MANUAL
ON
ARTIFICIAL RECHARGE OF GROUND WATER

GOVERNMENT OF INDIA
MINISTRY OF WATER RESOURCES
CENTRAL GROUND WATER BOARD

सत्यमेव जयते

SEPTEMBER 2007

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FOREWORD

Ground water has become the major source of water to meet the requirements of domestic, industrial and irrigation sectors in India in the last few decades on account of its ubiquitous occurrence, easy availability and reliability. These qualities have led to its indiscriminate exploitation in some parts of the country without due regard to recharging options. This has resulted in considerable depletion of the ground water table in some areas causing concerns for the long-term sustainability. There is an urgent need for augmentation of the limited ground water resources by taking appropriate measures including suitable management interventions.

Artificial recharge to ground water through scientifically designed structures has been proven as a viable option for augmentation of ground water resources. It also provides an opportunity to utilize the surplus monsoon runoff which otherwise is lost to sea unutilized.

The pioneering efforts of Central Ground Water Board (CGWB) have been instrumental in popularizing cost-effective recharge augmentation techniques suitable for various hydrogeologic regions in the country. CGWB has already implemented a large number of pilot schemes for artificial recharge of ground water across the length and breadth of the country.

The Manual on "Artificial Recharge of Ground Water" is the latest in a series of publications on recharge augmentation brought out by the Board. The manual deals with various aspects of artificial recharge of ground water including planning of artificial recharge schemes, artificial recharge techniques and design of structures, monitoring of augmented water levels and water quality, economic evaluation of recharge projects and issues related to operation and maintenance of artificial recharge structures. Roof top rainwater harvesting techniques which are particularly suitable for urban areas have also been included and described in detail.

We hope that this manual will be of immense help to all those who are concerned with the onerous task of sustainable development and management of ground water resources including the administrators, planners, scientists, local bodies, non governmental organizations and the people at large.

I would like to place on record my appreciation of the efforts made by Shri B.M. Jha, Chairman, Central Ground Water Board and his team of experts in bringing out this manual. I sincerely hope that the manual will be made use of by various agencies and individuals involved in activities related to ground water augmentation through artificial recharge.

(Gauri Chatterji)

PREFACE

The dependence on ground water as a reliable source for meeting the requirements for irrigation, drinking and industrial uses in India has been rising rapidly during the last few decades. Ground water development has occupied an important place in Indian economy because of its role in stabilizing agriculture and as a means for drought management. Over the years, particularly since the launching of Five Year Plans, there have been continued efforts in India for development of ground water resources to meet the increasing demands of water supply for various sectors. In many parts of the country, ground water development has already reached a critical stage, resulting in acute scarcity of the resource. Over- development of the ground water resources results in declining ground water levels, shortage in water supply, intrusion of saline water in coastal areas and increased pumping lifts necessitating deepening of ground water abstraction structures. These have serious implications on the environment and the socio-economic conditions of the populace. Worsening ground water quality has also adversely affected the availability of fresh ground water in several areas. The prevailing scenario of ground water development and management in India calls for urgent steps for augmentation of ground water resources to ensure their long-term sustainability. The diverse nature of the terrain and complexities of hydrogeological settings prevailing in the country makes this a challenging task.

Central Ground Water Board has been in the forefront of activities for augmenting ground water resources through scientifically designed artificial recharge structures for harvesting non-committed surplus runoff which otherwise runs off into sea. A number of pilot schemes and demonstrative artificial recharge schemes have been implemented by the Board in association with various State Government organizations since the 8^{th} plan period. These are aimed at popularizing cost-effective ground water augmentation techniques suitable for various hydrogeological settings, to be replicated by other agencies elsewhere in similar areas. Based on the valuable experience gained from such activities, the Board has also brought out a number of publications on various aspects of artificial recharge. The ‘Manual on Artificial Recharge of Ground Water’ is the latest in this series and has updated information on various aspects of investigation techniques for selection of sites, planning and design of artificial recharge structures, their economic evaluation, monitoring and technical auditing of schemes and issues related to operation and maintenance of these structures. Roof top rainwater harvesting, suitable especially for urban habitations is also dealt with in detail. This publication will be of immense use to all those who are engaged in planning and implementation of ground water augmentation schemes in various parts of the country.

The work done by Central Ground Water Board and other Central, State and non-governmental agencies involved in the water sector have provided the basic inputs necessary for the preparation of this manual. I would like to specially acknowledge the efforts of Shri.C.S.Ramasesha, Commissioner (GW) and Member (SML) (Retd), Shri.Nandakumaran.P, Dr.S.K.Jain, Shri.K.R.Sooryanarayana and Shri.Y.B.Kaushik, Senior Hydrogeologists, in bringing out this publication.

I hope this manual will be useful to all agencies engaged in planning and implementation of artificial recharge schemes across the country in a scientific manner to ensure optimum benefits. Comments and suggestions on various aspects of artificial recharge dealt within this document will be highly appreciated and will be useful for updating the manual in the years to come.

Faridabad
September 2007

(B.M.Jha)
Chairman
Central Ground Water Board

MANUAL ON ARTIFICIAL RECHARGE OF GROUND WATER

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1. INTRODUCTION

1.1 Background

Ground water, which is the source for more than 85 percent of India's rural domestic water requirements, 50 percent of its urban water requirements and more than 50 percent of its irrigation requirements is depleting fast in many areas due to its large-scale withdrawal for various sectors. For example, out of a total of 5723 assessment units (Blocks/Mandals/Talukas) in the country, 839 have been categorised as 'Over-exploited' as assessed on 31st March 2004, with ground water extraction in excess of the net annual recharge. There are also 226 'Critical' assessment units where the ground water draft is between 90 and 100 percent of the annual replenishment, apart from 30 blocks having only saline ground water (CGWB, 2006).

There have been continued efforts in India for development of ground water resources to meet the increasing demands of water supply, especially in the last few decades. In certain high demand areas, ground water development has already reached a critical stage, resulting in acute scarcity of the resource. Over- development of the ground water resources results in declining ground water levels, shortage in water supply, intrusion of saline water in coastal areas and increased pumping lifts necessitating deepening of ground water structures. Geogenic contamination of ground water due to concentration of Arsenic, Fluoride and Iron in excess of limits prescribed for drinking purposes (BIS, 2004) have also been observed in many parts of the country. To tackle the twin hazards of de-saturation of aquifer zones and consequent deterioration of ground water quality, there is an urgent need to augment the ground water resources through suitable management interventions. Artificial recharge has now been accepted world-wide as a cost-effective method to augment ground water resources in areas where continued overexploitation without due regard to their recharging options has resulted in various undesirable environmental consequences.

A 'Manual on Artificial Recharge of Ground Water', providing detailed guidelines on investigative techniques for selection of sites, planning and design of artificial recharge structures, monitoring and economic evaluation of artificial recharge schemes was brought out by Central Ground Water Board in 1994. It also included elaborate case studies and field examples of artificial recharge schemes from different parts of the world. The manual has been used extensively for planning and implementation of schemes for augmentation of ground water resources by various agencies.

Subsequent to the publication of the manual, Central Ground Water Board has brought out five publications on the topic in an attempt to disseminate the experiences gained during various ground water augmentation projects implemented by the Board in the country. They are:

1) Manual on Artificial Recharge of Ground Water (1994).
2) National Perspective Plan for Recharge to Ground Water by Utilising Surplus Monsoon Runoff (1996)
3) Guide on Artificial Recharge to Ground Water (1998)
4) Guide on Artificial Recharge to Ground Water (2000)
5) Master Plan for Artificial Recharge to Ground Water (2002)

Apart from these, Central Ground Water Board has also published technical brochures on various aspects of artificial recharge through its Regional Directorates, which served as guidelines to various governmental and non-governmental agencies and the general public. Some of the State Departments have also brought out manuals and guidelines on artificial recharge to ground water, which dealt with specific areas in most cases.

1.2 Present Endeavour

During 2004, it was decided to revise and update the existing manual by incorporating the latest advances in the fields of rainwater harvesting and artificial recharge. Accordingly, a Committee of the following officers was constituted:

Shri. C.S.Ramasesha Regional Director Central Ground Water Board, South Western Region, Bangalore.	Chairman
Shri. P. Nandakumaran, Scientist' D' Central Ground Water Board, South Eastern Coastal Region, Chennai	Member
Shri. S.K.Jain, Scientist' D' Central Ground Water Board, Central Region, Nagpur.	Member.
Shri. K.R.Sooryanarayana, Scientist 'D' Central Ground Water Board, South Western Region, Bangalore.	Member
Shri. Y.B.Kaushik, Scientist 'D' Central Ground Water Board, Central Headquarters, Faridabad.	Member Secretary

1.3 Outline of the Manual

The committee reviewed the available literature including information/data available on the experiences and case studies of various organisations as well as the material available on the web sites before finalizing the outline of the new manual. Emphasis was laid on Indian case studies and field experiences. Various aspects of assessing the availability of source water for recharge have been included in the chapter on 'Source Water'. Design aspects of artificial recharge of ground water have been further elaborated and new chapters on Roof Top Rainwater Harvesting and issues related to operation and maintenance of artificial recharge structures have been included.

The manual broadly covers the following topics

i) Background information on the global and Indian water scenario including status of ground water development and the demand - supply situation,
ii) Assessment of source water availability for artificial recharge,

iii) Need for artificial recharge and its historical perspective,
iv) Planning of artificial recharge schemes,
v) Studies involved in selection of sites for implementation of artificial recharge schemes,
vi) Techniques of artificial recharge and design aspects of recharge structures,
vii) Roof top rainwater harvesting,
viii) Monitoring and impact assessment of artificial recharge schemes,
ix) Economic evaluation of artificial recharge schemes and
x) Operation and maintenance of recharge structures

This manual is intended to be used as a guide and reference by professionals engaged in the implementation of artificial recharge schemes at various levels.

2. WATER RESOURCES DEVELOPMENT SCENARIO

2.1 Global and Indian Water Scenario

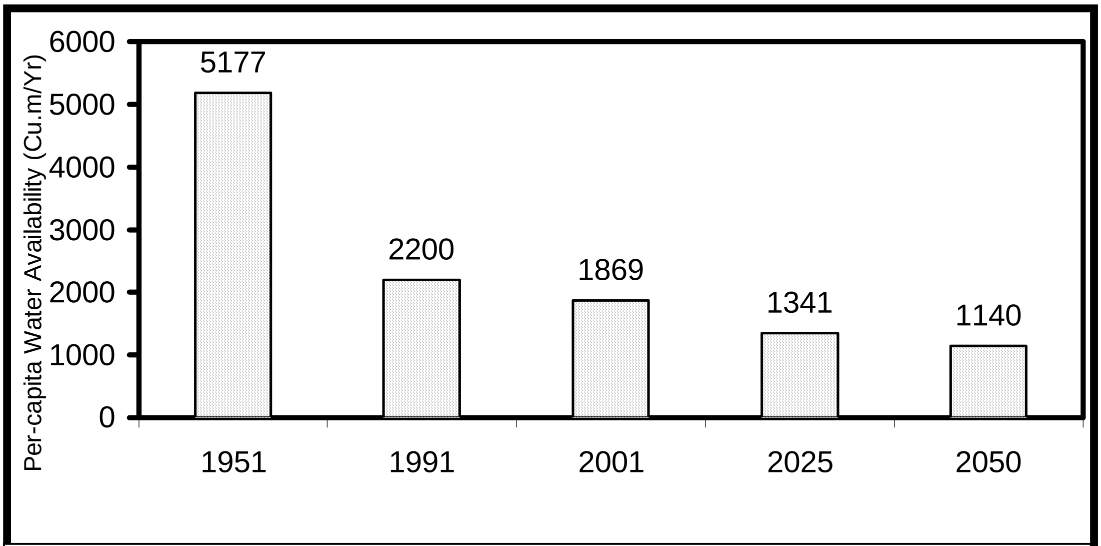
Many of us have an image of the world as a blue planet as 70 percent of the earth's surface is covered with water. The reality, however, is that 97 percent of the total water on earth of about 1400 Billion Cubic Meter (BCM) is saline and only 3 percent is available as fresh water. About 77 percent of this fresh water is locked up in glaciers and permanent snow and 11 percent is considered to occur at depths exceeding 800 m below the ground, which cannot be extracted economically with the technology available today. About 11 percent of the resources are available as extractable ground water within 800 m depth and about 1 percent is available as surface water in lakes and rivers. Out of the 113,000 BCM of rain and snow received on the earth, evaporation losses account for about 72,000 BCM, leaving a balance of about 41,000 BCM, out of which about 9000-14000 BCM is considered utilizable.

The annual precipitation including snowfall in India is of the order of 4000 BCM and the natural runoff in the rivers is computed to be about 1869 BCM. The utilizable surface water and replenishable ground water resources are of the order of 690 BCM and 433 BCM respectively. Thus, the total water resources available for various uses, on an annual basis, are of the order of 1123 BCM. Although the per capita availability of water in India is about 1869 cubic meters as in 1997 against the benchmark value of 1000 Cu m signifying 'water-starved' condition (**Fig.2.1**), there is wide disparity in basin-wise water availability due to uneven rainfall and varying population density in the country. The availability is as high as 14057 cu m per capita in Brahmaputra/ Barak Basin and as low as 307 cu m in Sabarmati basin. Many other basins like Mahi, Tapi, Pennar are already water stressed.

2.2 Historical Perspective

India is a vast country with very deep historical roots and strong cultural traditions. These are reflected in our social fabric and institutions of community life. In spite of social movements of varied nature through the millennia, we have retained the spirit and essence of these traditions and have remained attached to our roots. Some of our traditions, evolved and developed by our ancestors thousands of years ago have played important roles in different spheres of our life. One of the most important among these is the tradition of collecting, storing and preserving water for various uses.

The tradition probably started at the dawn of civilization with small human settlements coming up on the banks of rivers and streams. When, due to vagaries of nature, rivers and streams dried up or the flow in them dwindled, they moved away to look for more reliable sources of water. In due course of time, large settlements came up along the banks of perennial rivers that provided plentiful water. As the population increased, settlements developed into towns and cities and agriculture expanded. Techniques were developed to augment water availability by collecting and storing rainwater, tapping hill and underground springs and water from snow and glacier melt etc. Water came to be regarded as precious and its conservation and preservation was sanctified by religion. Various religious, cultural and social rituals prescribed purification and cleansing with water. Water itself had many applications in different rituals. Development of reliable sources of water such as storage reservoirs, ponds, lakes, irrigation canals etc. came to be regarded as an essential part of good governance. Emperors and kings not only built various water bodies but also encouraged the village communities and individuals to build these on their own. Wide-ranging laws were enacted to regulate their construction and maintenance and for conservation and preservation of water and its proper distribution and use.



BENCH MARKS

WATER STRESS	**-**	**Between 1700 and 1000 Cu.m /Year/Person**
WATER SCARCITY	**-**	**Below 1000 Cu.m/Year/Person**

Fig. 2.1 Per capita Water availability in India.

The *Satavahanas* (1^{st} Century B.C. - 2^{nd} Century A.D.) introduced brick and ring wells for extraction of water. Lake and well irrigation techniques were developed on a large scale during the time of *Pandya, Chera* and *Chola* dynasties in south India (1^{st} to 3^{rd} Century A.D) and large dams were built across *Cauvery* and *Vaigai* rivers. A number of Irrigation tanks were constructed by developing large natural depressions. Water resources development on a large scale took place during the *Gupta* era (300-500 A.D.). In the south, the *Pallavas* expanded the irrigation systems in the 7^{th} Century A.D. The famous *Cauvery Anicut* was built during this period. Large-scale construction of tanks (*Tataka*) for harvesting rainwater was also done during this period in Tamil Nadu. The *Chola* period (985-1205 A.D) witnessed the introduction of advanced irrigation systems, which brought about prosperity in the Deccan region. This included not only *anicuts* across rivers and streams but also a number of tanks with connecting channels. This new system was more reliable in terms of water availability and provided better flexibility in water distribution.

The *Rajput* dynasty (1000-1200 A.D) promoted irrigation works in northern India. The 647 sq km Bhopal Lake was built under King *Bhoja*. In eastern India, *Pal* and *Sen* Kings (760-1100 A.D) built a number of large tanks and lakes in their kingdoms. *Rajtarangini* of *Kalhana* gives a detailed account of irrigation systems developed in the 12^{th} Century in Kashmir.

