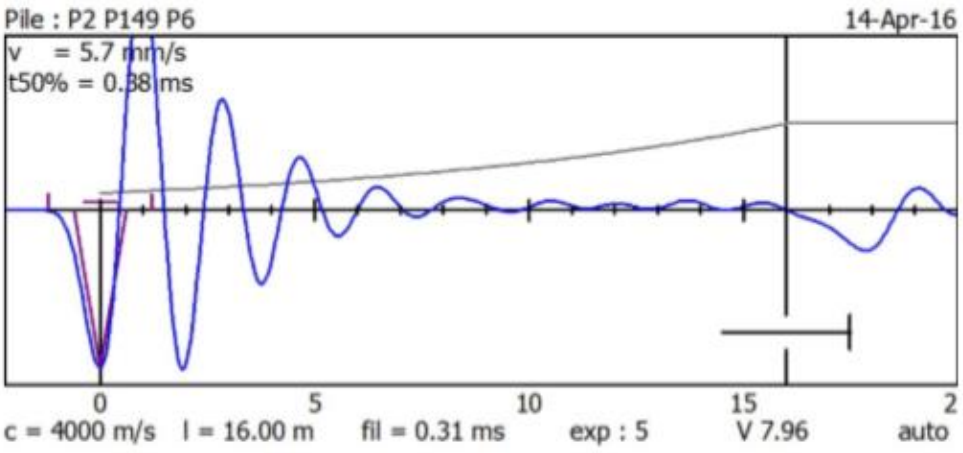



**Figure-5:** Poorly Prepared Pile Head

Based on S.I.T. interpretation with the aid of signal processing technique available in software, there was possibility of poorly prepared head/loose concrete on the pile head.

After interpretation of S.I.T. signal, retesting of this pile was recommended after preparing the pile head properly. Revised sonograph of the above pile is shown in Figure-6.

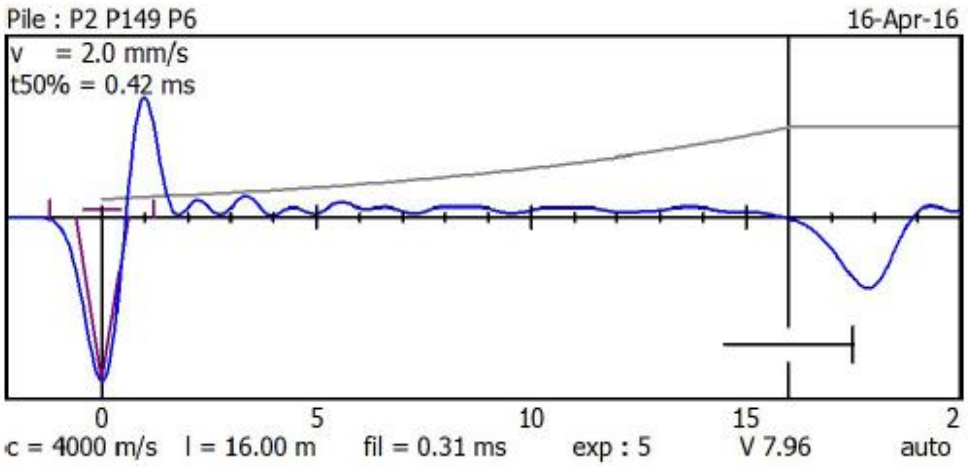



**Figure-6:** Sonograph after retest

Based on this revised sonograph, pile was considered satisfactory as there was no indication of any significant anomaly.

## Case History 3: (Shorter Length of pile at LOLMTP)

The sonograph related to the case of shorter length was also collected from the project described in Case History 1. The sonograph of this pile having diameter of 1.2 m and length of 16.5 m cast by dry auger bucket method is shown in Figue-7.

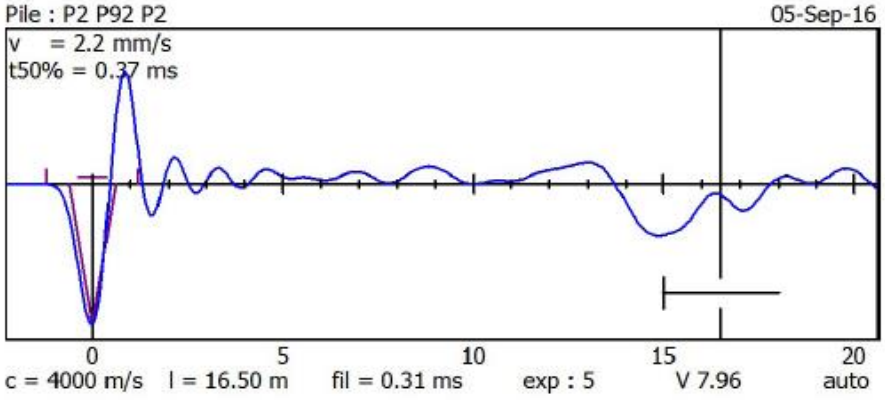



**Figure-7:** Shorter Length of pile

Based on S.I.T. interpretation with the aid of signal processing technique available in software, there was relative variation in cross sectional profile from 11.0 to 15.0 m, below pile cutoff level. Moreover, length of pile was 15.0 m below pile cutoff level which was shorter as compared to the actual pile length 16.5 m.

Based on this result, strengthening measure in terms of a supplementary pile was recommended.

## Case History 4: (Reduction in impedance in pile at LOLMTP)

The case of reduction in impedance was also collected from LOLMTP. The sonograph of this particular case is shown in Figure-8.

Interpretation of S.I.T. sonograph with the aid of signal processing technique available in software was carried out and it was revealed that there were high variations in signal and excessive reduction in impedance was visible from 5.0 to 9.0 m, below pile cutoff level.

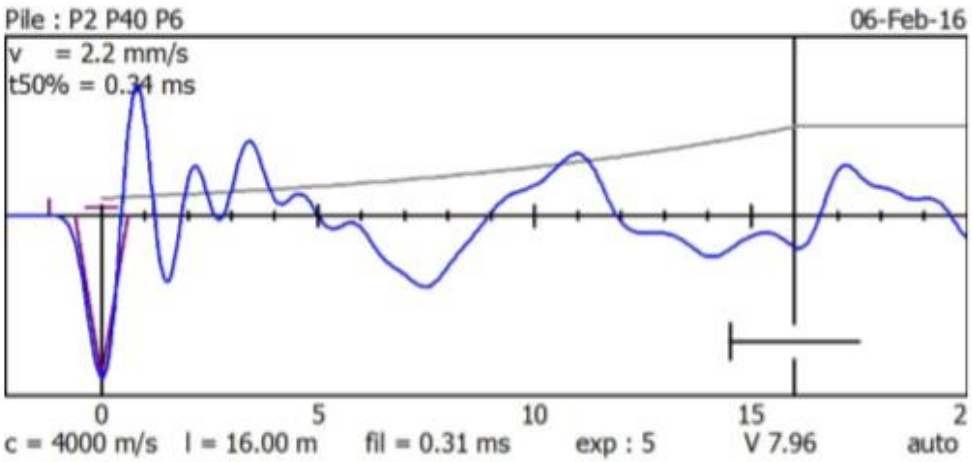



**Figure-8:** Reduction in impedance in pile

Based on this result, strengthening measure in terms of a supplementary pile was recommended.

## Case History 5: (Mud Joint expected in pile at Chaubucha Underpass)

Lahore Development Authority (LDA) planned to widen the Canal Bank road in Lahore. An Underpass was also proposed for this project. Piles were recommended for this underpass after soil investigations. This project is under-construction at the moment.

Boreholes with standard penetration tests (SPT) were carried out at the site for knowing the information about subsurface soils. Based on subsurface soil investigations, there was silty clay up to 6.0 m followed by medium-dense to dense silty sand.

S.I.T. was performed on one of the anomalous piles (UP1, AB1, P5) having possibility of mud joint mentioned in the pile construction record. This pile having length of 46.0 m and diameter of 1.80 m was cast by reverse rotary drilling method with bentonite. Interpretation of S.I.T. sonograph with the aid of signal processing technique available in S.I.T. software was carried out. It was revealed that major joint/inclusion was at 34.0 m below pile cutoff level / reduction in impedance was from 32.0 to 36.0 m & 37.0 to 40.0 m below pile cutoff level. The sonograph of this pile is shown in Figure-9.

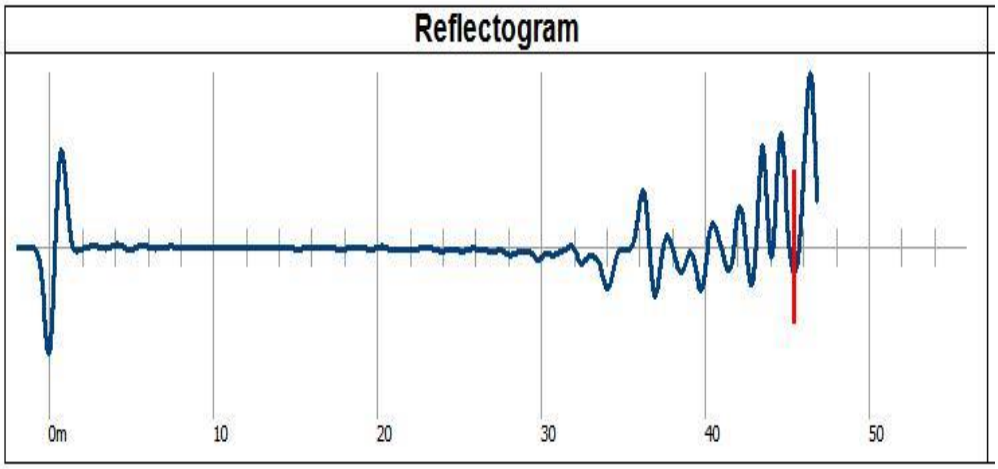



**Figure-9:** Mud Joint expected at 34.0 m

According to pile construction, tremie was choked at the depth of 34.0 m below pile cutoff level. So there was mud joint expected at this depth which was also confirmed by S.I.T. sonograph as shown in above figure.

Based on the results of S.I.T. and pile construction record, the pile was considered up to only to be 34.0 m long and the remaining load was balanced by providing the supplementary pile at the back of the existing pile.

### Case History 6: (Relative variation in X-sectional profile and reduction in impedance of pile at Khanewal - Lodhran Road Project)

Khanewal-Lodhran Road project was planned to improve/rehabilitate the existing Khanewal-Lodhran road along with the new bypass at Khanewal. The length of the proposed road was 94 km and the length of the proposed bypass was 12 km. The purpose of this project was to reduce the increased traffic congestion & travel time and to provide rapid access between Khanewal, Lodhran and adjoining areas. This project is under-construction now.

Based on geotechnical investigation, subsurface consisted of lean/silty clay underlain by medium-dense to dense silty sand.

Three pile load tests were proposed for the verification of the pile design. For this purpose, three test piles were cast by reverse rotary method with bentonite slurry in uncontrolled condition.

S.I.T. was performed on these test piles. An anomaly found in one of these test piles, having length of 25.0 m and diameter of 1.20 m. The anomalous signal of this pile is shown in Figure-10.

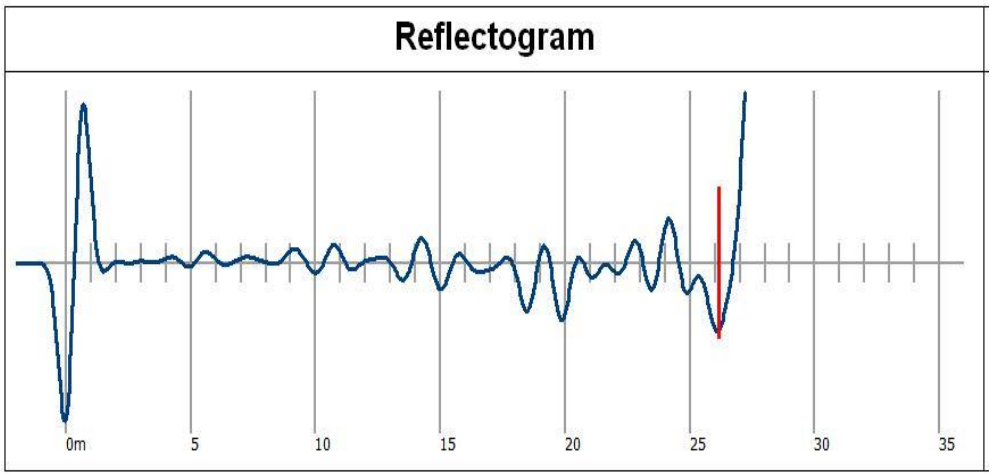



**Figure-10:** Reduction in impedance

Based on S.I.T. interpretation, slight reduction in impedance and relative variation in cross sectional profile were indicated at various depths.

After pile load test, it was revealed that the behavior of load settlement curve was inappropriate showing construction defects in the test pile which was confirmed by S.I.T. results also that indicating the intrusion of weak material in the pile concrete. Load-settlement curve of load test is shown in Figure-11.

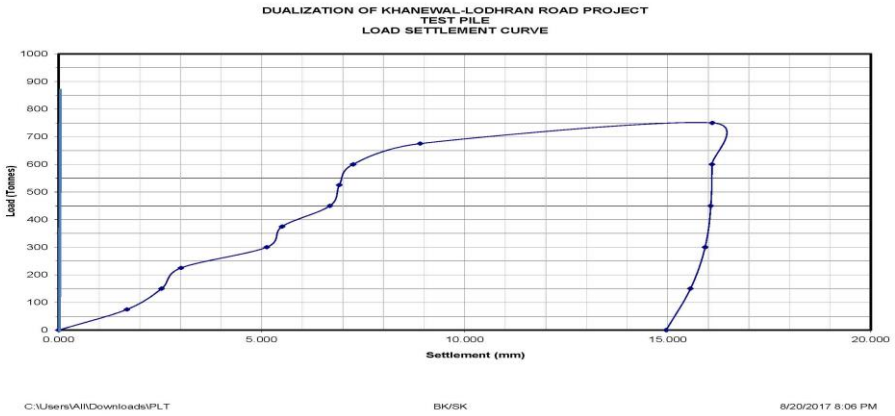



**Figure-11:** Load Settlement Curve

## Case History 7: (Major joint in pile at RLNG Power Plant, Balloki)

1223 MW combined cycle power plant was planned for construction in Balloki, District Kasur. This project was implemented by National Power Parks Management Company (Pvt.) Limited, Government of Punjab.

Based on the subsurface soil investigations, the soil deposits consisted of lean/silty clay and silt up to 6.0 m followed by medium-dense to dense silty sand/poorly graded sand.

Deep foundations were proposed at some structure locations and S.I.T. was recommended for checking the integrity of piles. One of the problematic sonogrpahs of a pile is shown in Figure-12.

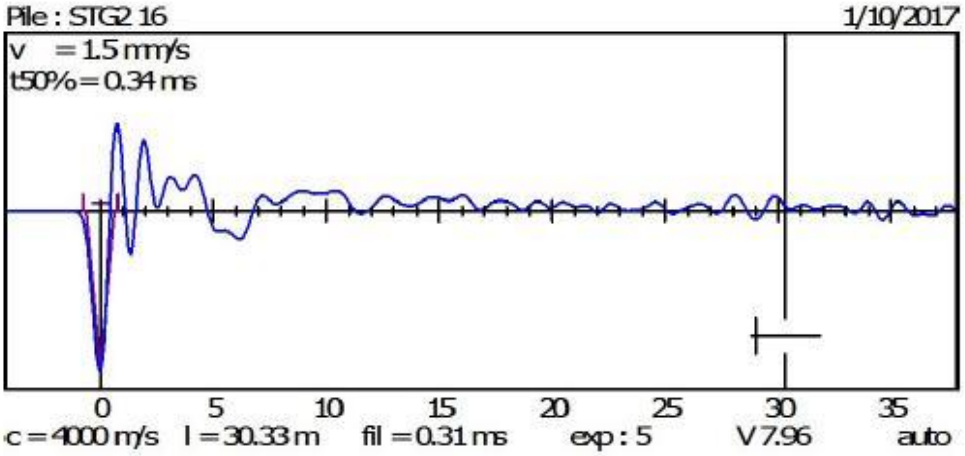



**Figure-12:** Major Joint at 6.0 m

Interpretation of S.I.T. reveals that there was a major joint at 6.0 m below pile cut off level.

Subsequently, coring was recommended on this pile for further clarification. Also strength tests were proposed on the core samples. Based on the strength results of cores, the UCS was only 2400 psi instead of design strength of 4000 psi, which indicated the presence of contaminated concrete. This pile was not considered satisfactory.

Therefore, strengthening measure in terms of compensating pile was proposed.

## Case History 8: (Short length and reduction in impedance in pile at RLNG Power Plant, Bhikki)

1180 MW combined cycle power plant was planned for construction in Bhikki, District Sheikhupura. The project was implemented by Quaid-e-Azam Thermal Power (Pvt.) Ltd., Government of Punjab.

Based on investigations, the soil deposits in this project area comprised of firm to stiff silty clay/silt up to 4.0 m followed by medium-dense to dense silty sand. The beds were largely hard, laminated and sandy with interbeds of clay and layers of sand. The alluvial deposits were very thick.

S.I.T. was performed on all the piles cast at different structure locations. The problematic sonograph of one of the piles having length of 35.0 m and diameter of 0.76 m is shown in Figure-13.