In the Medieval period, *Mohammad Bin Tughlaq* (1325-1351 A.D.) encouraged the farmers to build their own rainwater harvesting systems and wells. *Feroze Shah Tughlaq* (1351-1388 A.D.) built the Western *Yamuna* Canal in 1355 to extend irrigation facilities in the dry land tracts of the present-day Haryana and Rajasthan. Emperor *Shahjahan* built many canals, prominent among them being the *Bari Doab* or the *Hasli Canal*. Under the rule of *Rangila Muhammad Shah*, the Eastern *Yamuna* Canal was built to irrigate large tracts in Uttar Pradesh.

The *Vijaynagar* Kings (1336-1548 A.D.) in the south took keen interest in building large and small storage tanks. *Anantraj Sagar* tank was built with a 1.37 km long earthen dam across the *Maldevi* River. The well-known *Korangal* dam was built under King *Krishnadevaraya*. The *Bahmani* rulers (1388-1422 A.D.) introduced canal irrigation for the first time in the eastern provinces of the Deccan. *Sultan Zain Uddin* (1420-1470 A.D.) introduced extensive network of canals in Utpalpur, Nadashaila, Bijbihara and Advin areas of Kashmir.

2.2.1 Pre - Independence Scenario

Agriculture has been the backbone of the Indian economy since time immemorial as bulk of the population in rural areas depended on agriculture for its livelihood. References to irrigation abound in the folklore and ancient literature of the country. The physiographical features of the area largely conditioned the nature of these works. In the arid and semi-arid plains of north India, perennial rivers like *Indus* and the *Ganges (Ganga)* easily diverted floods through inundation channels. In the peninsular part, where rivers are not perennial and rainfall is scanty, the practice of trapping storm water in large tanks for agricultural and domestic purposes was popular. In areas where high ground water table permitted lift irrigation, wells were common. The Grand *Anicut* across *Cauvery* River still remains by far one of the greatest engineering feats of ancient India. The *Viranrayana* and *Gangaikondacholapuram* tanks in Tamil Nadu and *Anantaraja Sagara* in Andhra Pradesh were constructed in the 10th and 13th centuries. The Western and Eastern *Yamuna* Canals and *Hasli* Canal in the *Ravi* were dug in the 16th and 18th centuries.

Under the British rule, irrigation development continued with renovation and improvement of existing irrigation works and with this experience, more new diversion works such as Upper *Ganga* Canal, Upper *Bari Doab* Canal, *Krishna* and *Godavari* delta systems were taken up and completed between 1836 and 1866. By the second half of the 19th century, irrigation potential to the tune of about 7.5 million hectares (m ha) had been developed.

Based on recommendations of the First Irrigation Commission, the period during 1900-1947 saw more irrigation development and the potential created increased to 22.5 m ha at the time of independence. There was a distinct shift from diversion works to survey, investigation and implementation of storage works during this period. Dams like *Krishnaraja Sagar* and *Mettur* were constructed across *Cauvery River* during this period. Storages were identified on *Tungabhadra*, *Krishna, Narmada*, *Sabarmati, Mahi* and *Sutlej* rivers. One reason for this shift was the realization that cheap diversion sites had already been exhausted. The need for productive irrigation and not merely protective irrigation was another. It was also realized that arid and drought areas could be benefited only by transferring water from other areas, which would be possible only with storage dams.

2.2.2 Post - Independence Scenario

After independence, the tempo of irrigation development was sharply accelerated with the objective of attaining self-sufficiency in food grains to meet the needs of a growing population. Construction of large storages like *Bhakra, Hirakud, Nagarjunasagar* - called by Pandit Nehru as ‘Temples of Modern India’, were taken up and completed. The criteria for economic evaluation of storage projects were changed from the financial return evaluation to a benefit- cost ratio evaluation. The return to the Government on investment was, thus, no more relevant but benefit to the farmer (at a cost to the Government) became the main evaluation criterion. The development of irrigation potential took place in successive plans by leaps and bounds and reached an impressive 89.5 m ha by the end of the Eighth Five Year Plan. The country achieved self-sufficiency in food grains by producing 200 million tones (MT) and import of food grains became a thing of the past.

The Second Irrigation Commission, set up in 1969, while not advocating any major change in the policy of irrigation development, cautioned in its report that areas like conjunctive use of surface and ground water, command area development, watershed development, increase in water rates to meet O & M costs as well as a part of the interest on investment also needed attention.

In pursuance of the above recommendations, Government of India took a number of policy decisions relating to command area development, protection of environment and forests, conjunctive use, flood plain zoning, regulation on use of ground water, preservation of water quality and the like. These measures have met with varying degrees of success and have had a bearing on the irrigation development achieved so far and also in shaping the future strategy in this sector.

Ground water development in India is primarily sustained by the farmers themselves or by institutional finance. The public sector outlay is mostly limited to ground water surveys, construction of deep tube wells for community irrigation, services provided and grants extended to small and marginal farmers. The flow of institutional finance is generally about 60 percent of the total outlay for ground water development.

Ground water development has, therefore, occupied an important place in Indian economy because of its role in stabilizing agriculture and as a means for drought management. During periods of droughts, additional dependence is laid on this resource since the storage levels in surface reservoirs dwindle and the impact of vagaries of weather on ground water is not as pronounced or is delayed. The stage of ground water development in the country as estimated in 1991 was about 32 percent. By March 2004, the stage of development has reached approximately 58 percent. This is also evident from the growth of ground water abstraction structures from the pre-plan period till date.

The number of ground water abstraction structures has increased from 5.8 million in 1982-83 to more than 18.5 million in 2000-2001. The growth of ground water abstraction structures in the country since 1982 is given in **Table 2.1** and **Fig 2.2**.

Table 2.1 Growth of Ground Water Abstraction Structures in India (1982-2001)

Type of Structure	**Number of Structures**			
	1982-1983	**1986-1987**	**1993-1994**	**2000-2001**
Dug well	5384627	6707289	7354905	9617381
Shallow Tube well	459853	1945292	3944724	8355692
Deep Tube well	31429	98684	227070	530194
Total	**5875909**	**8751265**	**11526699**	**18503267**

(Source: Report of the 3rd Minor Irrigation Census – 2000-2001)

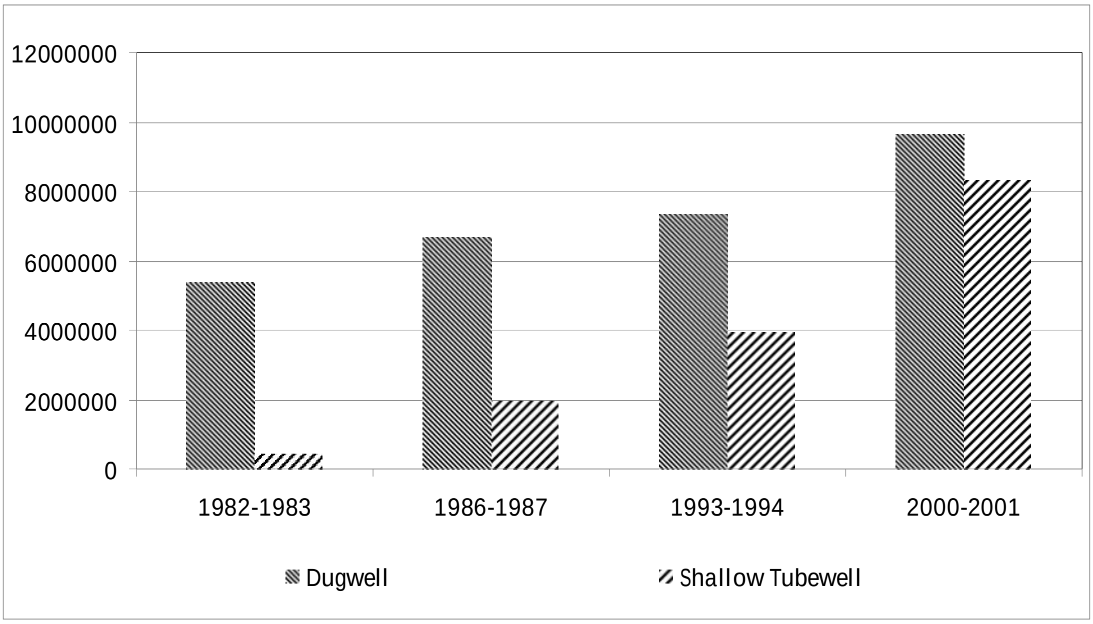


Fig. 2.2 Growth of Ground Water Abstraction Structures in India (1982-2001)

2.3 Efficacy of Ground Water Resource Development

Ground water plays an important role in sustaining India's economy, environment, and standard of living. It is not only the main source for water supply in urban areas for domestic uses, but also is the largest and most productive source of irrigation water. The investments in this sector, as part of Minor Irrigation by public and private sector, are going on since independence. The Minor Irrigation potential created, which was just 32 percent during the 1st Plan, increased to 84.4 percent during the 7th Plan period (1992-1997). This is in spite of the fact that investment in Minor Irrigation Sector had been decreasing in subsequent plans from 56.6 percent (1966-69 Annual Plan) to 19.7 percent (1997-2000) of the total plan outlay for irrigation.

Ground water irrigation began to expand rapidly with the large-scale cultivation of high yielding crop varieties in the second half of the 1960s. More than 50 percent of

India’s agricultural output comes from areas irrigated with ground water. Because agriculture and allied activities contribute roughly 30 percent of India’s Gross Domestic Product (GDP), with crops accounting for three- fourths of this, the contribution of ground water (with a package of associate inputs) to India’s GDP is about 9 percent. The relation between new technology and ground water development has been two-way. Not only has ground water irrigation helped to spread new technology, some of the profits earned through new technologies have been ploughed back into ground water development, leading to the well- known “tub well explosion” in northwest India. This was facilitated by the government’s efforts to promote rural electrification and the banking industry’s institutional credit support, especially after 1969.

The significance of ground water in the economy is due to the fact that agricultural yields are generally higher by one – third to half in areas irrigated with ground water than in areas irrigated with water from other sources. This is primarily due to the fact that ground water offers greater control over the supply of water when compared to other sources of irrigation. As a result, ground water irrigation encourages complementary investments in fertilizers, pesticides, and high-yielding crop varieties, leading to higher yields.

The strong link between ground water and economic growth has underlain the development strategy of the country. A special agricultural strategy launched for eastern India (comprising eastern U.P, Bihar, West Bengal, Assam, Orissa, and eastern Madhya Pradesh) in the mid 1980s, for instance, relied heavily on the exploitation of ground water. Since 1987, drilling of free tube wells and subsidies for pump sets have been provided throughout the region. This approach appears to have paid rich dividends. Rice production, the main food crop of this region, increased rapidly and reasonable progress was made on poverty reduction.

Development of ground water has led to increased “drought proofing” of India’s agricultural economy. In the 1960s, ground water was a relatively insignificant source of irrigation, particularly in eastern India. In 1965-66, monsoon rainfall (June to September) was 20 percent below normal, leading to drought conditions, which resulted in the decline of food production by 19 percent at the national level. In contrast, in 1987-88, rainfall was almost 18 percent below normal, but food grain production declined only by 2 percent over the previous year’s level. Much of this improvement can be attributed to the spread of irrigation in general and of ground water irrigation in particular.

The impact of Ground water development has extended beyond the owners of wells. Studies have indicated that farmers who own wells achieve the highest yields, while farmers who purchase water achieve higher yields than farmers who depend on canal irrigation alone, but not as high as the yields achieved by the owners of wells. They also consume more fertilizer, labour, and other inputs. Thus, the expansion of ground water irrigation is a major catalyst for rural development.
Ground water plays a key contributory role to agricultural GDP and drought mitigation. The spread of ground water irrigation supports employment generation and thus rural development and poverty alleviation. Small and marginal farmers, having 76 percent of land holdings, account for 38 percent of net area irrigated by

wells. As productivity is higher in irrigated land, better access to irrigation reduces poverty in rural areas.

2.4 Emerging Challenges

Ground water management is the foremost challenge being faced today by the organizations dealing with ground water in India. The activities and policies affecting ground water need to reflect the priority issues with the overall objective of providing water security through ground water management in a major part of the country.

Central Ground Water Board, being the apex organization at the central level with vast experience in the ground water sector, has taken a proactive role in identifying various key issues, which need immediate attention. These issues are discussed below in brief

2.4.1 Ground Water Depletion

Indiscriminate ground water development has led to substantial ground water level declines both in hard rocks and alluvial areas threatening sustainability of this resource. Long-term decline of ground water levels is being observed in many areas, mostly in the states of Rajasthan, Gujarat, Tamil Nadu, Punjab, Delhi and Haryana. Apart form this, in most of the cities depending on ground water for drinking water supplies, water level declines up to 30 m and more have been observed. Traditional water harvesting methods, which were in vogue in arid and semi-arid areas of the country have either been abandoned or have become defunct in most cases. There is an urgent need to revive these methods.

In some parts of the country like Rajasthan, Gujarat, Haryana and Delhi, the in-storage or static ground water resources are also limited and depletion in ground water level is resulting in ground water drought scenario. In such areas, it may become very difficult for the State to provide water security for various users. Excessive ground water development has resulted in deterioration of ground water quality in coastal areas due to saline water ingress. Ground water development, therefore, needs to be regulated and augmented through suitable measures to provide sustainability and protection. Dependence on use of ground water for agriculture due to monsoon failures is accelerating ground water depletion. Excessive withdrawal of ground water is further compounding the stress on ground water system due to free/subsidized power in some States. In order to tackle the burgeoning problem of water level decline, it is necessary to take up schemes for water conservation and artificial recharge to ground water on priority.

2.4.2 Ground Water Pollution

Ground water resources in several areas of the country are getting polluted due to over application of fertilizers and pesticides, indiscriminate disposal of effluents from industries and urban sewerage. Surveillance studies to determine the type and migration of pollution and measures for its control have become an absolute necessity from the point of view of long-term sustainability of ground water resources. Purpose-driven studies need to be undertaken to find suitable mitigation measures to combat this problem.

Presence of naturally occurring Arsenic, Fluoride and Iron in ground water in excess of permissible limits recommended for human consumption prohibits its use for drinking purposes in several states of India. Research and Development studies need to be taken up for finding cost-effective solutions to this problem. Dilution of pollutant concentration through ground water recharge can be one of the effective ways to mitigate the hazards of high concentration of these constituents. It is also desirable that rural water supply schemes be formulated and arrangements made to utilize fluoride / arsenic rich water for purposes other than drinking.

2.4.3 Drinking Water Shortage in Urban Areas

There are several urban areas in the country where water supply systems are based mainly on ground water resource. Sustainability of urban water supply is one of the core issues the planners across the country are facing at present. The problem may get aggravated in near future with the rapid pace of urbanization being witnessed in India. Potable drinking water is an important input for providing municipal supply to the urban complexes. Due to steep increase in population, the stress on ground water system has increased tremendously resulting in steep water level declines in and around these cities. These problems could be solved to some extent by

i) Shifting of ground water pumpage from the center of the cities to flood plain areas having proven capabilities of sustaining high yielding tube wells wherever possible,
ii) Recycling and reuse of water,
iii) Dual water supply systems for drinking and other domestic uses,
iv) Roof Top Rainwater Harvesting and
v) Regulatory measures through proper pricing and metering of water supplied.

2.4.4 Seawater Ingress in Coastal Aquifers

The unconsolidated deltaic and coastal sediments form thick and regionally extensive aquifers having prolific yield potential that can sustain deep, moderate to high capacity tube wells. Although considerable fresh ground water resources have been identified in regionally extensive deltaic and coastal tracts, particularly along the east coast, inherent quality problems restrict their development. The ground water in these aquifers exists in a fragile dynamic equilibrium with seawater. Indiscriminate exploitation of ground water from such aquifers can disturb this equilibrium and result in the development of landward hydraulic gradient, ultimately leading to seawater intrusion into the fresh water aquifers. Coastal aquifers in parts of Gujarat, Tamil Nadu and Andhra Pradesh are already suffering from the problem of salinity ingress.
Measures to prevent/control saline water intrusion into coastal aquifers include

i) Regulation of ground water development in coastal areas.
ii) Formation of a freshwater ridge parallel to the coast through artificial recharge.
iii) Formation of a pumping trough through a series of pumping wells aligned parallel to the coast

3. ARTIFICIAL RECHARGE OF GROUND WATER

The term artificial recharge has different connotations for various practitioners. Artificial recharge to ground water is defined as the recharge that occurs when the natural pattern of recharge is deliberately modified to increase recharge (ASCE 2001). The process of recharge itself is not artificial. The same physical laws govern recharge, whether it occurs under natural or artificial conditions. What is artificial is the availability of water supply at a particular location and a particular time. In the broadest sense one can define artificial recharge as "any procedure, which introduces water in a pervious stratum".