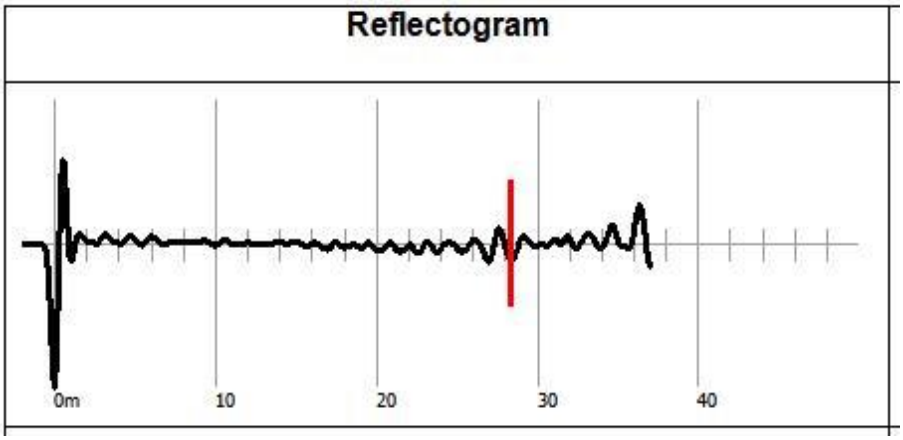



**Figure-13:** Short length and reduction in impedance

Based on S.I.T. interpretation along with the aid of signal processing technique, length of pile was about 28.4 m instead of 35.0 and reduction in impedance was indicated from 26.0 to 28.0 m below pile cutoff level.

Subsequently, this pile was recommended for high strain test for checking the pile capacity. Picture of high strain test in progress is shown in Figure-14.

As per result of high strain test, pile capacity was less as compared to the actual capacity. So a compensating pile was proposed.



**Figure-14:** High strain test at Bhikki

## Case History 9: (Major joint in pile at RLNG Power Plant, Jhang)

1100-1400 MW Punjab Power Plant has also planned for construction in Haveli Bahadur Shah, Jhang.

Based on geotechnical investigations, the soil deposits consisted of silt up to 3.0 m followed by silty sand/poorly graded sand with silt.

Six test piles were proposed by reverse rotary method with bentonite. Till now, all the test piles and pile load tests has been completed. S.I.T was also performed on all the test piles.

Based on pile construction record, an anomaly was found in one of the test piles having length of 35.0 m and diameter of 0.76 that was also confirmed by S.I.T. The problematic signal of this pile is shown in Figure-15.

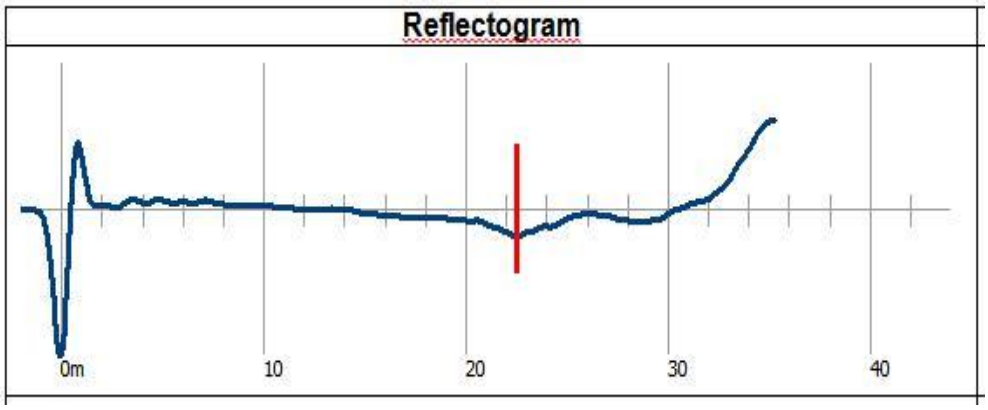



**Figure-15:** Major joint at 22.0 m

Based on S.I.T. interpretation, there was a major joint at 22.0 m below pile cut off level / reduction in impedance from 20.0 to 26.0 m below pile cut off level.

## Case History 10: (No Evident toe response of pile having L/D >40 at River Ravi Bridge Project)

A bridge was constructed over River Ravi for metro bus services. Piles were cast by reverse rotary method with bentonite, having length of 66.0 m and diameter of 1.50 m.

Based on the geotechnical investigation, the subsurface conditions at this site consisted of thin cover of firm to stiff lean / silty clay underlain by medium-dense to dense silty sand.

S.I.T. was performed on one of its piles and it was revealed that there was no toe reflex at the required depth. The S.I.T. signal of this pile is shown in Figure-16.

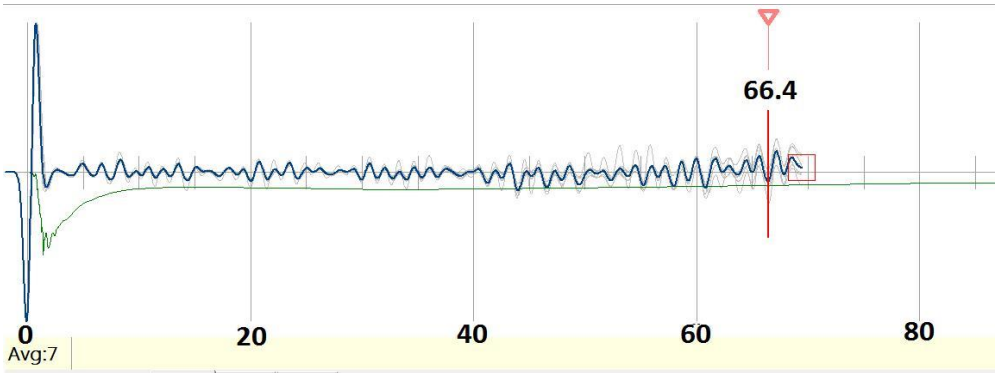



**Figure-16:** No evidence toe response

Interpretation of S.I.T. with the aid of signal processing technique available in software was carried out. Based on S.I.T. interpretation, there was no evident toe reflex as L/D ratio of this pile was 44, which was greater than 40 i.e., the limitation of S.I.T.

## CONCLUSIONS

- Sonic Integrity Testing (S.I.T.) is a quick and an effective method to assess the pile integrity. This method offers several advantages over other testing methods, including other non-destructive tests, for its rapid deployment, mobility, speed and cost. A large number of piles can be tested in a short time interval.
- The presented case histories demonstrate that deep foundation integrity testing is useful and has significant importance.
- Signal interpretations have been compared with the pile construction records. Minor defects, such as small cracks can be predicted in many instances and their nature should be further by coring, pile load test and high strain dynamic test. The predicted defects and possible causes verified from the pile construction record, coring, pile load test and high strain dynamic test would help the piling contractors to be aware of the causes of defects associated with the construction practices.
- In some cases, the results of S.I.T. were used to reject the piles whereas in some cases they were used to re-evaluate or redesign the piles. Frequently, S.I.T. is used to confirm anticipated defects in the piles.

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User Manual – Pile Echo Tester (PET)

**Paper No. 790**

# BARRAGE PROJECTS-FLOOD MANAGEMENT

**Amjad Saeed & Muhammad Sajid Nazir**

# BARRAGE PROJECTS-FLOOD MANAGEMENT

By

**Amjad Saeed[1], Muhammad Sajid Nazir[2]**

## 1. General

Agriculture is one of the most important sector contributing towards economic growth of Pakistan and mainly depends on irrigation. Punjab province is holding the largest area measuring about 8.4 million hectares being fed through the largest contiguous irrigation system. Barrages (13 No.) are the kingpin structures across Indus, Jhelum, Chenab, Ravi and Sutlej rivers in Punjab and are meant for diverting 317,895 cusecs water through inter river link canals (08 No.) and main canals (22 No.) (PMIU, 2017). Punjab barrages are designed mainly for guaranteeing (i) safe and sustained diversion of requisite flows into canals emanating from the barrage and (ii) safe passage of all range of flood flows through the barrages avoiding any probable damages to the barrage, its allied structures and areas located on the country side of the marginal bunds. Barrages constitute the most strategic component of the large contiguous irrigation systems. Their structural stability and effective operations are important for agricultural production and averting potential disaster during flood. These barrages have served their intended function in general during the design life by feeding the canals and passing the floods safely. Life span of barrages located in Punjab is from 45 years to 125 years and most of them have outlived their useful life. Varied river flows ranging from least design flow to the maximum highest flows (design floods or even more) at the barrages have caused various operational and structural problems. Flood discharging capacities have considerably reduced due to undesirable deposition immediately upstream of these barrages. Some of these barrages are found under designed for passing the 100-years return period discharges.

## 2. Floods

Floods can be defined as high discharges more than the dominant capacity of natural streams / river channels which spill over the dominant levels in the channels and cause flooding / inundation along the banks. The containment and control of flood waters is generally addressed through Flood containing and protection levees / bunds and hydraulic structures like Barrages.

It is a common misconception that all the floods are hazardous. Floods, if managed properly, contribute significantly towards the ground water recharge and soil fertility. However, poor management of floods causes devastation to life and property of the precinct. Riverine floods usually occur as a result of overflow of rivers and creeks following long duration rainfall over catchment areas. Floods are normally classified in following four categories, i.e. (i) Medium Flood; (ii) High Flood; (iii) Very High Flood; (iv) Exceptionally high Flood. The discharge values for each type of flood are peculiar to flood passing capacity of barrage structure and khadir plain of the river.

### 2.1 Generation of Floods

- Exceptionally high floods are generated by:
    - Exceptionally heavy rainfall or cloud burst in the river catchment.

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- Snow melt run-off from upper catchment is supplemented by widespread heavy rainfall in the lower catchment.
- Accidents like an avalanche or in some cases a heavy landslide blocks the river channel for some time like a dam and then giving way (Dam Break) like 1929 on Indus.

- Floods are mostly unpredictable in good time. Floods cannot be prevented and thus have to be managed to minimize losses.

## 2.2 Damages Caused by Exceptionally High Floods in Pakistan

| Flood Year | Monetary Losses (million $) | Lives Lost (No.) | Village Affected |
|---|---|---|---|
| 1973 | 5,134 | 474 | 9,719 |
| 1976 | 3,485 | 425 | 18,390 |
| 1992 | 3,010 | 1,008 | 13,208 |
| 2010 | 10,000 | 1,985 | 17,553 |
| 2014 | 440 | 367 | 3,000 |
| Total | 22,069 | 4,259 | 61,870 |

- The monetary break-up for 2010 losses is:
  - Housing $ 1,588 million
  - Agriculture $ 5,045 million
  - Transport and Communications $ 1,328 million
  - Miscellaneous $ 2,039 million
  - Total $ 10,000 million

## 3. Flood Management Strategies

Generally, the flood management programs follow one or a combination of the following strategies.

1. The resilience strategy
2. The Resistance strategy
3. The adaptive strategy

## 3.1 The Resilience Strategy

The main aim of resilience approach is to live with floods instead of fighting with floods. This approach was followed in Pakistan till about 5 decades ago when the river bank areas were undeveloped and the rivers were allowed to overflow the dominant channels and inundated the areas on both sides with small spill depths which were not only acceptable but in some cases even welcomed. With development of the river bank areas the concepts changed and structural measures mainly comprising flood bunds / levees were constructed on the river banks to stop the flooding of developed areas. Consequentially the active flood plains of the

rivers were raised with shallow flow depths. Construction of Barrages also supplemented this rise in increasing the intensity of flood effects in case of levee failures.

## 3.2 The Resistance Strategy

The resistance strategy aims at flood management / protection through mainly structural measures like levees, spurs and other water retaining and stream training works. The design of levees and other water retaining structures is usually based on an acceptable probability of overtopping and the portion of risk that remains is called residual risk (van Mannen and Brinkhu 2005). Residual flood risk behind levees is largely unaccounted. Levees are usually characterized by a uniform safety level, generally for a 50-year return period. This is unsatisfactory because the evolution of the flood events is unpredictable.

In Punjab, the main flood protection structures are the levees and spurs as enumerated in the following based on information received from Drainage & Flood Zone, Lahore: -

- Embankments 3,332 km
- Spurs 640 Nos

## 3.3 The Adaptive Strategy

The basic aim of Adaptive strategy is to provide holistic measures in consideration of the prevailing circumstances and facts. This can yield comprehensive and flexible design of flood structures capable of negotiating exceptional flood events even larger than the Probable Maximum Flood (PMP) design parameters and also have structural ductility.

The interdependence of structural and non-structural options in design is established in order to achieve an optimum configuration. The holistic design calls for integration of structural and non-structural measures in design in addition to operation. It is interesting to note that the holistic design is applicable for all flood management structures. We need to adopt Adaptive strategy in general.

## 4. Barrage Projects

Punjab government being cognizant of safety of these barrages launched a phased program in year 2004 for rehabilitation, up gradation and modernization of barrages to ensure their serviceability for another life span of fifty years.

While carrying out feasibility and detailed design studies, special emphasis was given to the aspect of flood management avoiding any possible damage to the barrage and allied infrastructures in its vicinity. Khanki, Trimmu and Panjnad Barrages located on Chenab, Balloki Barrage located on Ravi and Suleimanki and Islam Barrages on Sutlej river were found under designed in respect of maximum flood discharging capacity. Flood estimation was done adopting the statistical techniques for frequency analysis using the available data of the past peak flows and 100- years return period flows were taken as design flood for the rehabilitation and upgrading of barrages.

Different alternatives were studied during feasibility stage to rehabilitate and upgrade the existing barrages or go for construction of new barrage. Depending upon the existing condition and capacity of the barrages, decisions were made to rehabilitate and upgrade Taunsa, Jinnah, Balloki, Suleimanki, Islam, Trimmu and Panjnad barrages, whereas, replacement of Khanki headworks with a new barrage was recommended. Following table shows the original and revised capacities of the sick barrages where major interventions are made / being made in perspective of flood management.

| S. # | Barrage | Year of Construction/ Rehabilitation | Original Capacity/ Revised Capacity (Cusecs) |
|---|---|---|---|
| 1 | Khanki Headworks | 1892/2017 | 800,000/1,100,000 |
| 2 | Suleimanki Barrage | 1926/2017 | 325,000/440,000 |
| 3 | Balloki Headworks | 1913/2017 | 225,000/380,000 |
| 4 | Trimmu Headworks | 1939/2016 | 645,000/875,000 |
| 5 | Panjnad Headworks | 1932/2017 | 700,000/865,000 |

Brief description of the barrage projects with particular reference of flood management is given as follows:

## 4.1 New Khanki Barrage Construction Project (2013-2017)

Khanki headworks (1892) being the oldest one in Punjab was evaluated for the factors governing the reliability of the structure for the next 100 years in 2007 for making a decision either to go for remodeling or reconstruction. Earlier during years 1933-35, the headworks was extensively remodeled for 800,000 cusec discharge introducing left and central under sluices along with driving sheet piles. The existing structure was found unsafe for passing 100-year return period flood safely as widening will cause further formation of shoals on the upstream side due to increased looseness factor. Depth of the end sheet pile and floor thickness in the stilling basin were found much less in analysis of the existing structure for 100-year return period flood. Extensive dismantling & rebuilding operation were envisaged in remodeling the existing structure for 100-year return period discharge. In case of remodeling the existing, it will become a mixture of several design and construction techniques. Based on the evaluation study and recommendations of experts, option of replacement was preferred over remodeling/upgrading of the existing headworks.

A project captioned, “New Khanki Barrage Construction Project” costing Rs. 23,442 million was approved by the ECNEC in 2009 and is nearing completion with the financial assistance of Asian Development Bank. New Barrage is built 900 ft downstream of the old headworks with enhanced flood discharging capacity 1,100,000 cusec.

Length of the new barrage is same as that of old headworks whereas crest of the barrage (in main weir and under sluices portions) is fixed at lower elevation approximately equal to average river bed level thereby creating more water area to accommodate the enhanced discharge. Barrage is divided into three compartments main weir having 55 bays @ 60 ft wide each, left and right pockets each having 5 bays @ 60 feet wide.

Old headworks is demolished completely with the consideration that any component of old headworks having elevation higher than elevation upstream floor of new barrage, if left intact, will have adversely affect the hydraulic performance of the new barrage.

Moreover, new guide banks normal to barrage axis are provided as per recommendation of Model Study safeguarding passage of flood flows with high velocities through the well-defined hydraulic section. Height of guide banks and marginal bunds are increased in consideration of

HFL verified on the model and free board as notified by Federal Flood Commission to avoid any chance of overtopping during floods.

Barrage is equipped with radial gates and hydraulic hoisting system coupled with local control panels and central control. Power supply is ensured with dedicated 11 KV feeder and two standby DG sets one at each end of the barrage to avoid any failure in operation of barrage gates especially during floods. Intelligent Control and Telecommunication System is installed at the barrage for managing the efficient operation of the barrage during normal and flood operations as well.