The term artificial recharge refers to transfer of surface water to the aquifer by human interference. The natural process of recharging the aquifers is accelerated through percolation of stored or flowing surface water, which otherwise does not percolate into the aquifers. Artificial recharge is also defined as the process by which ground water is augmented at a rate exceeding that under natural condition of replenishment. Therefore, any man-made facility that adds water to an aquifer may be considered as artificial recharge (CGWB, 1994)

Artificial recharge aims at augmenting the natural replenishment of ground water storage by some method of construction, spreading of water, or by artificially changing natural conditions. It is useful for reducing overdraft, conserving surface run-off, and increasing available ground water supplies. Recharge may be incidental or deliberate, depending on whether or not it is a by-product of normal water utilization.

Artificial recharge can also be defined as a process of induced replenishment of the ground water reservoir by human activities. The process of supplementing may be either planned such as storing water in pits, tanks etc. for feeding the aquifer or unplanned and incidental to human activities like applied irrigation, leakages from pipes etc.

3.1. Concept of Recharge

Flow below the land surface takes place due to the process of infiltration. The soil will not get completely saturated with water unless water supply is maintained for prolonged periods. If water is applied only intermittently, there may be no recharge during the first infiltration or even between two subsequent infiltrations. The evolution of water in the soil during the period between two instances of infiltration is referred to as redistribution. Recharge may take place even when no hydraulic connection is established between the ground surface and the underlying aquifer.

The hydraulic effects generated by artificial recharge are basically of two types, *viz.* piezometric effect and volumetric effect. The piezometric effect results in a rise of the piezometric surface, the magnitude of which depends on the geologic and hydraulic boundaries of the aquifer being recharged and the type, location, yield and duration of the recharge mechanism. It is also related to the ratio of transmissivity (T) of the aquifer and the replenishment coefficient (C), which is equal to the storage coefficient. Other factors such as capillary forces, water temperature and presence of air bubbles in the aquifer also have an impact on the piezometric effect.

The volumetric effect is related to the specific yield, replenishment coefficient, transmissivity and the geologic and hydraulic boundaries of the aquifer. Model studies that were checked through field experiments have demonstrated that the bulk of the recharge water moves according to two systems of flow, one resulting in a spreading-out effect with a speed related to the recharge flow and the other in a sliding effect, with a speed related to ground water flow.

3.2 Need for Artificial Recharge

Natural replenishment of ground water reservoir is a slow process and is often unable to keep pace with the excessive and continued exploitation of ground water resources in various parts of the country. This has resulted in declining ground water levels and depletion of ground water resources in such areas. Artificial recharge efforts are basically aimed at augmentation of the natural movement of surface water into ground water reservoir through suitable civil construction techniques. Such techniques inter-relate and integrate the source water to ground water reservoir and are dependent on the hydrogeological situation of the area concerned.

Occurrence of rainfall in India is mostly limited to about three months in a year. The natural recharge to ground water reservoir is restricted to this period only in a major part of the country. Artificial recharge techniques aim at extending the recharge period in the post-monsoon season for about three or more months, resulting in enhanced sustainability of ground water sources during the lean season.

In arid regions of the country, rainfall varies between 150 and 600 mm/ year with less than 10 rainy days. A major part of the precipitation is received in 3 to 5 major storms lasting only a few hours. The rates of potential evapotranspiration (PET) are exceptionally high in these areas, often ranging from 300 to 1300 mm. In such cases, the average annual PET is much higher than the rainfall and the annual water resource planning has to be done by conserving the rainfall, by storing the available water either in surface or in sub-surface reservoirs. In areas where climatic conditions are not favourable for creating surface storage, artificial recharge techniques have to be adopted for diverting most of the surface storage to the ground water reservoirs within the shortest possible time.

In hilly areas, even though the rainfall is comparatively high, scarcity of water is often felt in the post-monsoon season, as most of the water available is lost as surface run-off. Springs, the major source of water in such terrains, are also depleted during the post monsoon period. In such areas, rainwater harnessing and small surface storages at strategic locations in the recharge areas of the springs can provide sustainable yields to the springs as well as enhance the recharge during and after rainy season.

3.3 Purposes and Principles of Artificial Recharge

There are many reasons why water is deliberately placed into storage in ground water reservoirs. A large percentage of artificial recharge projects are designed to replenish ground water resources in depleted aquifers and to conserve water for future use. Other such projects recharge water for various objectives such as control of salt-water encroachment, filtration of water, control of land subsidence, disposal of wastes and recovery of oil from partially depleted oil fields.

In certain coastal areas of the world, artificial recharge systems for blocking inland encroachment of seawater are in operation. Most of these schemes rely on the injection of fresh water through wells in order to build up a pressure barrier that will retard or reverse encroachment of salt water resulting from excessive withdrawals from the wells. In such schemes, most of the injected water is not directly available for use, but serves as a hydraulic mechanism to allow better use of existing ground water reserves.

Attempts have been made in a few places to overcome land subsidence caused by excessive extraction of ground water by forcing water under pressure into the underlying ground water reservoirs. The success of such experiments of repressurizing to stop land subsidence is inconclusive.

From the point of view of artificially storing water for future use, the basic requirement is to be able to obtain water in adequate amounts and at the proper times in order to accomplish this goal. Some schemes involve the impoundment of local storm runoff, which is collected in ditches, basins or behind dams, after which it is placed into the ground. In other localities, water is sometimes brought into the region by pipeline or aqueduct. In the latter case, the water is an import and represents an addition to whatever natural water resources occur in the region. Another approach is to treat and reclaim used water being discharged from sewer systems or industrial establishments.

3.4 Advantages of Artificial Recharge

Artificial recharge is becoming increasingly necessary to ensure sustainable ground water supplies to satisfy the needs of a growing population. The benefits of artificial recharge can be both tangible and intangible. The important advantages of artificial recharge are

- Subsurface storage space is available free of cost and inundation is avoided
- Evaporation losses are negligible
- Quality improvement by infiltration through the permeable media
- Biological purity is very high
- It has no adverse social impacts such as displacement of population, loss of scarce agricultural land etc
- Temperature variations are minimum
- It is environment friendly, controls soil erosion and flood and provides sufficient soil moisture even during summer months
- Water stored underground is relatively immune to natural and man-made catastrophes
- It provides a natural distribution system between recharge and discharge points
- Results in energy saving due to reduction in suction and delivery head as a result of rise in water levels

3.5 Implementation of Artificial Recharge Schemes

Successful implementation of artificial recharge schemes will essentially involve the following major components

- Assessment of source water
- Planning of recharge structures
- Finalisation of specific techniques and designs
- Monitoring and impact assessment
- Financial and economic evaluation
- Operation and maintenance

4. SOURCE WATER

Availability of source water is one of the basic prerequisites for taking up any artificial recharge scheme. The source water available for artificial recharge could be of the following types:

i) *Insitu* precipitation in the watershed / area
ii) Nearby stream/ spring / aquifer system
iii) Surface water (canal) supplies from large reservoirs located within the watershed/basin
iv) Surface water supplies through trans-basin water transfer
v) Treated Municipal/industrial wastewaters
vi) Any other specific source(s)

The availability of water for artificial recharge from all these sources may vary considerably from place to place. In any given situation, the following information may be required for a realistic assessment of the source water available for recharge.

i) The quantum of non-committed water available for recharge
ii) Time for which the source water will be available.
iii) Quality of source water and the pre-treatment required.
iv) Conveyance system required to bring the water to the proposed recharge site.

Rainfall and runoff available constitute the major sources of water for artificial recharge of ground water. Rainfall is the primary source of recharge into the ground water reservoir. Other important sources of recharge include seepage from tanks, canals and streams and the return flow from applied irrigation. For proper evaluation of source water availability, a thorough understanding of rainfall and runoff is essential. Collection and analysis of hydrometeorological and hydrological data have an important role to play in the assessment of source water availability for planning and design of artificial recharge schemes. These are elaborated in the following sections.

4.1 Rainfall

Rainfall in the country is typically monsoonal in nature. ‘Monsoon’ literally means seasonal wind. It is basically a part of the trade wind system. The southeast trade winds and northeast trade winds converge at the Inter-Tropical Convergence Zone (ITCZ). Due to uneven distribution of land and water masses, it is crooked in shape and keeps shifting seasonally. During its northwards movement, it draws the southeast trades along with it. After crossing the equator, the winds change direction by 90 degrees (due to Coriolis force), taking a southwesterly direction. Hence, these seasonal winds are named Southwest monsoon. It lasts for four months, from June to September. While traversing the vast stretches of water, (Bay of Bengal and Arabian Sea), these winds pick up lot of moisture. On an average, annually, about 1120 mm of rainfall is received in the country. Bulk of this rainfall occurs during Southwest monsoon.

These moisture-laden winds normally hits the Kerala coast around May end. As it advances over the peninsula, copious amounts of rainfall occur all along the west coast and the adjoining mountains. After crossing the mountains, the current weakens. At the same time, the Bay of Bengal branch of the monsoon gives rise to heavy rainfall in the Bay islands during the month of May. This branch encounters the hill ranges of Northeast and then takes a westerly course. As a consequence, heavy rains occur in the northeast as also along the foothills of Himalayas. As it advances further, rainfall decreases towards west, almost becoming negligible west of *Aravalli* hill ranges in Rajasthan. This monsoon normally takes a month's time to cover whole of the country (late June or early July). Thus, the entire country is covered by the summer monsoon for two months, July and August, making them the wettest months.

The monsoon starts withdrawing gradually by early September and leaves the country by middle of October. The withdrawal of the Southwest monsoon is a result of shifting of ITCZ southwards. In its wake, the Northeast monsoon sets in. This monsoon lasts for nearly three months, from October to December. It is a relatively dry season as compared to its summer cousin. It is largely confined to the southeast and interior southern parts of the country. Rainfall is confined mainly to the month of October and to a lesser extent up to the middle of November.

4.1.1 Measurement of Rainfall

Rainfall is measured by a rain gauge, either manual or automatic. It is installed in an open area on a concrete foundation. The distance of the rain gauge from the nearest object should be at least twice the height of the object. It should never be on a terrace or under a tree. The gauge as also other instruments may be fenced with a gate to prevent animals and unauthorized persons from entering the premises. Measurements are to be made at a fixed time, normally at 0830 hrs. In case of heavy rainfall areas, measurements are made as often as possible.

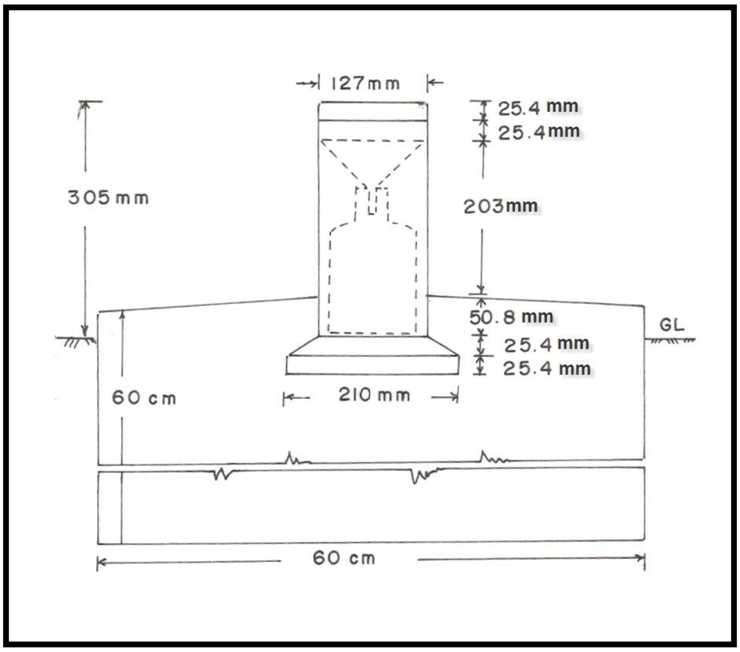


Fig. 4.1 Non-recording Rain Gauge

A manual gauge is basically a collector (funnel) and a bottle (**Fig 4.1**). It is made of Fibreglass-reinforced Plastic. The stem of the collector or receiver is led into a bottle kept in the base unit.

Rainwater collected in the bottle is measured by a calibrated jar or cylinder. The gauges are designed on the basis of cross sectional area of the collector or funnel, either 200 sq cm or 100 sq cm. The former is recommended for use where rainfall in 24 expected to be more than 200 mm. The latter is used where rainfall in a day does not exceed 200 mm. Thus, one cm rain gives 100 cc of water in the bottle in the case of 100 sq cm collector or 200 cc in the case 200 sq cm collector.

An automatic rain gauge gives a continuous record of rainfall. In addition to recording directly a) the total amount of rainfall that has fallen since the record was started, b) the times of onset and cessation of rain and therefore, c) the duration of rainfall. It also gives d) the rate of rainfall. Installation procedure is the same as that for manual gauge. Normally, automatic gauges are installed alongside manual gauges for use as a standard, by means of which the readings of the recording rain gauge can be checked, and if necessary, adjusted.

Rainwater entering the gauge at the top of the cover is led via the funnel to the receiver. The receiver consists of a float chamber and a siphon chamber. A pen is mounted on the stem of the float and as the water level rises in the receiver, the float rises and the pen records on a chart mounted on a clock drum. The drum revolves once in 24 hours or 7 days. Siphoning occurs automatically when the pen reaches the top of the chart and as the rain continues the pen rises again from zero line of the chart. If there is no rain, the pen traces a horizontal line from where it leaves off rising. The diameter of the funnel is 203 mm and height of the gauge is 600 mm.

There are other types of recording rain gauges such as tipping bucket, digital rain gauge etc, which facilitate telemetry of data.

4.1.2 Rain Gauge Network

For proper assessment of water resources, a good network of rain gauges is a must. The planning of such a network primarily depends on physical factors which affect hydrology such as topography, morphology, land use and soil types. In hilly areas, where heavy rainfall characterized by extreme variability is experienced, the network should be carefully planned. More the variability of rainfall, denser should be the rain gauge network. As per the IS: 4987-1968, the recommended rain gauge network density in plains is one rain gauge for every 520 sq km, whereas in moderately elevated areas (average elevation up to 1000 m), it is one in 260 to 390 sq km. Hilly areas, where very heavy rainfall is expected, are also areas of extreme rainfall variability. The network density in such areas, if economically feasible, should be one rain gauge for every 130 sq km. As far as possible, 10 percent of the rain gauge stations should be equipped with automatic (self recording) rain gauges.

An important factor in the design of the network is the accuracy with which rainfall over the catchment is to be assessed. A relation involving optimum number of rain gauges required and variability of rainfall among the existing rain gauges is expressed as

$$\mathbf{N = (Cv/P)^2 \quad or \quad P = (Cv)/(N^{1/2})}$$

where N is the optimum number of rain gauges, Cv is coefficient of variability of the rainfall values of the existing rain gauge stations and P, the permissible percentage error in the estimate of basin mean rainfall.

Normally, the mean rainfall is estimated up to an error (P) not exceeding 10 percent. If ‘N’ increases, the error would decrease. Thus, depending on the desired accuracy of the estimate, the number of rain gauges should be planned. For micro level studies of small watersheds, the error may not exceed 5 percent. In case of routine hydrological investigations, error of estimate may not exceed 10 percent. Relative distances between the gauges, accessibility, operating costs and availability of trained observers are to be taken into account while setting up new gauges.

The allocation of additional gauges depends on the spatial distribution of existing rain gauging stations and the variability of rainfall over the basin. For this purpose, isohyets of equal intervals are drawn, based on the existing gauges. Areas between two successive isohyets and their proportion with respect to total area are computed. The optimum number (i.e., existing plus additional) should be distributed to the different isohyetal zones in proportion of their areas.

Example: A catchment has 4 rain gauges recording an annual rainfall of 800, 540, 445 and 410 mm. Optimum number of rain gauges (N) required for estimating the average depth of rainfall is computed in the following manner.

Mean rainfall	= 548.75	Standard Deviation	= 176.26
Coefficient of variation (Cv)	= 32.12%	$N = (Cv / P)^2$	= 10.32 i.e. 10

It can be seen that the existing network of 4 rain gauges is inadequate and 6 new gauges are to be installed for a realistic estimate of average depth of rainfall.

4.1.3 Normals of Rainfall Data

Length of rainfall data records to be considered is an important factor in the analysis of rainfall. If the frequency distribution of mean annual rainfall becomes stable after a certain period, the addition of further years of observations does not add significantly to the accuracy. The length or period of record needed to achieve stability varies between seasons and regions. From experience it is observed that rainfall data of 30 years is adequate under Indian conditions. This period encompasses dry as well as wet cycles and is called the normal period. Averages of normal periods are termed normals. These normals need updating to account for changes in environment and land use. The current normal period is 1971-2000. Normals or averages, based on data for that period are used for making comparisons with data of the following decade.

4.1.4 Double Mass Curve

Consistent rainfall data is essential for resources evaluation. Due to change of location of a rain gauge or its exposure conditions, the data becomes inconsistent. This can be rectified by plotting cumulative rainfall of the gauge in question against average cumulative rainfall of a number of surrounding gauges. Such a plot is known as a Double Mass Curve (**Fig. 4.2**).

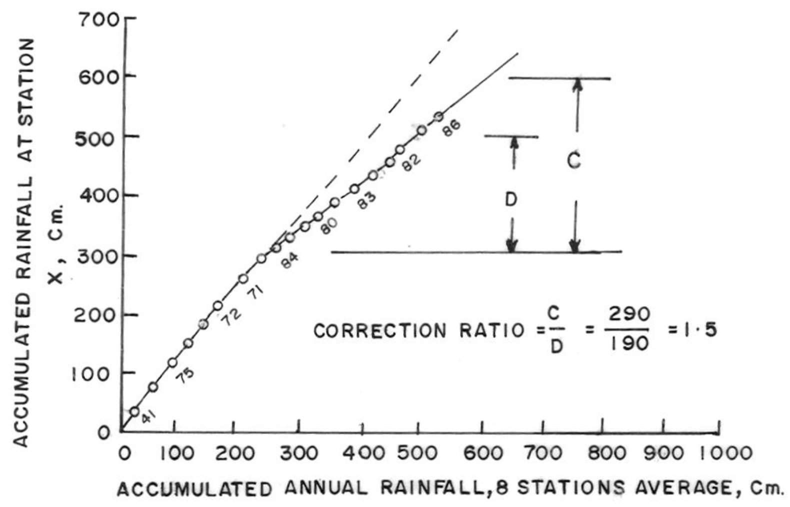


Fig. 4.2 Double Mass Curve

In case of inconsistency, there will be a change in slope of the straight plot. It is assumed that the average of group of stations used as basis of comparison does not have any inconsistency in itself. Only significant changes in slope are considered. Usually, values of previous period i.e., before change in slope are adjusted to the current period i.e., after change of slope. The adjustment factor is the ratio of the slope after change and slope before change.

4.1.5 Moving Averages

Change in the trends of an event such as rainfall may either be real, caused by climatic conditions or be apparent, resulting from inconsistencies due to damage to the instrument or change of location. The apparent change can be verified, and if necessary, rectified by means of double mass curves as described above. Evidence of real trends may be ascertained from the study of progressive long-term averages using 3 year or 5 year moving averages (**Fig.4.3**). For three year moving average, the rainfall is averaged over successive three-year periods. The first average is obtained for years one, two and three, the second average for years two, three and four and so on. The first average is plotted against year two, the second one against three and so on. In the case of 5 year moving average, years one, two, three, four and five are averaged and plotted against year three and so on. Selection of odd period is to facilitate ease of plotting. The period can be changed to 7 years or even 9 years.

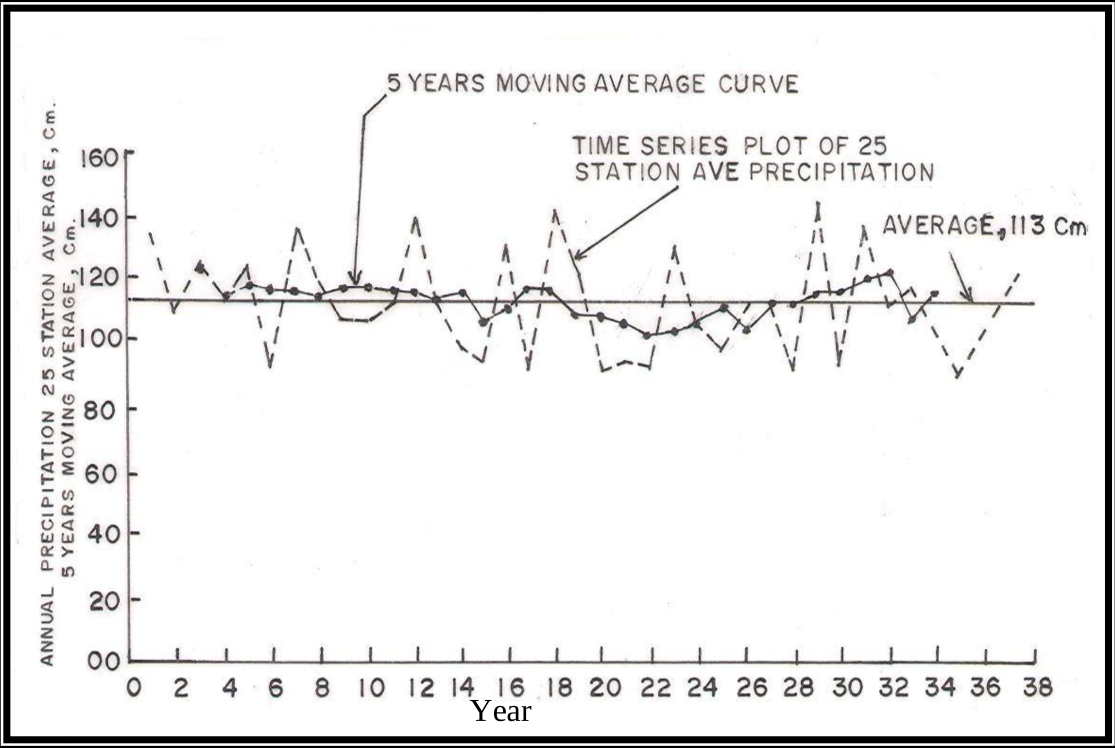


Fig. 4.3 Moving average

4.1.6 Supplementing Data

It is often necessary to supplement incomplete rainfall records by estimating values that are missing at one or more stations. Short period missing data of say, a single storm can be interpolated from an isohyetal map drawn with available data. For longer periods, say a month or an year, normal rainfalls are considered for interpolation. In one approach, it is assumed that the ratio of monthly or annual rainfall of two adjacent gauges is equal to the ratio of the normal rainfalls for the same period of the two gauges. It is expressed as:

$$Rx = Ry\ (Nx / Ny)$$

where 'x' is the rain gauge whose records are to be interpolated and 'y', the nearest rain gauge station. Rx is the rainfall at gauge 'x' and Ry, the rainfall at gauge 'y'. Nx and Ny are normal rainfall at 'x' and 'y' respectively.

Example:

Rain gauge	Normal	Actual
X	1125	?
Y	910	865

Actual Rainfall at X = 865(1125/910) = 1069

In another approach, at least three rain gauges with continuous records, as close to and as evenly spread around the gauge with missing records as possible, are chosen. If the normal rainfall at each of the gauge is within 10 percent of that for the gauge with missing records, then the arithmetic mean of the rainfall of the three surrounding gauges is used as that of the gauge in question. If the percentage is more than 10, the following equation is used:

$$Rx = \frac{1}{3}\left[\frac{Nx}{Na}Ra + \frac{Nx}{Nb}Rb + \frac{Nx}{Nc}Rc\right]$$

where Nx, Na, Nb and Nc, are the normal rainfalls of the gauge in question and the three surrounding gauges and Rx, Ra, Rb and Rc are the interpolated value of the gauge in question and the actuals of three gauges.

Example :	Rain gauge	Normal	Actual
	A	1125	875
	B	910	1021
	C	765	915
	X	830	?

Rainfall at X = 1 / 3 (((830/1125)* 875)+((830/910)* 1021) + ((830/765)*915))) = 856.5

4.1.7 Determination of Average Rainfall

In resource evaluation of a drainage basin, average depth of rainfall of a number of rain gauges is required. The average is usually obtained by any of the three methods; Arithmetic mean, Theissen polygon and isohyetal. The first one is a simple average whereas the other two give weighted averages.

4.1.7.1 Arithmetic Mean Method

In this method, the average depth of rainfall is computed as

$$P = (P_1 + P_2 + P_3 + \ldots\ldots\ldots\ldots\ldots\ldots + P_n)/ n$$

where, P is the average rainfall, n, the number of years of data and P_1, P_2, P_3 ……… P_n, precipitations measured at stations 1, 2,3 ……… n.

4.1.7.2 Theissen Polygon Method

In this method, weights are assigned to each rain gauge depending on its relative location. This method involves constructing polygons around each gauge, which are a result of perpendicular bisectors of lines joining two adjacent rain gauges. The polygon thus formed form the boundary of the effective area assumed to be controlled by the gauge, or in other words, the area closer to the gauge than to any other gauge. The ratio of the area of each polygon to the total area is the weight. The average or weighted rainfall is the sum of the product of the rainfall and weight of each gauge. P1 to Pn are the rainfall at gauges 1 to n and A1 to An are the areas of the respective polygons (**Fig. 4.4**).

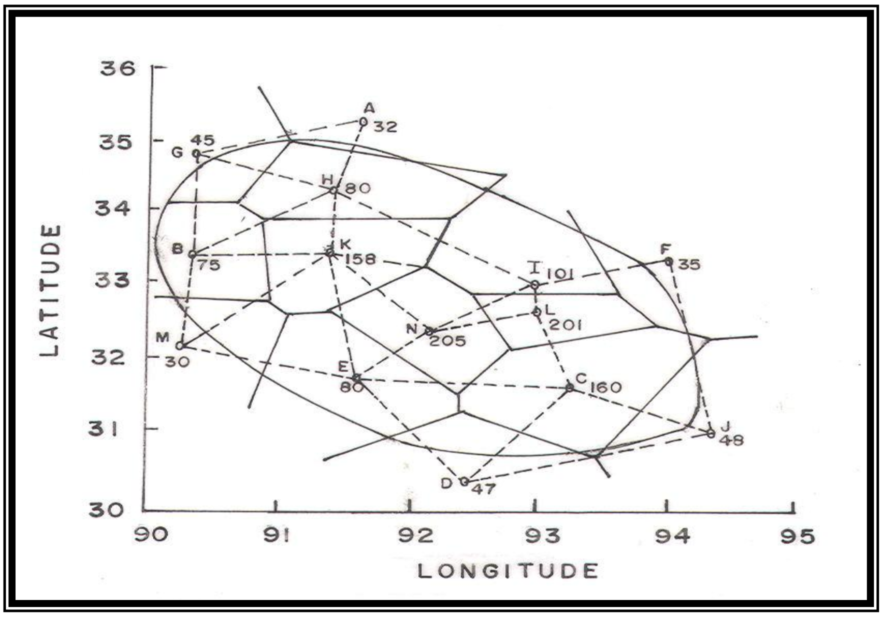


Fig. 4.4 Theissen Polygons

An example for working out average rainfall using this method is shown in **Table. 4.1**.

Table 4.1 Example to Work out Weighted Average Using Theissen Polygon Method

Rainfall (mm)	**Area** (sq km)	**Weight**	**Weighted Average Rainfall (mm)**
165	18	0.011	1.8
371	311	0.192	71.2
488	282	0.174	84.9
683	311	0.192	131.1
391	52	0.032	12.5
757	238	0.147	111.3
1270	212	0.131	166.4
1143	197	0.121	138.3
Total	1621	1.000	717.5

Weighted average is worked out as 717. 5 mm

4.1.7.3 Isohyetal Method

In this method, isohyets are drawn connecting points of equal rainfall and areas between two successive isohyets computed (**Fig.4.5**). The weight in this case is the ratio of the area of two successive isohyets to the total area. The weighted average is given by the sum of the products of weights and average contour value of corresponding isohyets.

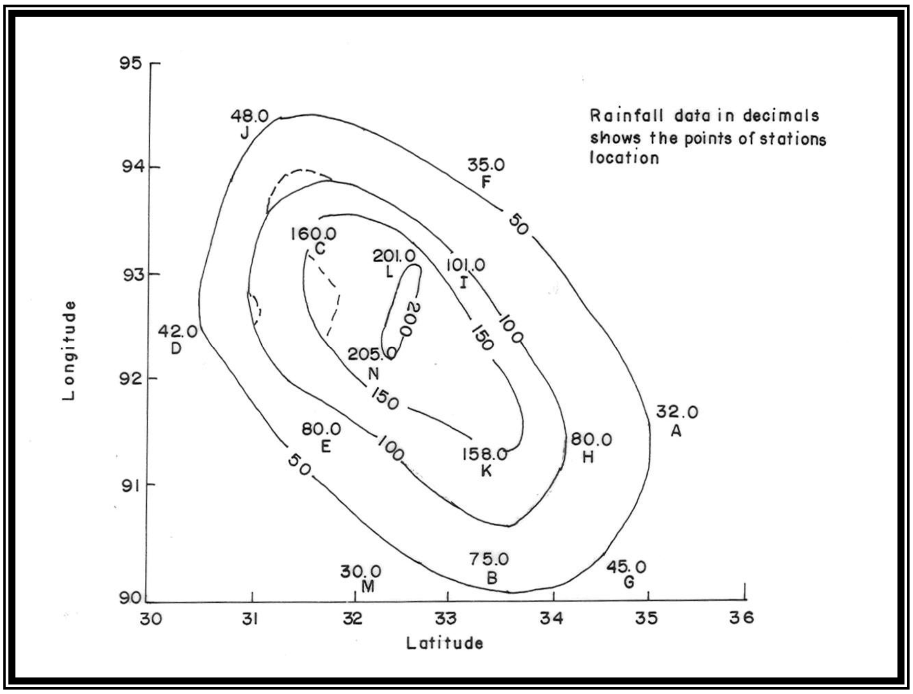


Fig 4.5 Isohyetal Method

An example for working out average rainfall using isohyetal method is shown in **Table. 4.2.**

Table 4.2 Example to Work out Weighted Average Rainfall Using Isohyetal Method

Isohyet (mm)	Area enclosed (Sq km)	Area Net (Sq km)	Weight	Mean Rf (mm)	Weighted Average Rainfall (mm)
1270	34	34	0.021	1350	28.35
1020	233	199	0.123	1170	143.91
760	534	300	0.185	890	164.65
510	1041	508	0.313	640	200.32
250	1541	500	0.308	380	117.04
<250	1621	80	0.049	200	9.80
		1621			**664.07**

Weighted average is worked out as 664.07 mm

The arithmetic mean method gives equal weights to all the rain gauges. Apart from being quick and easy, it yields fairly accurate results if the rain gauges are uniformly distributed and are under homogeneous climate. The polygon method is laborious. But once weights of each rain gauge are computed, it gives results quickly. This method takes care of non-uniform distribution of rain gauges. However, variability in rainfall due to elevation differences is not taken care of. Another serious drawback of this method is a situation when the polygons are to be redrawn due to addition or deletion of rain gauges to the network. The third method overcomes most of the deficiencies of the other two. However, drawing isohyets and computing weights every time estimations are to be made is tedious. The accuracy of the method also depends on the skill with which the isohyets are drawn. It is reasonable to rely on polygon method for averages of plain areas and the isohyetal method for hilly areas.

4.2 Runoff

Precise estimation of runoff is the basic and foremost input requirement for the design of recharge structures of optimum capacity. Unrealistic runoff estimates of catchments yield often leads to the construction of oversized or undersized structures, which, in any case, must be avoided.

Runoff is defined as the portion of the precipitation that makes its way towards rivers or oceans as surface or subsurface flow. After the occurrence of infiltration and other loses from the precipitation (rainfall), the excess rainfall flows out through the small natural channels on the land surface to the main drainage channels. Such types of flow are called *surface flows.* A part of the infiltrated rainwater moves parallel to the land surface as subsurface flow, and reappears on the surface at certain other points. Such flows are called *interflows*. Another part of the *infiltrated* water percolates downwards to ground water and moves laterally to emerge in depression and rivers and joins the surface flow. This type of flow is called the *subsurface flow* or *ground water flow.*

4.2.1 Hydrograph

A plot of the stream discharge against the elapsed time gives the flow hydrograph. The time scale could correspond to a storm period, a month, a season, a year or any other similar scale. The stream flows are classified as perennial, intermittent and ephemeral. Perennial streams always carry some water on account of replenishment by ground water throughout the year. Intermittent streams receive varying supplies of ground water, which is more during the wet season and dries up in the summer. Ephemeral streams do not get any supply of ground water and behave like storm drains in which the flow occurs only due to the overland flow caused by a stream. These streams cause flash floods too.