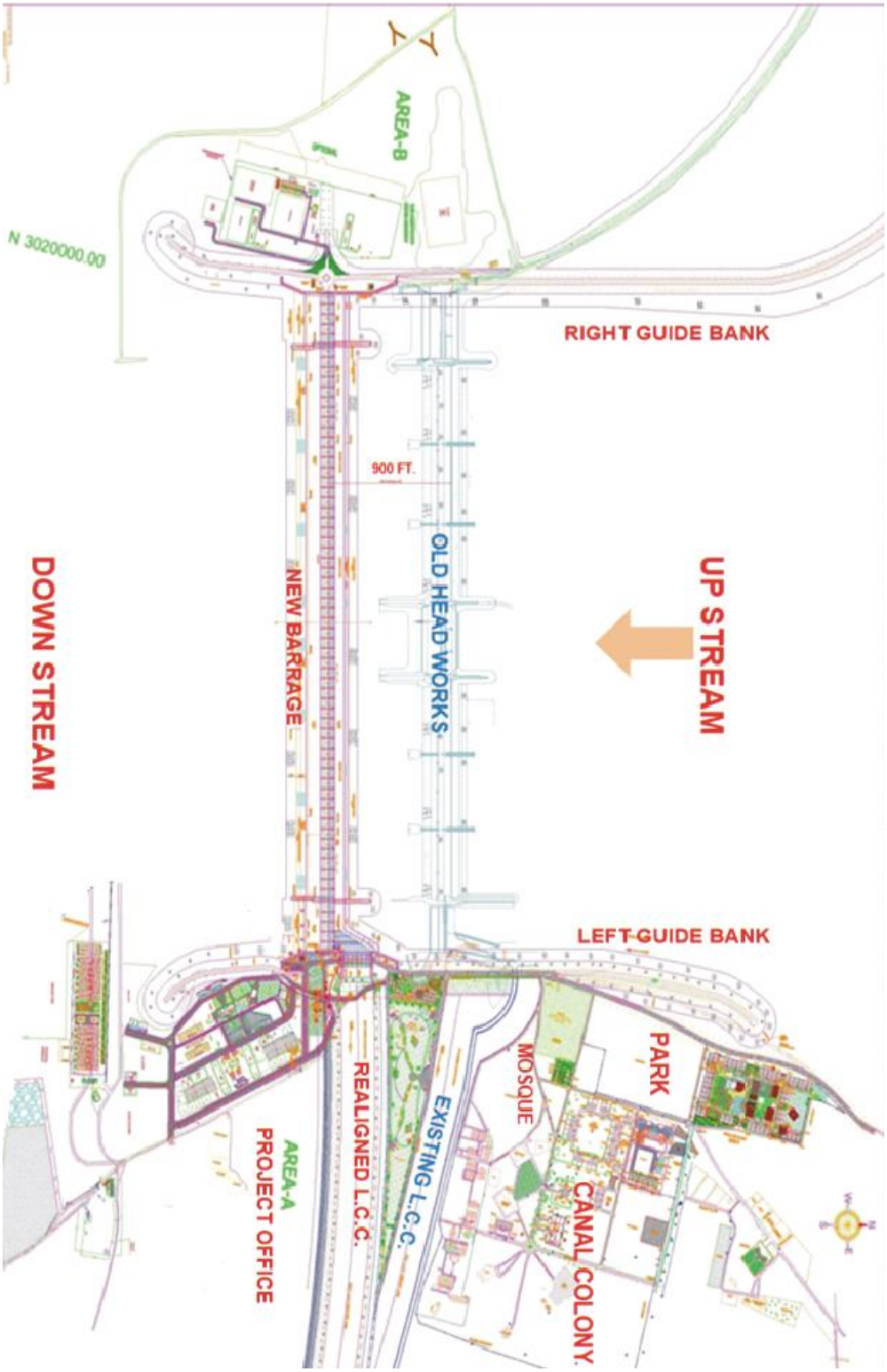



***Figure 1:*** Key Plan-Old Khanki Weir and New Khanki Barrage with Re-Aligned L.C.C

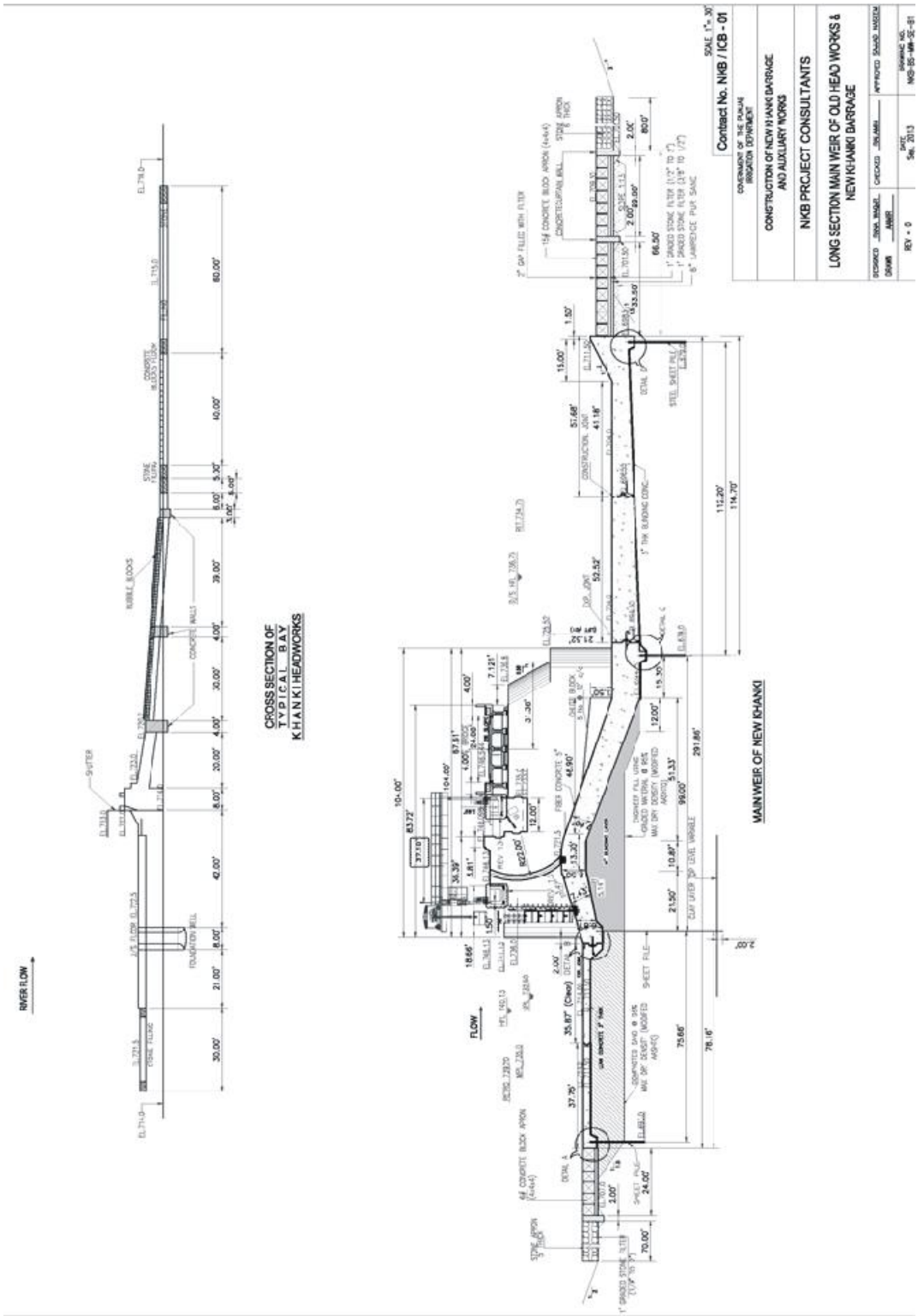



***Figure 2:*** Typical L-sections of Old Weir and New Weir of Khanki Barrage

## 4.2 Rehabilitation and Upgrading of Suleimanki Barrage (2014-17)

Suleimanki Barrage constructed in 1926 on Sutlej river under Sutlej Valley Project (SVP) experienced floods four times higher than its designed flood discharging capacity of 325,000 cusecs. Major devastations occurred in the adjoining areas due to occurrence of breaches in the marginal bunds of the barrage. Barrage functioning was seriously affected due to construction of storage structures in India and implementation of Indus water Treaty. Drastic reduction in discharges reaching at barrage for a larger part of the year, distorted pattern of flow and oblique entry within the guide bank zone, inadequate and deferred maintenances, have rendered the river bed to a highly silted configuration causing severe masking in two-third width of the river close to the barrage. With existence of vegetated islands, Model study depicted maximum flow as 250,000 cusecs through the barrage observed at critical gauge of 575.00.

Gigantic formation of islands with in the guide banks has reduced flood discharging capacity of the barrage by 25% with respect to existing design capacity and about 40% with respect to 100-year design discharge. Like other barrages, Safety evaluation, feasibility and detailed design studies were carried out to evolve a scheme for safe passage of floods and ensuring reliable and sustained supplies to the dependent canal commands through three canals off taking from the barrage. Based on the hydrological studies and deliberations on design flood, Design flood for Suleimanki Barrage is recommended as 430,000 cusecs against existing design of 325,000 cusecs.

Rehabilitation and upgrading of Suleimanki Barrage is taken up as tranche-3 of PIAIP (ADB-Financed MFF) under phase-1 of barrage projects aiming i) restoration of existing flood discharging capacity 325000 cusecs of the barrage and ii) ensuring sustainable canal supplies to the dependent command areas. Bela in substantial quantity has been removed and ample waterway is made available upstream of the barrage for taking high flows without causing any increase in flood heights on the upstream side.

Under sluices gates (16 No.) and head regulators gates of three off-taking canals have been replaced in addition of rehabilitation of main weir gates (24 No.) of the barrage. New hoisting system with motorized operation is introduced at the barrage for efficient operation of the gates during normal flow and flood season as well. Upstream floor with sheet pile cut off at its upper end and concrete overlay on the barrage floors in main weir portion of the barrage have been provided for strengthening of floors and avoiding any failure of barrage against parallel flow. Over PRs. 3.0 Billion are spent on these works.

Implementation of rehabilitation and upgrading works will avert the possibility of damages due to abnormal increase in flood heights and potential threat of breaches in marginal bunds of the barrage.

Furthermore, in order to manage 100-year flood at Suleimanki, a project captioned, “Flood Management at Suleimanki Barrage” is also prepared under PIAIP. Options were studied to accommodate the discharge of about 115,000 cusecs in excess of existing design capacity 3,25,000 cusecs giving due consideration to socio-economic and strategic aspects. Construction of a gated spill weir on right side of the barrage was recommended as most viable option in comparison of other options like i) raising HFL ii) lowering crest of the barrage and iii) live with operation of breaching section. Gated spill weir, level crossing cum AR Bridge for Pakpattan Canal and spill channel for 115,000 cusecs discharge will cost about PRs. 6,000 Million for which financing window is being managed by the Punjab Government.




***Figure 3:*** Bela Removal- Before & After-Suleimanki Barrage

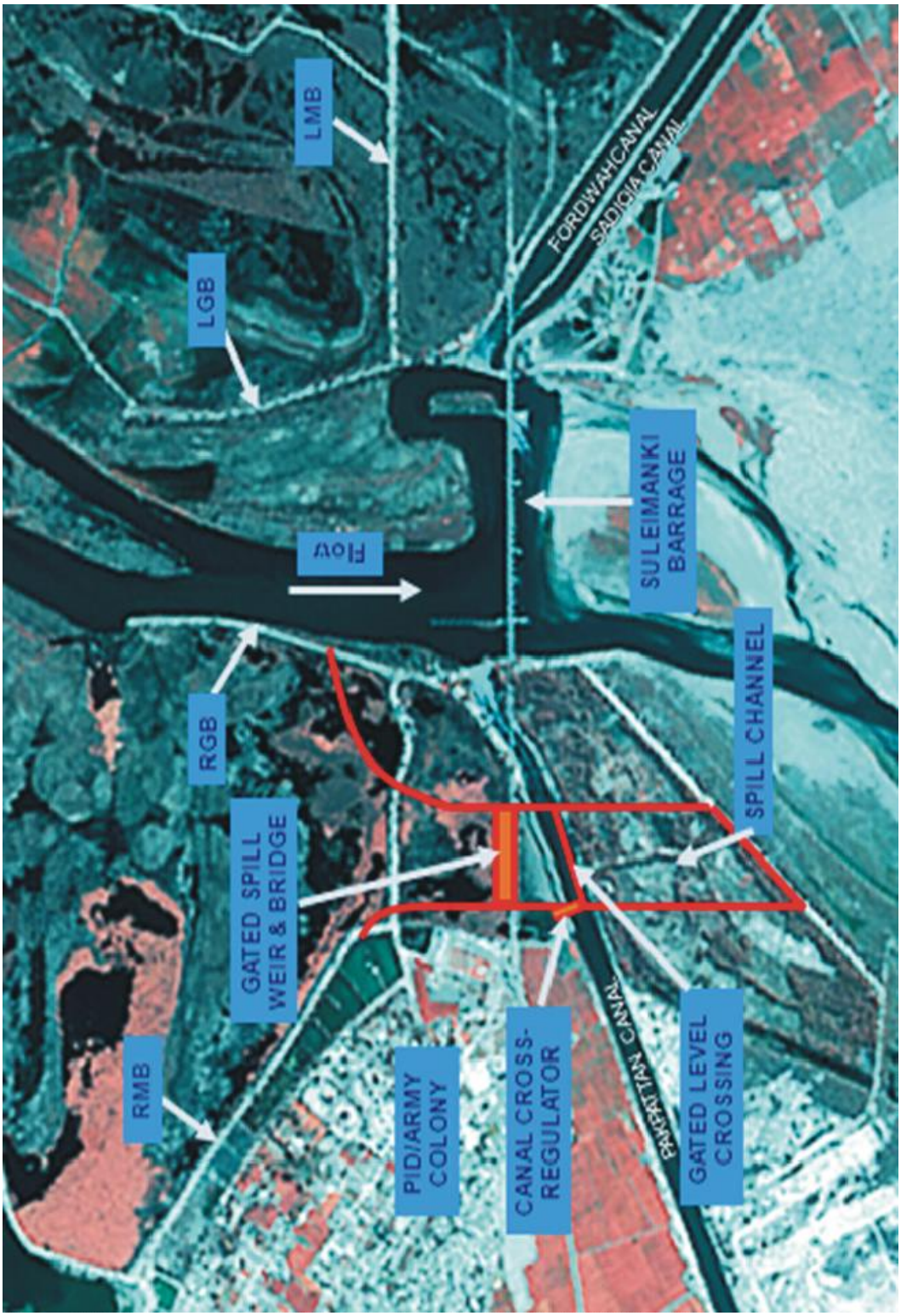



***Figure 4:*** Flood Management at Suleimanki Barrage-Schematic Plan

## 4.3 Rehabilitation and Upgrading of Balloki Barrage (2011-2017)

Balloki Barrage constructed during 1911-13 has two off-taking Canals on its left flank, i.e., 1) Lower Bari Doab Canal (LBDC) and 2) Balloki – Suleimanki Link Canal (B.S. Link). Originally, Balloki Barrage was constructed for feeding LBDC. Thereafter, a head regulator for the B.S. Link Canal was added alongside LBDC head regulator in 1954.The pitched island and shank were constructed on the upstream opposite to bays No. 21 and 22 in 1956-57 and the barrage was extensively remodeled in 1964-65 to enhance its design discharge from 1,39,000 cusecs to 2,25,000 cusecs. In 1988, exceptionally high flood of about 380,000 cusecs was recorded at Balloki inclusive of 140,000 cusecs flood water passed through the breaches occurring upstream of the barrage. The pitched island breached during exceptionally high flood in 1988 in addition of the other damages to the barrage appurtenant.

The Balloki Barrage, head regulators, gates and hoisting system were more than 100-year-old and were suffering from age and deferred maintenance. The flood capacity of the barrage was inadequate and pitched island was no longer functional. In order to Rehabilitate and Upgrade Balloki Barrage and its allied structures, a project titled. “Rehabilitation and Upgrading of Balloki Barrage and Canal Head Regulators” has been implemented with the financial assistance of ADB during the year 2011 to 2017 as part of Tranche-1 of PIAIP. The project was aimed to ensure safe passage of 100- year return period flood (380,000 cusecs) through the barrage besides diverting reliable and sustained supplies in off-taking canals. A gated flood spillway comprising 16 bays (Each 40 feet wide) is constructed on the right side adjacent to the barrage to accommodate discharge of 120,000 cusecs. Flood discharging capacity of existing barrage (35 bays @ 40 feet) has been remodeled to 260,000 cusecs mainly with the conversion of pitched island into concrete divide wall.