4.2.2 Estimation of Runoff

Runoff can be estimated by various methods. These can be classified under the following headings:

Empirical formulae and tables
Runoff Estimation based on Land Use and Treatment.
Rational Method and
Empirical formulae for flood peak

4.2.2.1 Empirical Formulae and Tables

4.2.2.1.1 Binnie's Percentages: Sir Alexander Binnie was probably among the first to study the relationship of runoff to rainfall with a view to express the former as a percentage of the latter (**Table.4.3**)

Table 4.3 Binnie's Percentages for Computation of Runoff

Annual Rainfall (mm)	Runoff (%)	Annual Rainfall (mm)	Runoff (%)
500	15	900	34
600	21	1000	38
700	25	1100	40
800	29		

The percentages are based on observations on two rivers in Madhya Pradesh.

4.2.2.1.2 Coefficients: The runoff 'R' in cm and rainfall 'P' in cm can be correlated as R = KP, where 'K' is the runoff coefficient. The runoff coefficient depends on factors affecting runoff. This method is applicable only for small projects. The usual values of K are as given in **Table 4.4**.

Table 4.4 Usual Values of Runoff Coefficients (K)

Type of Area	K
Urban Residential	0.3 - 0.5
Forests	0.05 - 0.2
Commercial & Industrial	0.9
Parks, farms, Pastures	0.05 - 0.3
Asphalt or concrete pavement	0.85

4.2.2.1.3 Barlow's Tables: T.G. Barlow carried out studies of catchments mostly under 130 sq km in Uttar Pradesh and gave the following values of K (in percentage) for various types of catchments (**Table 4.5**).

Table 4.5 Barlow's Percentage Runoff Coefficients

Class	Description of Catchment	Percent runoff
A	Flat, cultivated and black cotton soils	10
B	Flat, partly cultivated-various soils	15
C	Average	20
D	Hills and plains with little cultivation	35
E	Very hilly and steep, with hardly any cultivation	45

These percentages are for the average type of monsoon and are to be modified by the application of the following coefficients according to the nature of the season as shown in **Table 4.6**.

Table 4.6 Barlow's Runoff Coefficients for Different Natures of Season

Nature of Season.	Class of catchments.				
	A	B	C	D	E
1. Light rain, no heavy downpour	0.70	0.80	0.80	0.80	0.80
2. Average or varying rainfall, no continuous downpour	1.00	1.00	1.00	1.00	1.00
3.Continuous downpour	1.50	1.50	1.60	1.70	1.80

He divided special tropical rainfall into the following four classes:

(i) Negligible falls: All rainfalls under 12 mm a day unless continuous for several days; also rainfalls 12 to 40 mm a day, when there is no rain.
(ii) Light falls: All rainfalls up to 25 mm a day followed by similar or heavier falls. Steady pours of 25 to 40 mm a day, when there is no rain of similar or greater amount before or after that.
(iii) Medium falls: Rainfalls from 25 to 40 mm a day when preceded or followed by any but light falls.
(iv) Heavy Falls:
(a) All rainfalls over 75 mm a day or continuous falls at 50 mm a day.
(b) All rainfalls of an intensity of 50 mm or more per hour.

He gave the runoff percentages as shown in the following table by combining the type of catchment and nature of the season (**Table 4.7**).

Table 4.7 Barlow's Runoff Percentages

Nature of rainfall	Percentage of Flow in Catchments of Different Type				
	A	**B**	**C**	**D**	**E**
1. Negligible falls	-	-	-	-	-
2. Light falls	1	3	5	10	15
3. Medium falls	10	15	20	25	33
4. Heavy falls	20	33	40	55	70

4.2.2.1.4 Strange Tables: These tables provide quick and easy access to daily runoff, which is given as a percentage of total monsoon rainfall (**Table 4.8a**) or as a percentage of daily rainfall (**Table 4.8b**). These are based on extensive studies in the then Bombay Presidency but can be applied in similar areas.

Table 4.8(a) Strange Table Showing Depth of Runoff as Percentage of Total Monsoon Rainfall and Yield of Runoff

Good Catchment				Average Catchment			Bad Catchment		
Total Mon-soon Rainfall in inches	Percent-age of Runoff to Rainfall	Depth of Runoff due to Rainfall in inches	Yield of Run-off from Catchment per square mile in Mcft	Percent-age of Runoff to Rainfall	Depth of Runoff due to Rainfall in inches	Yield of Run-off per square mile in Mcft	Percent-age of Run-off to Rainfall	Depth of Runoff due to Rainfall in inches	Yield of Runoff per square mile in Mcft.
1	2	3	4	5	6	7	8	9	10
1	0.1	0.001	0.002	0.1	0.001	0.001	0.05	0.0005	0.000
2	0.2	0.004	0.009	0.15	0.003	0.006	0.1	0.002	0.004
3	0.4	0.012	0.028	0.3	0.009	0.021	0.2	0.006	0.014
4	0.7	0.028	0.65	0.5	0.021	0.048	0.3	0.014	0.032
5	1.0	0.050	0.116	0.7	0.037	0.087	0.5	0.025	0.058
6	1.5	0.090	0.209	1.1	0.067	0.156	0.7	0.045	0.104
7	2.1	0.147	0.341	1.5	0.110	0.255	1.0	0.073	0.170
8	2.8	0.224	0.520	2.1	0.168	0.390	1.4	0.112	0.260
9	3.5	0.315	0.732	2.6	0.236	0.549	1.7	0.157	0.366
10	4.3	0.430	0.999	3.2	0.322	0.749	2.1	0.215	0.499
11	5.2	0.572	1.329	3.9	0.429	0.996	2.6	0.286	0.664
12	6.2	0.744	1.728	4.6	0.558	1.296	3.1	0.372	0.864
13	7.2	0.936	2.174	5.4	0.702	1.630	3.6	0.463	1.087
14	8.3	1.162	2.699	6.2	0.871	2.024	4.1	0.581	1.349
15	9.4	1.410	3.276	7.0	1.057	2.457	4.7	0.705	1.638
16	10.5	1.600	3.930	1.8	1.260	2.927	5.2	0.840	1.951
17	11.6	1.972	4.581	8.7	1.479	3.435	5.8	0.986	2.290
18	12.8	2.304	5.353	9.6	1.728	4.014	6.4	1.152	2.676
19	13.9	2.641	6.135	10.4	1.980	4.601	6.9	1.420	3.067
20	15.0	3.000	6.970	11.25	2.250	5.227	7.5	1.500	3.485
21	16.1	3.381	7.855	12.0	2.535	5.891	8.0	1.690	3.927
22	17.3	3.806	8.842	12.9	2.854	6.631	8.6	1.903	4.421
23	18.4	4.232	9.832	13.8	3.174	7.374	9.2	2.116	4.916
24	19.5	4.680	10.873	14.6	3.510	8.154	9.7	2.340	5.436
25	20.5	5.150	11.964	15.4	3.862	8.973	10.3	2.575	5.982
26	21.8	5.668	13.168	16.3	4.251	9.876	10.9	2.834	6.584
27	22.9	6.183	14.364	17.1	4.637	10.773	11.4	3.091	7.182
28	24.0	6.720	15.612	18.0	5.040	11.709	12.0	3.360	7.806
29	25.1	7.279	16.911	18.8	5.459	12.683	12.5	3.639	8.455
30	26.3	7.890	18.330	19.7	5.917	13.747	13.8	3.945	9.165
31	27.4	8.495	19.733	20.5	6.370	14.799	13.7	4.247	9.866
32	28.5	9.120	21.188	21.3	6.840	15.891	14.2	4.560	10.594
33	29.6	9.768	22.693	22.2	7.326	17.019	14.8	4.884	11.345
34	30.8	10.472	24.323	23.1	7.854	18.246	15.4	5.236	12.164
35	31.9	11.165	25.939	23.9	8.373	19.454	15.9	5.582	12.969
36	33.0	11.880	27.600	24.7	8.910	20.700	16.5	5.940	13.800
37	34.1	12.617	29.312	25.5	9.462	21.984	17.0	6.308	14.656
38	33.53	13.414	31.163	27.4	10.060	23.372	17.6	6.760	15.591
39	36.4	14.196	32.980	22.3	10.647	24.735	18.2	7.098	16.490
40	37.5	15.000	34.848	28.1	11.250	26.136	18.7	7.500	17.424
41	38.8	15.826	36.767	28.9	12.537	27.575	19.3	7.913	18.383
42	39.8	16.716	38.835	29.8	13.190	29.126	19.9	8.358	19.417
43	40.9	17.587	40.858	30.6	13.860	30.643	20.4	8.793	20.429
44	42.0	18.480	42.933	31.5	13.546	32.199	21.0	9.240	21.466
45	43.1	19.395	45.058	32.3	15.283	33.793	21.5	9.697	22.529
46	44.3	20.378	47.342	33.2	16.003	35.506	22.8	10.189	23.671
47	45.4	21.338	49.572	34.0	16.724	37.179	22.7	10.669	24.786
48	46.5	22.320	51.854	34.8	17.493	38.890	23.2	11.160	25.927
49	47.6	23.324	54.186	35.7	17.493	40.639	23.8	11.662	27.093
50	48.8	24.400	56.686	36.8	18.336	42.514	24.4	12.200	28.343
51	49.9	25.449	59.123	37.4	19.086	44.342	24.9	12.724	29.561
52	51.0	26.520	61.611	38.2	19.890	46.208	25.5	13.260	30.805
53	52.1	27.613	64.151	39.0	20.709	48.313	26.0	13.806	32.075
54	53.3	28.782	66.866	39.9	21.586	50.149	26.6	14.391	33.433
55	54.4	29.920	60.510	41.8	22.440	52.132	27.2	14.960	34.755
56	55.5	31.080	72.205	41.6	23.310	54.453	27.7	15.540	36.102
57	56.6	32.262	74.951	42.4	24.196	56.213	28.3	16.131	37.471
58	57.8	33.524	77.883	43.3	25.143	58.412	28.9	16.762	38.941
59	58.9	34.751	80.734	44.1	26.063	60.550	29.4	17.375	40.367
60	60.0	36.000	83.035	45.0	27.000	62.726	30.0	18.000	41.817

Table 4.8(b) Strange Table Showing Daily Runoff Percentage

Daily Rain-fall, mm	Runoff Percentage and Yield when the State of Ground is					
	Dry		Damp		Wet	
	%	Yield	%	Yield	%	Yield
5	-	-	4	0.20	7	0.35
10	1	0.10	5	0.25	10	1.00
20	2	0.40	9	1.80	15	3.00
25	3	0.75	11	2.75	18	4.50
30	4	1.20	13	3.90	20	6.00
40	7	2.80	18	7.20	28	11.20
50	10	5.00	22	11.00	34	17.00
60	14	8.46	28	16.80	41	24.60
70	18	12.61	33	25.10	48	33.60
75	20	15.00	37	27.75	52	41.25
80	22	17.60	39	31.20	55	44.00
90	25	22.50	44	39.60	62	55.80
100	30	30.00	50	50.00	70	70.00

Note: - for good or bad catchment, add or deduct up to 25% of yield.

For use of these tables, catchments have been classified as Good, Average or Bad as follows:

Good catchment: Hills or plains with little cultivation and moderately absorbent soil.

Average catchment: Flat partly cultivated stiff gravely/Sandy absorbent soil

Bad catchment: Flat and cultivated sandy soils.

4.2.2.1.5 Ingles and De Souza's formulae: Based on studies carried out for catchments in Western *Ghats* and plains of Maharashtra, C.C. Inglis and D’Souza gave the following relations:

For *Ghat* (Hilly) area, $R = 0.85 P - 30.5$

Where ‘R’ and ‘P’ are runoff and precipitation respectively, both expressed in cm.

For plains $$R = \frac{(P-17.8)\,P}{254}$$

4.2.2.1.6 Lacey's Formula: As per this formula, runoff (R) can be computed as

$$\frac{P}{1 + \frac{304.8}{P}\left(\frac{F}{S}\right)}$$

Where S = a catchment factor
F = monsoon duration factor

Lacey's values for the factor F / S for Barlow's classification of catchments are given in **Table 4.9**.

Table 4.9 Values of Lacey's Factor (F / S)

Sl. No.	Monsoon Class	Class of Catchments				
		A	B	C	D	E
1	Very short	2.0	0.83	0.50	0.23	0.14
2.	Standard length	4.0	1.67	1.00	0.58	0.28
3.	Very long	6.0	2.50	1.50	0.88	0.43

4.2.2.2 Estimation of Direct Runoff from Rainfall

In this method of runoff estimation, the effects of the surface conditions of a watershed area are evaluated by means of land use and treatment classes. Land use is the watershed cover and it includes every kind of vegetation, litter and mulch, and fallow as well as non-agricultural uses such as water surfaces (lakes, swamps, etc) and impervious surfaces (roads, roofs, etc.). Land treatment applies mainly to agricultural land uses and includes mechanical practices such as contouring or terracing and management practices such as grazing control or rotation of crops. The classes consist of use and treatment combinations actually to be found on watersheds. Land use and treatment classes are readily obtained either by observation or by measurement of plant and litter density and extent on sample areas.

4.2.2.2.1 Hydrological Soil Groups: There are four soil groups that are used in determining the hydrological soil cover complexes, which are used in a method for estimating the runoff from rainfall. A generalised soil map of India, giving the broad classification of all the major soils in India is shown in **Fig.4.6**. Major characteristics of these groups are described in **Table 4.10**. The classification is broad but the groups can be divided into sub-groups whenever such a refinement is justified. The infiltration rates and permeability of soils in different groups are shown in **Table 4.11** and **Table 4.12** respectively. In these tables, infiltration rate is the rate at which water enters the soil at the surface and which is controlled by surface conditions and permeability rate is the rate at which water moves in the soil, which is controlled by the nature and characteristics of soil horizons.

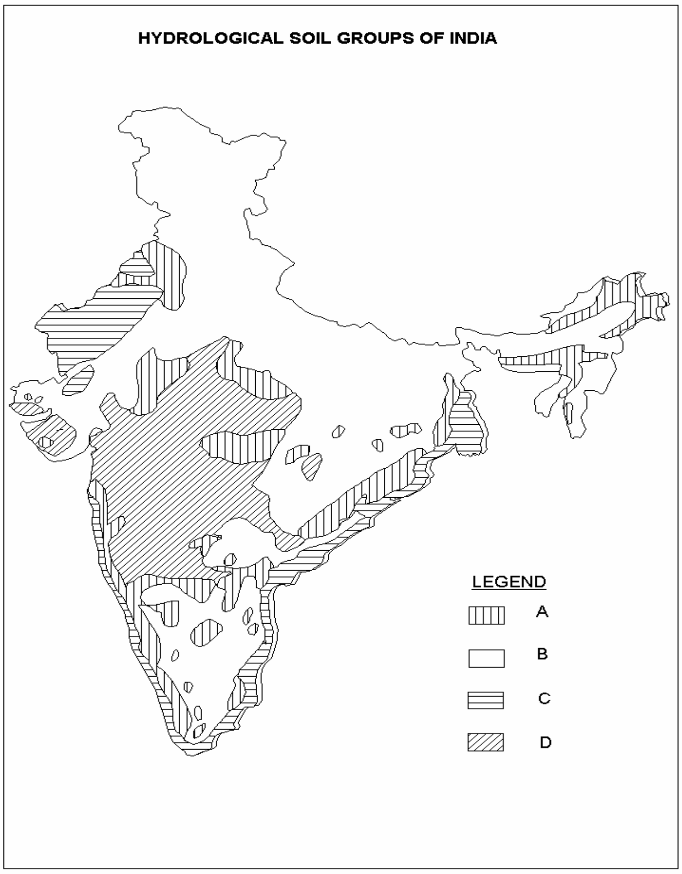


Fig.4.6 Hydrological Soil Groups of India.

Table 4.10 Hydrological Soil Groups

Soil Group	Description
A	Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.
B	Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
C	Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.
D	Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Table 4.11 Infiltration Rates

Sl. No.	Class	Rates / hr in		Remarks
		Inches	Millimeters	
1.	Very Low	Below 0.1	Below 2.5	Highly clayey soils
2.	Low	0.1 - 0.5	2.5 - 12.5	Shallow soils, clay soils, soils low in organic matter
3.	Medium	0.5 - 1.0	12.5 - 25.0	Sandy loams, silt loams
4.	High	Above 1.0	Above 25.0	Deep sands, well aggregated soils

Table 4.12 Relative Classes of Soil Permeability

Class	Permeability	
	Inches / hr.	mm/hr.
Slow		
1) Very slow	Less than 0.05	1.30
2) Slow	0.05 to 0.20	1.31 to 5.00
Moderate		
3) Moderately slow	0.20 to 0.30	5.01 to 20.00
4) Moderate	0.80 to 2.50	20.01 to 50.00
5) Moderately Rapid	2.50 to 5.00	50.01 to 130.00
Rapid		
6) Rapid	5.00 to 10.00	130.01 to 250.00
7) Very Rapid	Over 10.00	Over 250.00

4.2.2.2.2 Land Use and Treatment Classes: The commonly used land use and treatment classes are briefly described below. These classes are used in determining hydrologic soil- cover complexes, which are used in one of the methods for estimating runoff from rainfall.

a) Cultivated lands: These include all field crops such as maize, sugarcane, paddy and wheat.
b) Fallow lands: These are lands taken up for cultivation, but are temporarily out of cultivation for a period of not less than one year, and not more than 5 years. Current fallow lands are cropped areas kept fallow during the current year.
c) Uncultivated lands include:
   a. Permanent pastures and other grazing lands.
   b. Cultivable waste, which are lands available for cultivation whether or not taken up for cultivation or abandoned after a few years for one reason or another. Land once cultivated but uncultivated for 5 years in succession shall also be included in this category.
d) Forest area includes all lands classed as forest under any legal enactment dealing with forest or administered as forest whether State owned or private and whether wooded or maintained as potential forest land.
e) Tree crops include woody perennial plants that reach a mature height of at least 8 feet and have well defined stems and a definite crown shape.

f) Lands put to non-agricultural uses are areas occupied by buildings, roads, railroads etc.
g) Barren and uncultivable lands include areas covered by mountains, deserts etc.

4.2.2.2.3 Rainfall – Runoff Equations: The data generally available in India comprise rainfall measured by non-recording rain gauge stations. Rainfall-runoff relation developed for such data is given below

$$Q = [(P-Ia)^2] / [(P-Ia)+S]$$

Where Q is the actual runoff in mm, S, the potential maximum retention in mm, and Ia, initial abstraction during the period between the beginning of rainfall and runoff in equivalent depth over the catchment in mm.

In areas covered by black soils having Antecedent Moisture Conditions (AMC) II and III, Ia in the equation is equal to 0.1S, whereas in all other regions including those with black soils of AMC I, Ia is equal to 0.3S.

In order to show this relationship graphically, 'S' values are transformed into 'Curve Numbers (CN)' using the following equation

$$CN = 25400 / (254 + S)$$

Using the above equation, the following equations have been developed:

$$Q = [(P-0.3S)^2] / [(P+0.7S)] \text{ -------------------- } 1$$

$$Q = [(P-0.1S)^2] / [(P+0.9S)] \text{ -------------------- } 2$$

Equation 1 is applicable to all soil regions of India except black soil areas referred to in the section on 'Hydrological Soil Groups'. Equation 2 applies to black soil regions. This equation should be used with the assumption that cracks, which are typical of these soils when dry, have been filled. Therefore, equation 2 should be used where AMC falls into groups II and III. In cases where the AMC falls in group I, equation 1 should be used. The rainfall limits for AMC conditions are shown in **Table 4.13**

Table 4.13 Rainfall Limits for Antecedent Moisture Condition

AMC	5 – day Total Antecedent Rainfall (cm)	
	Dormant Season	Growing Season
I	< 1.25	< 3.5
II	1.25 to 2.75	3.50 to 5.25
III	> 2.75	> 5.25

Values of CN for different soils are given in **Table 4.14**

Table 4.14 Runoff Curve Numbers for Hydrologic Soil Cover Complexes (For watershed Condition II and Ia = 0.25)

Land Use/ Cover	**Treatment/ Practice**	**Hydrologic Condition**	**Curve Number for Hydrologic Soil Group**			
Fallow Row Crops	Straight Row	-	77	86	91	94
	Straight Row	Poor	72	81	88	91
	Straight Row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured and Terraced	Poor	66	74	80	82
	Contoured and Terraced	Good	62	71	78	81
Small Grain	Straight Row	Poor	65	76	84	88
	Straight Row	Good	63	75	83	87
	Contoured	Poor	63	74	82	85
	Contoured	Good	61	73	81	84
	Contoured and Terraced	Poor	61	72	79	82
	Contoured and Terraced	Good	59	70	78	81
Close seeded legumes or rotation meadow	Straight Row	Poor	66	77	85	89
	Straight Row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and Terraced	Poor	63	73	80	83
	Contoured and Terraced	Good	51	67	76	80
Pasture or Range		Poor	68	79	86	89
		Fair	49	69	79	84
	Contoured	Good	39	69	79	84
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
		Good	6	35	70	79
Meadow (Permanent)		Good	30	58	71	78
Woodlands (Farm Woodlots)		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads			59	74	82	86
Roads (Dirt)			72	82	87	89
Roads (Hard Surface)			77	84	90	92

4.2.2.3 Rational Method

This method was originally developed for urban catchments. Thus, the basic assumptions for development of this method were made for urban catchments. However, this method is fairly applicable to small agricultural watersheds of 40 to 80 hectares size (Chow, 1964).

The Rational method is based on the assumption that constant intensity of rainfall is uniformly spread over an area, and the effective rain falling on the most remote part of the basin takes a certain period of time, known as the time of concentration (Tc) to arrive at the basin outlet. If the input rate of excess rainfall on the basin continues for the period of time of concentration, then the part of the excess rain that fell in the most remote part of the basin will just begin its outflow at the basin outlet and with it, the runoff will reach its ultimate and the maximum rate. That is, the maximum rate of outflow will occur when the rainfall duration is equal to the time of concentration.

The above processes are explained in **Fig. 4.7**. Consider a drainage basin, which has rainfall of uniform intensity and of longer duration. On plotting the relationship between the cumulative runoff rate Q and time, the rate of runoff shows a gradual increase from zero to a constant value. The runoff increases with increase in flow from remote areas of the basin to its outlet. If the rainfall continues beyond the time of concentration, then there is no further increase in the runoff, and it remains constant at its peak value.

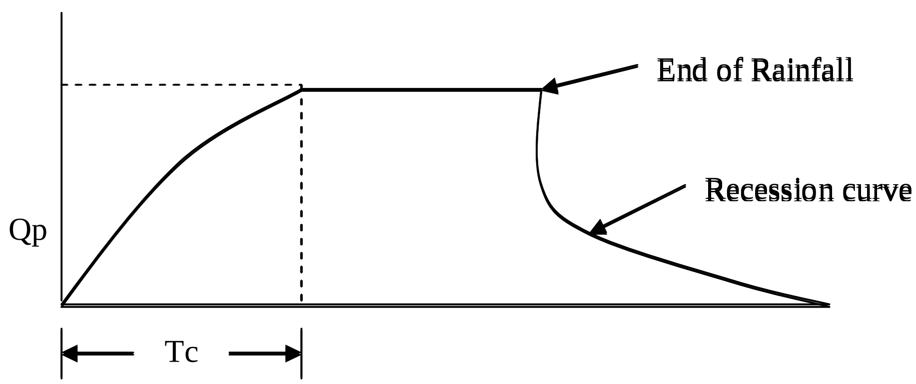


Fig. 4.7 Runoff Hydrograph Due to Uniform Rainfall

The relationship for peak runoff Q_p is then expressed as

$$Q_p = C\,I\,A$$

Where, C = coefficient of runoff
A = area of the catchment (drainage basin)
I = intensity of rainfall.

In metric units, this equation is expressed as.

$$Q_p = \frac{1}{3.6} \; C \; I \; A$$

Where, Q_p = peak runoff rate (m^3/s)

C = coefficient of runoff
I = mean intensity of precipitation (mm/h) for a duration equal to time of concentration, and for an accidence probability.
A = area of the drainage basin (km^2).

4.2.2.3.1 Runoff Coefficient Factor (C): The runoff coefficient factor (C) encompasses all other factors that affect the surface runoff, except the area (A) and the intensity of rainfall (I). It is defined as:

$$C = \frac{Q_p}{AI}$$

Under ideal conditions, C represents the ratio of runoff volume to rainfall volume. Ideal conditions are rare. Consequently, the values of C are significantly lower than the values obtained through the above ratio. A summary of the values of C developed by different research works in India for different soil conditions are given in **Table 4.15.**

Table 4.15 Values of Runoff Coefficient Factor (C) for Different Soil Conditions in India.

Type of Vegetation	Slope Range (%)	Runoff Coefficient (C) in		
		Sandy Loam Soil	Loam / Loam Clay Soil	Stiff Clay Soil
Woodland and forests	0-5	0.1	0.3	0.4
	5-10	0.25	0.35	0.5
	10-30	0.3	0.5	0.6
Grassland	0-5	0.1	0.3	0.4
	5-10	0.16	0.36	0.55
	10-30	0.22	0.42	0.6
Agricultural land	0-5	0.3	0.5	0.6
	5-10	0.4	0.6	0.7
	10-30	0.52	0.72	0.82

4.2.2.3.2 Intensity of Rainfall: The formula for the intensity of rainfall is expresses as.

$$I = \frac{KT_r^{\,a}}{(T_c + b)^n}$$

Where I is the intensity of rainfall, Tr, the recurrence interval, T_c, Time of concentration, and a, b, n are constants.

The values of parameters K, a, b, n for different zones of India have been developed by the ICAR scientists, and are shown in **Table 4.16**.

Table 4.16 Values of Parameters for Intensity – Duration - Return Period Relationships for Different Zones of India.

Zone	K	A	b	n
Northern zone	5.92	0.162	0.50	1.013
Central zone	7.47	0.170	0.75	0.960
Western zone	3.98	0.165	0.15	0.733
Southern zone	6.31	0.153	0.50	0.950

4.2.2.3.3 Time of Concentration (T_c): For determination of the time of concentration, the most widely used formula is the equation given by Kirpich (1940). However, for small drainage basins, the lag time for the peak flow can be taken to be equal to the time of concentration. The lag time can be determined by the Snyder's equation.

The Kirpich's equation is given as

$$Tc = 0.01947\ L^{0.77}\ S^{-0.385}$$

Where

Tc = time of concentration (min)

L = maximum length of travel of water (m)

S = slope of the drainage basin = HIL

H= difference in elevation between the most remote point of the basin and its outlet (m) and L, the maximum length of travel (m)

The time of concentration can also be determined as

$$Tc = 0.1947\ (K)^{0.77}$$

Where $K = \sqrt{\frac{L^3}{H}}$

The time of concentration is sometimes also determined by dividing the length of run with the average velocity of flow based on the slope of the channel as given in **Table 4.17.**

Table 4.17 Average Velocity Based on Channel Slope

Channel Slope %	Velocity (m/s).
1-2	0.6
2-4	0.9
4-6	1.2
6-10	1.5

4.2.2.4 Empirical Relationships for Determination of Peak Runoff

Empirical relationships can be applied to regions for which these are developed. There are some popular Runoff formulae in use in India, three of which are given below:

4.2.2.4.1 Dickens Formula: This formula was developed in the year 1865. It states that

$$Q_p = C_d\, A^{3/4}$$

Where

Q_p = peak discharge rate (m^3/s).
C_d = a constant (Dickens'), ranging from 6 to 30.
A = Drainage basin area (km^2).

For Indian conditions, suggested values for C_d are given in **Table 4.18**

Table 4.18 Suggested Values of C_d for Indian Conditions

Region	Topography	C_d
Northern states	Plains	6
	Hills	11-14
Central states	-	14.28
Coastal area	-	22.28.

4.2.2.4.2 Ryve's Formula

Ryve's formula was reported in the year 1884. It states that

$$Q_p = C_r\, A^{2/3}$$

Where

Q_p = Peak discharge rate (m^3/s).
A = Drainage basin area (km^2).
C_r = A constant (Ryves), as shown in **Table 4.19**

Table 4.19 Values of Ryves Constant

Region	C_r
Within 80 km from east coast	6.8
80-160 km from east coast	8.5
Hills	10.2

The Ryves formula is recommended for southern states of India.

4.2.2.4.3 Ingle's Formula: This formula was developed in areas of old Bombay state. It states that

$$Qp = \frac{123\ A}{\sqrt{A + 10.4}}$$

Qp = Peak discharge in Cumecs
A = Area of the catchment in sq km

Example

A catchment has an area of 5.0 km². The average slope of the land surface is 0.006 and the maximum travel depth of rainfall in the catchment is approximately 1.95 km. The maximum depth of rainfall in the area with a return period of 25 years is as tabulated in **Table 4.20**.

Table 4.20 Maximum Depth of Rainfall in an Area with a Return Period of 25 Years.

Time duration (min)	5	10	15	20	25	30	40	60
Rain fall depth (mm)	15	25	32	45	50	53	60	65

Consider that 2.0 Km² of the catchment area has cultivated sandy loam soil (c=0.2) and 3.0 Km² has light clay cultivated soil (c = 0.7). Determine the peak flow rate of runoff by using the Rational method.

Solution: The time of concentration is given by Kirpich's equation.

$$Tc = 0.01947\, L^{0.77} S^{-0.385}$$
$$= 0.01947\, (1950)^{0.77}\, (0.006)^{-0.385} \text{ min}$$
$$= 47.65 \text{ min.}$$

The maximum rainfall depth for 47.65 min duration would fall between the periods of 40-60 min and is located at 7.65 min after the 40 min period at which the maximum rainfall depth is 60 mm, as per the available data.

The rainfall depth during the 7.65 min period $= \frac{65-60}{20} \times 7.65 = 1.9$ mm

Therefore, for 47.65 min duration, the rainfall depth = 60 + 1.9 = 61.9 mm.

The average rainfall intensity (During the period of time of concentration) $= \dfrac{\text{maximum rainfall depth}}{Tc}$

$$= \frac{61.9 \times 60}{47.65} = 77.96 \text{ mm/hr}$$

Runoff coefficient, C $= \dfrac{(20 \times 0.2) + (3.0 \times 0.7)}{5.0}$

$$= \frac{0.4 + 2.}{5.0} = 0.5$$

Peak runoff rate, Qp $= \dfrac{CIA}{3.6}$ m³/s

$$= 1/3.6 \times 0.5 \times 77.96 \times 5.0$$

$$= 54.138 \text{ m}^3/\text{s.}$$

4.3 Quality of Source Water

The physical, chemical and biological quality of the recharge water also affects the planning and selection of recharge method. Physical quality of recharge water refers to the type and amount of suspended solids, temperature, and the amount of entrapped air whereas chemical quality refers to type and concentration of dissolved solids and gases. Biological quality refers to type and concentration of living organisms. Under certain conditions, any or all of these characteristics can diminish recharge rates.

4.3.1 Physical Quality

If suspended solids are present in the recharge water, surface application techniques are more efficient than subsurface techniques. Even though suspended particles may cause clogging, the infiltration surfaces are accessible for remedial treatment. Where indirect methods of recharge are used, suspended solids pose virtually no problem. Under such conditions, induced recharge would probably be one of the best methods. Ditch and furrows method is also well suited for large amounts of suspended solid loads because the steady flow of water inhibits settling. Basins should not be indiscriminately subjected to turbid water because surface clogging is almost certain to occur. If basins must be used for recharge with turbid water, they can be used in series, whereby the first basin acts as a clarifier for subsequent basins. This method requires more land, however, and is feasible only where land is readily available.

Where suspended solid loads in recharge water are high, subsurface application techniques, including deep pits, shafts, and wells, are prone to failure. Unless pre-treatment measures are provided, subsurface techniques should not be considered when the source water is turbid because clogging of injection wells is particularly troublesome, and well redevelopment is costly.

4.3.2 Chemical Quality

Recharge water should be chemically compatible with the aquifer material through which it flows and the native ground water to avoid chemical reactions that would reduce effective porosity and recharge capacity. Chemical precipitation and unfavourable exchange reactions, as well as the presence of dissolved gases, are causes for concern. Cation exchange reactions involving sodium in recharge water may cause clay particles to swell or disperse, thereby decreasing infiltration rate or aquifer permeability. Dissolved gases may alter aquifer pH or come out of solution, forming gas pockets that occupy pore space and decrease aquifer permeability.

Toxic substances in excess of established health standards must not be present in the recharge water unless they can be removed by pre-treatment or chemically decomposed by a suitable land or aquifer treatment system. If artificial recharge is for drinking purpose, then the source water must conform to the drinking water standards in vogue.

4.3.3 Biological Quality

Biological agents such as algae or bacteria may also be present in recharge water. Organic wastes may contain harmful bacteria or promote their growth and decay or organic materials may produce excess nitrate or other by-products. Growth of algae and bacteria during recharge can cause clogging of infiltration surfaces and may lead to the production of gases that further hinder recharge efforts. Although surface spreading removes most bacteria and algae by filtration before the recharge water reaches the aquifer, surface clogging can reduce the infiltration rate considerably. Injection of water containing bacteria and algae through wells is generally not recommended because it causes clogging of well screens or aquifer materials, which is difficult and costly to remedy.