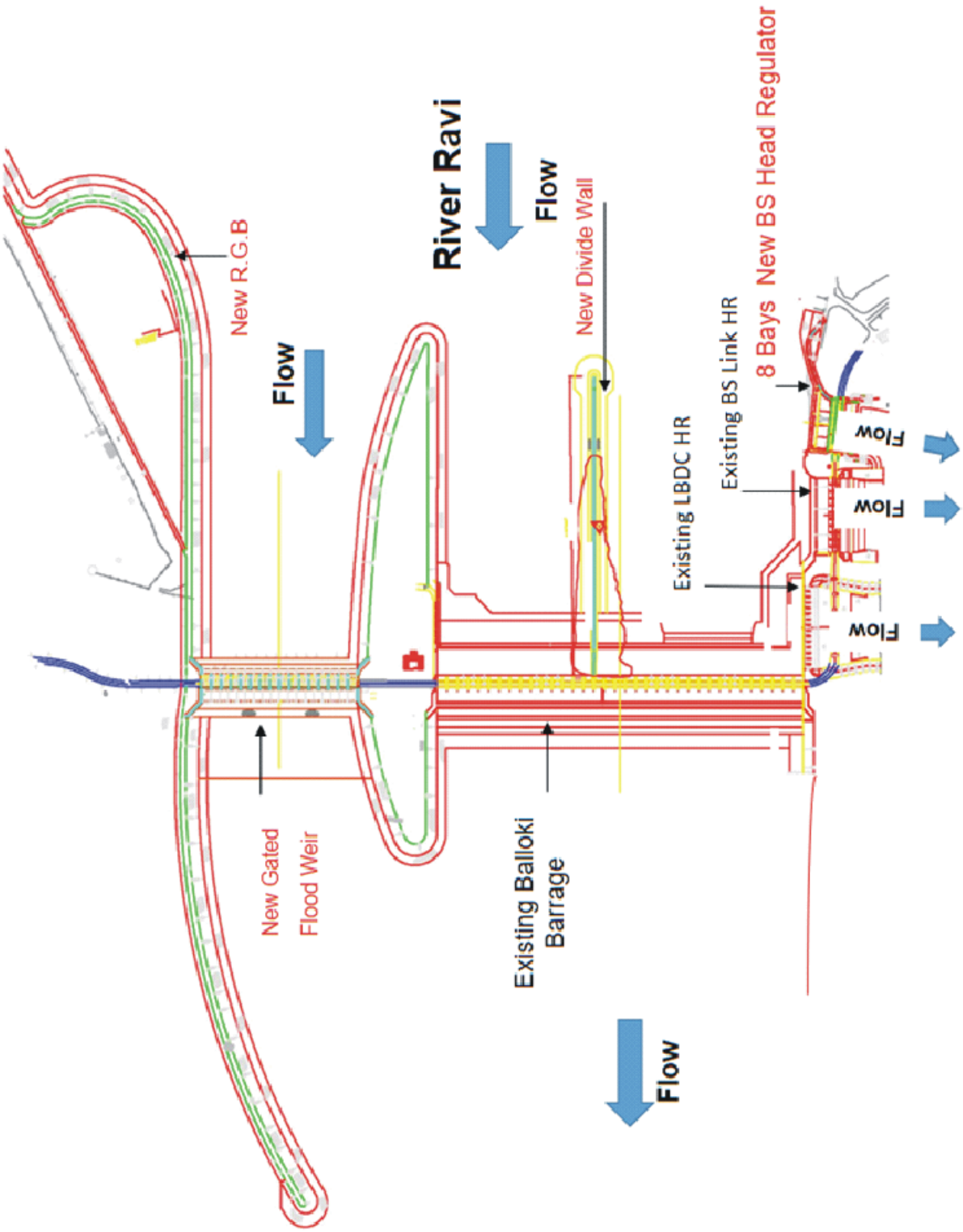



***Figure 5:*** Balloki Headworks- Key Plan

## 4.4 Trimmu-Panjnad Barrages Improvement Project (2016-2020)

### 4.4.1 Rehabilitation and Upgradation of Trimmu Barrage

Trimmu barrage was constructed in 1939 under Haveli Canal Project and remodeled in 1962 under Indus Basin Project works with the addition of Trimmu-Sidhnai Link Canal. The barrage is located 5 km downstream of confluence of Jhelum and Chenab rivers. The barrage is meant for diverting about 22585 cusecs water for nine south districts (Jhang, Toba Tek Singh, Muzaffargarh, Khanewal, Multan, Vehari, Lodhran and Bahawalpur) of Punjab. Total command area depending on Trimmu Barrage is about 2.70 million acres and dependent farm families are 400, 000.

The barrage was designed to pass a maximum flood discharge of 6,45,000 cusecs whereas it has experienced five floods more than its design capacity. After construction of Mangla Dam, imbalance flows in Jhelum and Chenab rivers have caused oblique river approach at the barrage. Consequently, sediment deposition on the inner side of the approaching flows along left guide bank has occurred over a period of 50 years. The safe discharging capacity of the barrage is now reduced to 4,50,000 cusecs due to sediment deposition as an island upstream of the barrage. The hydrologic studies carried out during feasibility, detailed design (2012) and design review studies (2016) reveal that 100-year return period discharge at Trimmu Barrage comes out as 8,75,000 cusecs. Different scenarios of releases from Mangla reservoir and floods in Chenab river were studied to arrive at 100-year return period flood at Trimmu Barrage. Finally, based on 100-year flood in Chenab River at Marala and 1992-flood at Mangla with apprehended delay of 3- hour in reduction of Mangla outflow peak, design discharge of Trimmu barrage is adopted as 8,75,000 cusecs for implementation of rehabilitation and upgrading works at the barrage. Following the recommendations of feasibility and detailed design studies and advisory group, a project captioned, “Trimmu-Panjand Barrages Improvement Project” is in the course of implementation with the financial assistance of ADB with a loan of US$ 150 Million. Extension of barrage on its right side by adding thirteen bays and bela removal are the main components of the project in perspective of flood management which are being executed to accommodate the enhanced discharge of 2,30,000 cusecs. Moreover, barrage and canal head regulators gates and hoisting system are being replaced with local and central control operating systems to ensure efficient regulation during normal flow season and flood season as well.

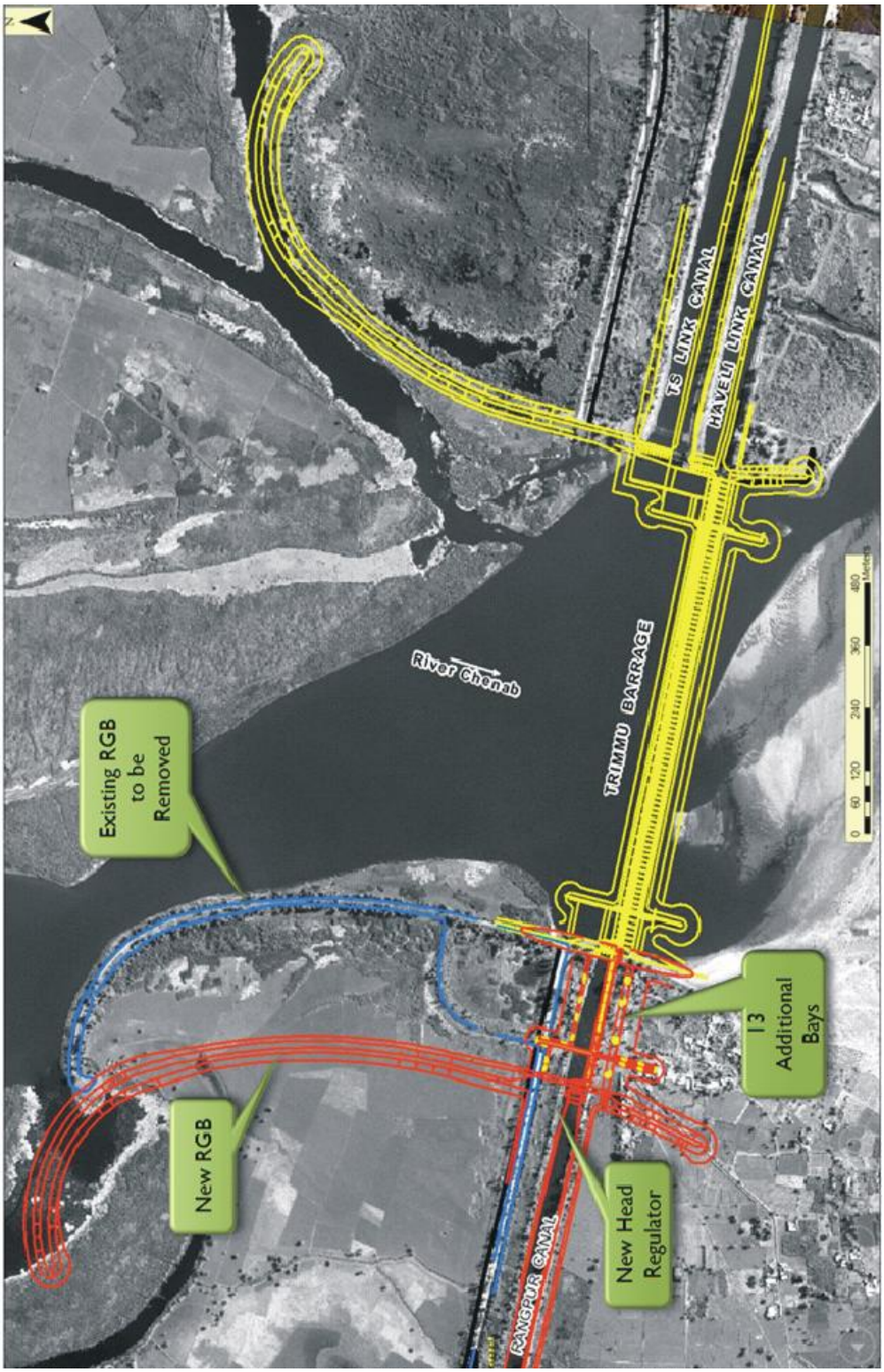



***Figure 6:*** Rehabilitation and Upgradation of Trimmu Barrage- Key Plan

### 4.4.2 Rehabilitation and Upgradation of Panjnad Barrage

The Panjnad barrage is the last barrage constructed on River Chenab, downstream of the confluence point of River Sutlej. The barrage was constructed during the period 1925-1929 with design discharge of 450,000 cusec. In September 1929, a flood of 550,000 cusec was received and its design capacity was raised to 700,000 cusec by adding 14 bays (bays no. 34 to 47) and junction groyne. The barrage has experienced three floods more than its design capacity. The off-taking canals from barrage irrigate 2.68 million acres area of Bahawalpur and Rahimyar Khan Districts by diverting 14,433 cusec through Panjnad Main Line, Abbassia Link and Abbasia Canals.

The oblique river approach and sediment deposition over the years has caused the masking of Panjnad Barrage. Bela formation is mainly affecting the upstream of barrage and area along and between the divide wall and junction groyne, thus, seriously reducing the capacity of barrage. Due to continuous wear and tear and deterioration with the passage of time, there is leakage through gates, chocked pressure pipes and cracked floor further aggravate the condition of barrage. Frequency analysis and hydrodynamic flood routing scenarios for 100-year return period of flood estimates the peak discharges as 849,176 cusecs and 905,858 cusecs respectively, however, the design flood of 865,000 cusecs is adopted as 100-year design flood, which is little higher than the average of these two analyses.