The quality of source water is thus vitally important wherever direct recharge techniques are contemplated. In cases where *insitu* precipitation or water supplied from canals are used for recharge, no constraints on account of water quality may arise. However, in cases where waters in the lower reaches of rivers or recycled municipal/industrial waste waters are proposed to be used, the quality of water requires to be precisely analysed and monitored to determine the type and extent of treatment required.

In cases where the recharge is contemplated through spreading techniques, raw waste water can be used after primary sedimentation and secondary (biological) treatment to take advantage of filtration and bio-degradation that occurs as the water passes through the upper soil layers and zone of aeration. On the other hand, if the water is to be used for direct recharge, secondary treatment should be followed by chemical clarification (coagulation-flocculation-clarification). The water is then allowed to pass through adequate filter beds. The filtration is followed by tertiary treatment involving air tripping, granular activated carbon treatment, reverse osmosis and disinfection, in that order.

The consideration of chemical quality of source water will thus lead to decisions about the extent and type of treatment required, arrangements for treatment plants and the cost of source water. In case it is not possible to ensure the desired quality standard from the treatment, such source(s) may be avoided for recharging the ground water.

5. PLANNING OF ARTIFICIAL RECHARGE SCHEMES

Proper planning is essential for the successful outcome of any artificial recharge scheme. Planning of artificial recharge schemes involves the formulation of a suitable plan, under a given set of natural conditions, to augment the natural ground water recharge. An artificial recharge scheme may be aimed at recharge augmentation in a specific area for making up the shortage in ground water recharge compared to the ground water draft either fully or partially.

The area involved in artificial recharge projects may range from a watershed, a limited area covering an urban, rural or industrial centre or administrative units like Mandal/Block to large basins or larger administrative units like Districts/States. Though the steps involved in planning are essentially the same, the planning is done on different scales as per the required objectives and the area involved. Thus, planning of recharge scheme may be done at Mega level (State or Basin level), Macro level (District or sub-basin level) and Micro level (Block or Watershed level) at progressively larger scales. It is advisable to do State/Basin level planning at 1: 2000,000 scale, District / Sub-basin level planning at 1:250,000 scale and Block / Watershed level planning at 1:50,000 scale and so on.

Proper scientific investigations aimed at assessing the need and feasibility of an area for artificial recharge are necessary prerequisites for planning and implementation of any successful artificial recharge project. Detailed consideration of the following aspects is necessary for evolving a realistic plan for an artificial recharge scheme.

i. Establishment of ground facts, which includes
    - Need for artificial recharge
    - Estimation of sub-surface storage capacity of the aquifers and quantification of water required for recharge
    - Prioritisation of areas for artificial recharge
    - Source water availability
    - Assessment of source water
    - Source water quality
    - Suitability of the area for recharge in terms of climate, topography, soil and land use characteristics and hydrogeologic set-up
ii. Appraisal of economic viability
iii. Finalisation of Physical Plan.
iv. Preparation of a Plan document covering all the aspects mentioned above

5.1 Establishment of Ground Facts

An appraisal of the ground facts relevant to the need and suitability of an area for artificial recharge helps in deciding upon the most suitable scientific strategy for the formulation of artificial recharge schemes. The most important considerations in this regard are described in brief in the following sections.

5.1.1 Establishing the Need

Assessing the need for recharge augmentation in a scientific and objective manner forms the first step in planning a recharge scheme. Artificial recharge may be required for tiding over deficit situations in summer/winter seasons though sufficient water may be available for the year as a whole, or to combat perennial deficit situations getting compounded over the years. In the former case, there is need for building up additional ground water storage as and when it is available and to conserve it to ensure that the available supplies last through the lean season. In case where the ground water deficit gets compounded due to overexploitation, artificial recharge measures will often have to be coupled with economy measures for preventing misuse of water and regulation of ground water development through legislation for them to be effective.

The need for artificial recharge also requires to be prioritized according to its importance in the overall development perspective of the nation. Such prioritization will also help in deciding the economic viability of the scheme being contemplated. Recharge for catering to drinking water needs in adverse situations, preventive recharge to combat saline water ingress/land subsidence and augmentation of water supply to projects of strategic importance fall under the highest priority. A benefit cost ratio of 0.9: 1 or even less may be acceptable depending upon the conditions under which the project is being implemented. Providing subsistence irrigation in semi-arid and drought-prone areas comes under the next category where a benefit cost ratio of 1 may be considered adequate. Recharge augmentation for industrial use and irrigation augmentation in humid areas have the least priority and a benefit cost ratio of 1.5:1 or higher may be required in such situations.

In cases where the objective of recharge is replenishment of de-saturated aquifer zones or to arrest/reverse decline in ground water levels, the benefit cannot be directly reflected in terms of BC ratio as the benefits are mostly intangible. In such cases, a long-term declining trend of ground water levels, in the absence of significant negative departure of rainfall, may be attributed to over-development of ground water resources. Data pertaining to a period of at least 10 years is recommended for examining the trend of ground water levels in an area.

5.1.2 Estimation of Sub-surface Storage Capacity of Aquifers

The scope for artificial recharge in an area is basically governed by the thickness of unsaturated material available above the water table in the unconfined aquifer. Depth to water level, therefore, provides the reference level to calculate the volume of unsaturated material available for recharge. Depth to water level recorded during post-monsoon period is used for the purpose as areas where the natural recharge is not enough to compensate the ground water withdrawal, can be easily identified using the water level data. The average water levels for a period of at least 5 years is to be used in order to nullify the effects of variation in rainfall.

Contour maps prepared from the average post-monsoon water level data with suitable contour intervals can be used for assessment of available storage space. The inter-contour areas between successive contours are determined and the total area in which the water levels are below a certain cut-off level (say 3.00 m.bgl in phreatic aquifers), multiplied by the specific yield of the aquifer material gives the volume of sub-surface storage space available for recharge. The cut-off water level is so selected to ensure that the recharge does not result in water logging conditions in the area.

After assessing the subsurface storage space, the actual requirement of source water is to be estimated. Based on the experience gained from field experiments, the average recharge efficiency of the individual structure is to be specified (say 60-90%). To arrive at the total volume of actual source water required at the surface, the volume of water required for artificial recharge is calculated by multiplying the volume of sub-surface storage space with the reciprocal of recharge efficiency of the structure proposed.

Sample worksheets for estimation of sub-surface storage capacity and volume of water required for recharge is shown in **Table 5.1** and **Table 5.2** respectively.

Table 5.1 Sample Worksheet for Estimation of Sub-surface Storage Capacity

Sl. No.	Basin	Water-shed	Geographi-cal area (sq.km)	Area identified for artificial recharge (sq.km)	Depth to water level (Post-monsoon) below cut-off level (m)	Volume of unsaturated zone (M Cu m)	Average specific yield (%)	Total subsurface storage potential as volume of water (M Cu m)
1	2	3	4	5	6	7= (5x6)	8	9=(7x8)
1								
2								
3								
4								

Table 5.2 Sample Worksheet for Estimation of Volume of Water Required for Recharge

Sl. No	Basin / Sub basin / Watershed	Area Identified for Artificial Recharge* (Sq.km)	Sub surface Storage Potential** (M Cu m)	Recharge Efficiency (%)	Surface Water Requirement (M Cu m)
(1)	(2)	(3)	(4)	(5)	(6)=(4)x 100 / (5)
1					
2					
3					

*As in column 5 & ** column 9 of Table 5.1

5.1.3 Prioritisation of Areas for Artificial Recharge

It may not always be possible to implement artificial recharge projects in the entire area even though the need is established, due to various constraints such as lack of source water, shortage of funds for implementation of the projects etc. In such cases, it may be necessary to identify areas that require recharge augmentation most and to implement recharge projects accordingly.

Prioritisation of areas for artificial recharge is normally done by overlaying post-monsoon depth to water level maps with maps depicting the long-term trend of ground water levels. From these maps, it is possible to demarcate areas with various combinations of depth to water levels and water level trends. For example, if a depth to water level map having 3 m contour intervals is combined with a water level trend map with 0.1 m/year contour interval, it is possible to demarcate areas having

a) water levels in the range of 3 to 6 m.bgl and declining trend of 0.10 to 0.20 m/year.
b) water levels deeper than 9.00 m bgl and declining trend in excess of 0.40 m/year or
c) water levels deeper than 12.00 m bgl, but with a long term rising trend of 0.2 to 0.4 m/year.
d) Water levels in the range of 5.0 to 10.0m with declining trends during both pre-monsoon and post-monsoon season.

Normally, areas having deeper water levels and declining water level trends are given higher priority identification of area feasible for artificial recharge. Areas having shallow water levels / rising water level trends are not considered for inclusion in artificial recharge plan.

5.1.4 Availability of Source Water

A realistic assessment and quantification of the source water help design the storage capacity of the structure. Otherwise, there is a possibility of arriving at an improper design of the recharge structure. Various aspects of assessment of source water availability have been dealt with in the chapter on ‘Source Water’. In cases validated data on non-committed surplus runoff / any other possible source of water and its distribution in time and space is available with appropriate agencies, the same can be considered. The quality aspects of the water to be utilized for recharge needs to be ascertained from the available data and if required through detailed analysis.

5.1.5 Suitability of Area for Recharge

The climatic, topographic, soil, land-use and hydrogeologic conditions are important factors controlling the suitability of an area for artificial recharge. The climatic conditions broadly determine the spatial and temporal availability of water for recharge, whereas the topography controls the extent of run-off and retention. The prevalent soil and land use conditions determine the extent of infiltration, whereas the hydrogeologic conditions govern the occurrence of potential aquifer systems and their suitability for artificial recharge.

5.1.5.1 Climatic Conditions

In regions experiencing high (1000 to 2000 mm/year) to very high (>2000 mm/year) rainfall, such as the Konkan and Malabar coasts, North-eastern States, parts of lesser Himalayas in Uttar Pradesh and Himachal Pradesh, eastern part of Madhya Pradesh and parts of Bihar and Bengal, a major part of the water received during the rainy season goes as surface runoff. Only 5 to 10 percent of the total precipitation may infiltrate into the ground and reach the water table, which may be sufficient for adequate recharge. In areas of very high rainfall, the phenomenon of rejected recharge may also occur.

Most of such areas may not require artificial recharge of ground water and the best option is to store as much of the surplus water available as possible in large surface reservoirs, to be released to downstream areas during non-monsoon periods for direct use or to be used as source water for artificial recharge in suitable areas. The second and third order streams in such regions may have flow throughout the winter and the major rivers are normally perennial. The water in these streams and rivers, diverted, lifted or drawn through induced recharge may also be used as source water for artificial recharge.

In areas having moderate rainfall (750 – 1000 mm/year) such as eastern parts of Punjab and Maharashtra, eastern and central parts of Madhya Pradesh, parts of *Godavari* delta, eastern coast and Karnataka, adequate ground water resources are generally available only during the rainy season. A major component of the precipitation goes as surface runoff in these areas too and recharge may be 10 to 15 percent of annual precipitation. Ground water recharge is normally not sufficient to saturate the water table aquifers in deficit rainfall years. The second and third order streams normally do not have any flow during a major part of winter and only major streams may have some flow during summer.

The non-availability of surplus runoff beyond the rainy season may impose a severe constraint on artificial recharge to ground water in these areas. Diversion of water released from surface water reservoirs in the upper reaches of the catchments, water transferred from surplus basins or lifted from rivers wherever available may be required for sustaining irrigation water supplies. Hence, conserving as much of surface runoff as possible through watershed treatment measures, inducing additional recharge during and after rainy season and conserving ground water outflow through subsurface dykes may be suitable for such areas.

In semi-arid regions with low to moderate rainfall in the range of 400 to 700 mm/year, the annual precipitation may not even suffice to meet the existing water demand, and droughts may occur with regular frequency due to variations in rainfall. Western part of Punjab and Haryana, eastern Rajasthan and parts of Gujarat, Saurashtra, central Maharashtra and Telengana and Rayalseema regions of Andhra Pradesh fall under this category. The evapotranspiration losses in these areas are quite high and even though 15 to 20 percent of water gets infiltrated into the ground, the total ground water recharge will be limited because of the low rainfall. The stream flow in these regions is mostly restricted to the rainy season.

The replenishment of aquifers during rainy season generally is not enough to cater to the irrigation requirements during Rabi season in such areas, though it may be adequate for drinking water use through winter. Shortage of drinking water supplies is common during summer, which may be acute in years of deficit rainfall. Though recharge augmentation is warranted, due to lack of availability of source water, the only option available is to conserve as much of the surplus surface runoff during the short rainy season. Rainwater harvesting and runoff conservation measures for augmenting the ground water resources are appropriate in such situations.

In areas falling in arid zone, such as western Rajasthan desert, parts of Kutch region of Gujarat and Ladakh region of Jammu and Kashmir, the annual precipitation is less than 400 mm, the number of rainy days between 20 and 30 or even less and the coefficient of variation of rainfall is normally between 30 and 70 percent. The major component of outflow is evaporation and drainage is poorly developed in these areas. Infiltration of water may rarely exceed field capacity of soils and ground water recharge may be very small or negligible. Such areas may be left out of consideration for artificial recharge in spite of need unless trans-basin water is available. Rainwater harvesting may be contemplated in such regions for augmenting drinking water supplies. In case imported water is available, spreading or injection methods (for confined aquifers) may be considered depending on surface conditions (sandy/rocky), topographic set-up and salinity profiles of soils and the zone of aeration.

5.1.5.2 Topographic Set-up

The topographic set-up of an area controls the retention period of surface and ground water within a topographic unit. The gradients are very steep (more than 1:10) in the runoff zones, with very little possibility of infiltration. Such areas on hill-slopes may be suitable only for water conservation measures like gully plugging, bench terracing or contour trenching, aimed at slowing down surface runoff and thereby causing more infiltration, which may go as delayed subsurface seepage either to the unconfined or deeper confined aquifer systems.

Moderate topographic slopes between 1:10 and 1:100 usually occur on valley sides, downward of piedmont foothill regions. Surface and subsurface retention of water in these areas will be for longer durations depending upon slope and other conditions. The piedmont zone, with characteristically deep water table is located immediately at the foothills. The surface drainage is generally located above the water table. The se areas are suitable for locating recharge basins and percolation ponds for recharging the water table aquifer. These unconfined aquifers may or may not recharge the deeper aquifers depending upon their hydraulic connectivity. At elevations just below the piedmont zone, artificial recharge through percolation ponds, recharge pits, trenches and recharge basins is normally feasible. In this transition zone, the piezometric heads of deeper aquifers may be initially located below the phreatic surface but at lower elevations, the situation may be the reverse. In the former situation, recharge of deeper aquifers through shafts, gravity inflow wells or injection wells may be feasible if sufficient source water supply is available.

The broad valley floors or the zone of lowest elevation occurring along the major rivers may typically have gentle to very gentle gradients. The movement of both surface and ground water in these areas is sluggish and retention time, in general, is high. These areas are generally categorized as ground water storage zones as all the water moving down the water table gradient converges in this zone. The deeper semi-confined aquifers often contribute water to the unconfined zone through upward leakage due to higher piezometric heads. The need for artificial recharge in such areas may arise only when they are located in low rainfall zones or have adverse hydrogeologic conditions. In such situations, induced recharge of unconfined aquifer along the river channel will be feasible if the river has some flow. Soil Aquifer Treatment (SAT) of treated municipal waste water may also be possible in the vicinity of urban agglomerations.

5.1.5.3 Soil and Land Use Conditions

Soil and land use conditions are of vital importance if artificial recharge through surface spreading methods is contemplated in an area. Various factors such as the depth of soil profile, its texture, mineral composition and organic content control the infiltration capacity of soils. Areas having a thin soil cover are easily drained and permit more infiltration when compared to areas with thick soil cover in the valley zones. Soils having coarser texture due to higher sand-silt fractions have markedly higher infiltration capacity as compared to clay-rich soils, which are poorly permeable. Soils containing minerals, which swell on wetting like montmorillonite etc. and with higher organic matter, are good retainers of moisture necessary for crop growth but impede deeper percolation.

The land use and extent of vegetation also controls the infiltration capacity of soils. Barren valley slopes are poor retainers of water as compared to grass lands and forested tracts, which not only hold water on the surface longer, but also facilitate seepage during the rainy seasons through the root systems. Similarly, ploughed fields facilitate more infiltration as compared to barren fields.