A project titled, “Trimmu-Panjand Barrages Improvement Project” is under implementation with the financial assistance of ADB with a loan of US$ 150 Million. Addition of three depressed bays with all appurtenant works and scour protection arrangements alongwith the construction of two divide walls separating the depressed bays from existing barrage and removal of bela formation are envisaged as a capacity increasing measure. Strengthening of existing barrage structure and modernization of electro-mechanical components are the other major interventions that will be carried out under the scope of project.

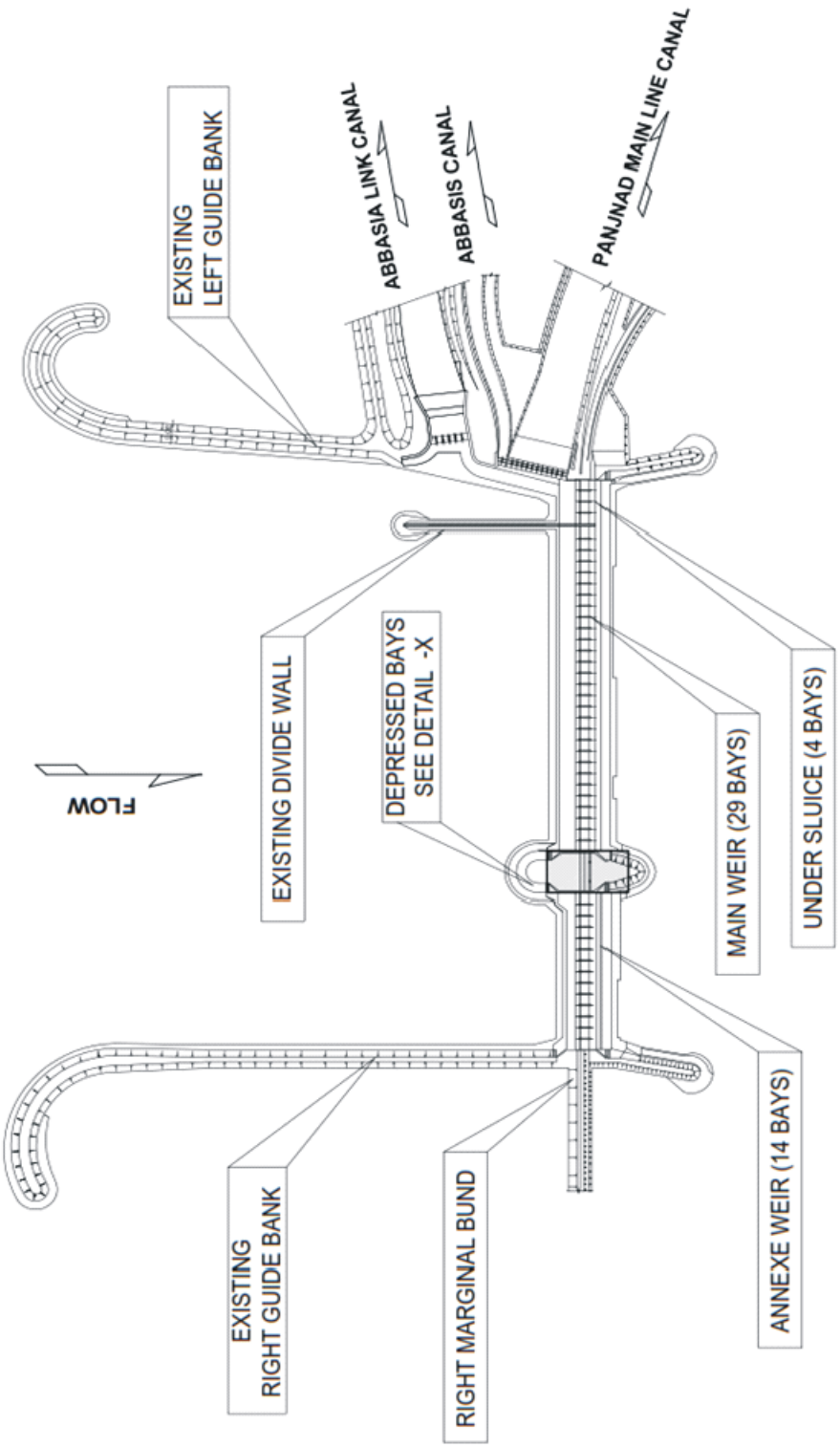



***Figure 7:*** Rehabilitation and Upgradation of Panjnad Barrage-Key Plan

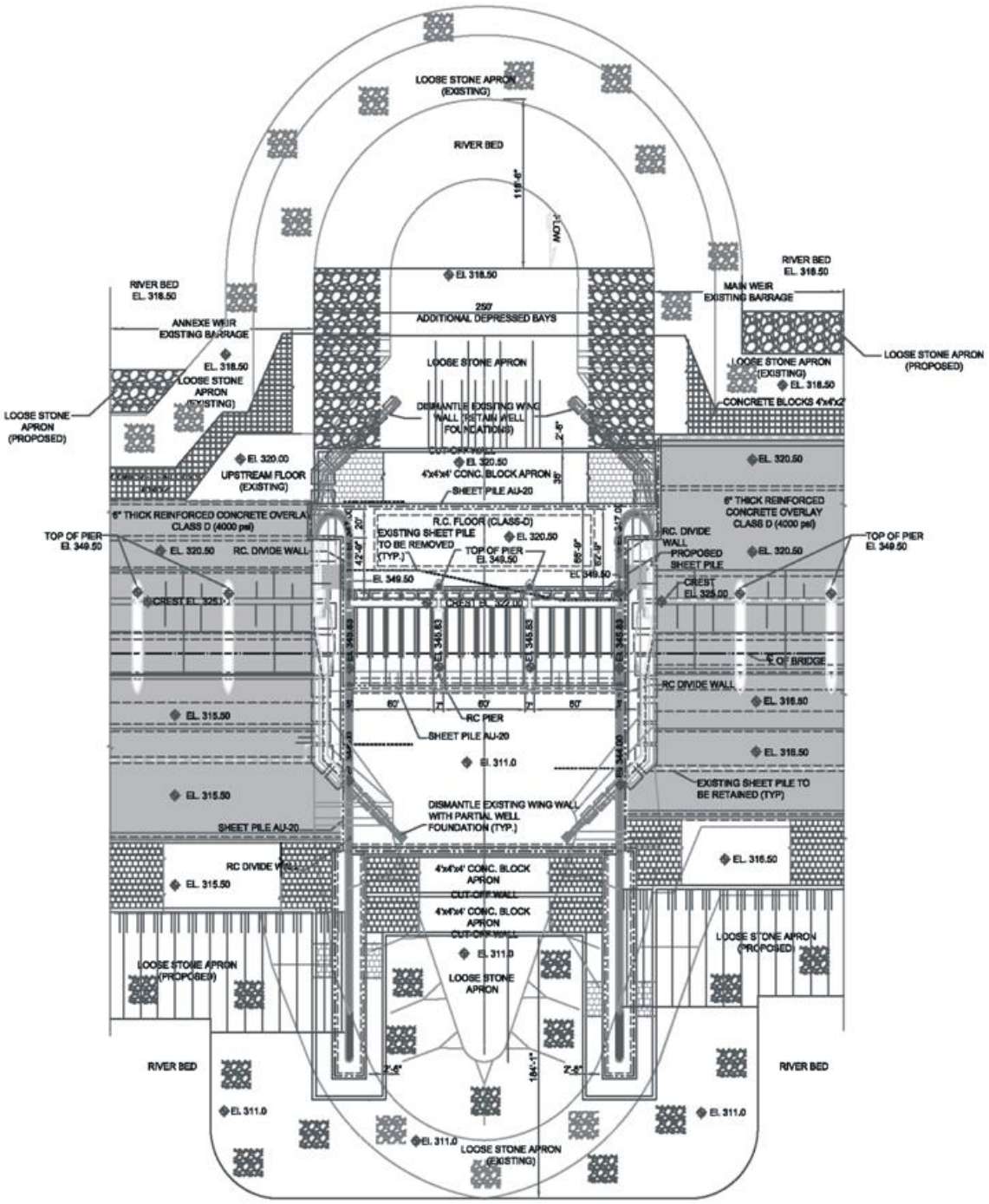



***Figure 8:*** Plan of Additional Depressed Bays (Detail-X)-Panjnad Barrage

## 5. Conclusion and Recommendations

From the foregoing narration it can be safely concluded that the referred Barrages are now fit for designed performance during floods provided that regulation rules and O &M manuals are carefully followed and observed. The barrages being very sensitive and strategic structures need extreme care. Intervention of flood bypass channels/gated spillway to accommodate 100-year return period floods on barrages need to be critically reviewed.

Based on the studies of the Barrages and implementation of the projects, the following recommendations are made:

i) The evaluation of safety of all barrages should be made periodically.

ii) Pond area of barrages, where perennial flows are not available should be kept free of Shale (bela) formation either by regulation or dredging to ensure availability of adequate water way to safely pass design floods.

iii) The surveillance and maintenance of barrages should be given top priority by the officials incharge. Adequate funds should be provided for each barrage for timely maintenance.

iv) The barrage operation should be given due care and dedication by the offices/officials incharge.

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