5.1.5.4 Hydrogeological Factors

Hydrogeological conditions of the area are also among important factors in planning artificial recharge schemes. The recharged water moves below the soil zone in moisture fronts through the zone of aeration. The unsaturated flow is governed by the permeability of zone of aeration, which in turn varies with moisture content of the front. Usually, in case of consolidated and semi consolidated rock formations, the subsoil zone passes into weathered strata, which, in turn, passes into unweathered rock. The hydrogeologic properties of the weathered strata are generally much better as compared to the parent rock due to higher porosity and permeability imparted by weathering. The nature of soil, subsoil, weathered mantle, presence of hard pans or impermeable layers govern the process of recharge into the unconfined aquifer. The saturation and movement of ground water within unconfined and all deeper semi-confined and confined aquifers is governed by storativity and hydraulic conductivity of the aquifer material. Aquifers best suited for artificial recharge are those, which absorb large quantities of water and release them whenever required.

The geologic formations encountered in India have been classified into three groups based on their hydrogeologic properties and ground water potential. The broad hydrogeological characteristics of each group and the suitability of artificial recharge methods in each are given in **Tables 5.3**. The geologic formations in the highly mountainous Himalayan Region, except for the Quaternary valley fill deposits have not been covered in this classification on account of the adverse topographic conditions. Site selection criteria and design guidelines of artificial recharge structures mentioned in the tables have been described in the subsequent chapter.

Table 5.3 Suitability of Artificial Recharge Structures for Different Hydrogeological Settings

Group I - Consolidated Formations:

This group covers the hard crystalline igneous and metamorphic rocks, as well as hard massive indurate Pre-Cambrian sedimentary formations. The late Mesozoic, early Tertiary and Deccan and Rajamahal Volcanics, which cover a large area of the country, are also included in this group

Geologic Age	Rock Formation	Rock Types	Hydrogeologic Characteristics	Artificial Recharge Structures Suitable	Remarks
Archaean (4000 to 1500 million years) Pre-Cambrians (1500 to 600 million years) Jurassic Upper cretaceous to Eocene (110 to 60 million years)	Archaean Complex Dharwars Aravallis to equivalent formations. Cuddapahs, Delhi & equivalent systems. Rajmahal traps Deccan traps	(a)Granites Gneisses, Charnokites, Khodalites (b)Schists, Slates Phyllites Granulites (c)Banded Haematite Quartzites (Iron ore series) (a)Consolidated sandstones, shales, Conglomerates (b)Limestones, Dolomites (c)Quartzites, Marbles (d)Intrusive granites & Malani volcanics (a)Basalts, Dolerites (b)Diorites and other acidic derivatives of Basaltic magma.	These formations have negligible to poor primary porosity. Secondary openings like joints, fractures, shears and faults give rise to limited fracture porosity. Weathering and denudation aided by secondary openings and structural weak planes add to the porosity & permeability of rock mass. Solution cavities (Caverns) in carbonate rocks may, at places give rise to large ground water storage/circulation. Ground water circulation is generally limited to 100m depth but if major deep fractures are present, it may occur down to much deeper levels. Storativity value of unconfined aquifer is generally low. Hydraulic conductivity may vary widely depending on fracture incidence. Leaky confined/confined aquifers may be present in layered formations.	1. Percolation tanks 2. *Nalah* Bunds 3. Gully plugs 4. Contour Bund 5. Bench Terracing. 6. Recharge pits and shafts. 7.Gravity recharge wells 8. Induced recharge wells in favourable situations. 9. Ground water Dam (Under ground *Bandhara*) and Fracture sealing cementation. 10. Borehole Blasting & Hydro fracturing. 11. Various combination of above methods as per the site situations.	1. The storage capacity and diffusivity of aquifer being generally restricted; only limited artificial recharge may be accepted through a single structure, which benefits a limited area. More structures, spread over the watershed are required to create significant impact. 2. Injection recharge wells are not considered suitable due to limited intake possible in the deeper aquifers

Group-II: Semi Consolidated formations:

The sedimentary formations ranging in age between the Upper Carboniferous to Tertiary, which though lithified are relatively less consolidated and soft as compared to the consolidated formation have been included in this group. The hydrogeologic characteristics of the group are intermediate between the consolidated and the unconsolidated groups

Geologic Age	Rock Formation	Rocks Types	Hydrogeologic characteristics	Structures suitable for Artificial Recharge	Remarks
Upper Carboni-ferous to Jurassic (275 to 150 Million years)	Gondwana Group Jurassics of Kutch and Rajasthan, Bagh beds, Lametas & Cretaceous of Trichinapalli & Chharat	(a)Boulder pebble bed (b) Sandstones (c)Shales (d) Coal seams (a) Sandstones (b) Calcareous Sst. (c) Shales (d) Quartzites (e) Limestones	Among the sedimentary rocks included in this group, the pebble & gravel beds, sandstones and boulder conglomerates possess moderate primary porosity and hydraulic conductivity, which is governed by texture, sorting, degree of compaction and amount of cementing material. The hydrogeologic potential of limestones is governed by degree of karstification. The shales have poor potential. In the Gondwana group, the Talchir boulder bed, the Barakars, Kamthis and their equivalent formations possess moderately good potential. This group occurs in parts of West Bengal, Bihar, Orissa, Maharashtra and Andhra Pradesh.	1. Percolation Tanks 2. *Nalah* Bunds 3. Gully plug 4. Bench terracing 5. Contour Bund 6. Groundwater dams 7. Stream Modification 8. Recharge Basin, Pits and shafts 9. Gravity recharge wells 10. Induced Recharge	1. Sand-stones form the main rock type having potential for artificial recharge structures.
Eocene to Lower Pleisto-cene (60 to 1 Mill ion years)	Hill Limestone, Murees of Jammu, Rajmundri Sandstone, Subathus, Dagshai and Kasaulis of Shimla hills, Jaintia, Barail, Surma, Tipam, Dupitila and Dihing of Assam , upper, middle & lower Siwaliks of Himalayan Foot Hill Zone, Tertiary Strata of Rajasthan, Kutch, Gujarat, Pondicherry, A.P, Ratnagiri (Maharashtra), Baripada (Orissa), Quilon, Varkalli (Kerala), Cuddalore (Tamil Nadu)	(a) Nummulitic shales & limestones (b) Carbonaceous shale, (c) Sandstones (d) Shales (e) Conglomerates (f) Ferrugeneous sand stones (g) Calcareous sandstones (h) Pebble beds & boulder conglomerate (i) Sands (j) Clays	Tertiary sandstones of Rajasthan, Gujarat, Kutch, Kerala, Tamil Nadu, Andhra Pradesh and Orissa have relatively better hydrogeologic potential. All the semi-consolidated formations in the peninsular areas occur as innumerable small outcrops and do not have wide regional distribution. These are therefore only locally significant. The semi-consolidated group is extensively exposed in the lower and outer Himalaya ranges extending through J & K, H.P, Punjab, Haryana, U.P., Sikkim, West Bengal, Assam and the North Eastern States. The hydrogeologic potential of these formations becomes relevant only when these occur in the valley areas. The Murees, Dagshai, Kasauli, Subathus and lower Siwaliks are relatively hard & compact and have poor potential. The predominant sandstone members of middle Siwaliks lying at higher elevations do not form aquifers. The upper Siwaliks display moderate ground water potential in suitable topographic locations. Similar is the case with Tertiary Sandstones of N.E. States.	**Confined Aquifer** 1. Injection wells in favourable situation.	

Group-III: Unconsolidated Formations

In this group, the youngest geological formations of Pleistocene to Recent age, which are fluviatile or aeolean in origin, which have not been lithified and occur as loose valley fill deposits have been included. Such formations hold good hydrogeologic potential.

Geologic Age	Rock Formation	Rocks Types	Hydrogeologic characteristics	Structures suitable for Artificial Recharge	Remarks
1	2	3	4	5	
Pleisto-cene to Recent (1 Million yrs. To Recent)	(a) Morains of Himalayan Valleys & Ladakh Region. (b) Karewas of Kashmir (c) Bhabhar Tarai and equivalent piedmont deposits of Himalayan foothills. (d) Indo-Ganga-Brahmaputra alluvial plains (e) Narmada, Tapi, Purna alluvial deposits. (f) Alluvial deposits along courses of major peninsular rivers. (g) Coastal Alluvial and mud flats	(a) Mixed boulders, cobbles, sands and silts. (b) Conglomerates, sands, gravels, carbonaceous shales and blue clays (c) Boulder, cobble, pebble beds, gravels, sands, silt and clays (d) Clays & silts, gravels and sands of different textures, lenses of peat & organic matter, carbonate and siliceous concretions (Kankar) (e) Clays, silts, sands and gravels. (f) Clays, silts, sands and gravels. (g) Clays, silts and sands (salt marshes)	The morain deposits occupy valleys and gorges in interior Himalayas. Ground water development is negligible. It will be premature to think of artificial recharge in these areas. Karewas are lacustrine deposits displaying cyclic layers of clayey, silty and coarser deposits with two intervening well-marked boulder beds. Hydraulic connection between deeper and shallower beds is likely to be poor due to horizontality of intervening clayey layers. The Bhabhar piedmont belt contains many productive boulder, cobble, gravel and sand aquifers in fan deposits of major drainage. The surface gradients are high and the water tables deep. The rivers have shallow, broad and flat beds located much above water table. The deeper aquifers of alluvial plains are expected to merge with unconfined zone in Bhabhar region. Tarai belt represents down-slope continuation of Bhabhar aquifers having higher recharge heads. The deeper confined aquifers display artesian and flowing artesian conditions. The area was a marshy malarial tract due to shallow water table of unconfined aquifer. The Indo-Ganga-Brahmaputra alluvial plains form the most potential ground water reservoir with a thick sequence of sandy aquifers down to great depth. The unconfined sand aquifers have been known to extend down to moderate depth (125m). Within such depths, the aquifers locally behave like confined zones and could regionally form part of an unconfined system. Deeper aquifers below the regionally extensive clayey layers are leaky confined/confined. The texture of sand strata, degree of sorting and uniformity and compaction determines the Storativity and hydraulic conductivity of individual stratum. The older alluvium, occurring away from the present river channels, and strata below 400 m. depth are more compact and hence permeability is relatively less.	1. Flooding 2. Ditch & Furrow 3. Contour Trenches 4. Recharge Basin 5. Stream Modification 6. Surface irrigation 7. Injection well 8. Connector well 9. Recharge pits & shafts 10. Induced recharge. .	1. The valleys and gorges in interior and outer Himalayas have not been fully explored and exploited for ground water resources and thus any scheme for artificial in these areas is not suggested at this stage. 2. Bhabhar region, being the recharge zone for most of the deeper aquifer systems in alluvial plains, offer possibilities of augmenting ground water reservoir by construction of contour trenches recharge basins and pits. Stream flow, available for a very limited time during monsoon period requires to be fully utilized for recharge of deeper aquifer. 3. Tarai belt being a natural discharge zone in the foothill region is presently not conducive for any artificial recharge. Sluice valve control of artesian wells is required to conserve groundwater outflow from deeper aquifers

Geologic Age	Rock Formation	Rocks Types	Hydrogeologic characteristics	Structures suitable for Artificial Recharge	Remarks
1	2	3	4	5	
	(h) Aeolean Deposits of Western Rajasthan and parts of neighbouring states.	(h) Very fine to fine sands and silts.	The unconfined aquifers generally show high Storativity (5 to 25%) and high Transmissivity (500 to 3000 m^2/day) and have great capacity to accept and store recharged water. The leaky-confined aquifers receive recharge in areas where unconfined aquifers have higher hydraulic heads (tracts along major canals) and provide leakage recharge to the unconfined aquifer wherever the relative heads are reverse (mostly along courses of major streams). The deeper confined aquifers generally occurring below 200 to 300 m depth have low Storativity (0.005 to 0.0005) and high Transmissivity (300 to 1000 m^2/day). The alluvial valley fill deposits of Narmada, Tapi and Purna fault basins are predominantly silty/clayey with a few sand-gravel lenses within 100 m depth. Deeper strata are more clayey and are perhaps partly Pleistocene/tertiary. The quality of ground water at deeper levels is inferior. The aquifers have moderate ground water potential (Storativity 4×10^{-6} to 1.6×10^{-2} and Transmissivity 100 to 1000 m^2/day). The aeolean deposits (sand dunes) of western Rajasthan and parts of Haryana, Delhi and Punjab are very fine to fine grained, well-sorted sands and silts. Due to their location in arid region, they do not receive adequate natural recharge and water table is normally deep. The coastal sands and mud flats are generally restricted in width and thickness and do not merit detailed consideration.		4. In alluvial plains, canal irrigation over extensive tracts have given rise to incidental recharge of aquifers in most of the States, which forms the best supplementary recharge, provided the adverse effects like water-logging and salinisation of land are avoided through proper irrigation practices. 5. In aeolean deposits (sand dunes) of western Rajasthan, and parts of Haryana, Delhi and Punjab, unintended recharge may form the most appropriate option if canal water transferred from other basins becomes available.

5.2 Investigations for Proper Planning

Various inputs are necessary for proper and scientific planning of artificial recharge schemes in any terrain. Scientific investigations leading to a better understanding of the characteristics of sub-surface formations are to be taken up for realistic determination of these inputs. These can broadly be grouped into two categories namely *viz.* general studies and detailed studies.

5.2.1 General Studies

These studies are aimed at assessing the need and scope of artificial recharge in an area. The procedure to be followed for establishing the need for artificial recharge in an area to augment the ground water resources has already been described in detail in an earlier section of the chapter.

Once a case of overexploitation of ground water is proved, the need for augmentation of ground water resources through artificial recharge is justified. In case of entire watersheds, overlaying of maps depicting the long-term decline in water levels and cumulative departure of rainfall from the normal can help in identification of areas requiring recharge augmentation.

Once the areas requiring artificial recharge are identified, the next step is to decide on the appropriate techniques for recharging the aquifer. The synthesis of all available data relevant to ground water is the first step in this exercise. These data include a) all sources of recharge like rivers, tanks, canals etc., b) rainfall distribution pattern, c) hydrogeological parameters with emphasis on lithological characteristics, d) nature of the terrain, e) intensity of ground water development and irrigation practices and f) chemical quality of surface and ground water etc. The data is generally available in reports/records of various Central and State Government agencies. However, the data available often have considerable gaps. It is therefore necessary to have detailed studies to supplement the available data and for preparation of a scientific data base for proper implementation of suitable artificial recharge schemes.

5.2.2 Detailed Studies

Once the need for and suitability of the area for artificial recharge to ground water are identified on the basis of data collected from the general studies, areas identified as suitable for recharge augmentation are studied in detail using Remote Sensing techniques and through hydrometeorological, hydrological, geophysical, hydrogeological and hydrochemical investigations to ascertain the scope and feasibility of artificial recharge. These studies are to be oriented in such a way as to collect and analyse necessary data, which are to be used as inputs for proper planning of artificial recharge projects. The major inputs expected to be provided by the studies mentioned are given below (**Table 5.4)**

Table.5.4 Details of Studies Required for Planning Artificial Recharge Schemes

Sl.No	Type of Study	Inputs Anticipated
1	Remote Sensing Studies	Spatial variation in the infiltration characteristics of various litho-units. Drainage characteristics and Lineament intensity. Distribution of various geomorphic units.
2	Hydrometeorological Studies	Rainfall amount, duration, daily and hourly rainfall intensity, variability of rainfall.
3	Hydrological Studies	Source water availability, infiltration characteristics of major soil types and various land use categories
4	Geophysical Studies	Thickness of weathered zone in hard rocks Thickness and characteristics of granular zones in sedimentary terrain. Stratification of aquifer system and spatial variability in hydraulic conductivity. Vertical hydraulic conductivity Discontinuities such as dykes and fault zones.
5	Hydrogeological Studies	Regional hydrogeology and aquifer characteristics Behaviour of ground water levels Ground water potential Ground water flow pattern and hydraulic connection between ground water and surface water bodies.
6	Hydrochemical Studies	Quality aspects of source water for artificial recharge. Spatial and temporal variations in ground water quality.

5.2.2.1 Remote Sensing Studies

Remote sensing, with its advantages of spatial, spectral and temporal availability of data has now become a very useful tool in assessing, monitoring and conserving ground water resources. Satellite data provides quick and useful baseline information on various parameters controlling the occurrence and movement of ground water such as geology, structural features, geomorphology, soils, land use, land cover, lineaments etc. All these parameters used to be studied earlier independently due to non-availability of data and lack of integrating tools and modeling techniques. A systematic study of these factors leads to better delineation of areas suitable for artificial recharge, which are then studied in detail through hydrogeological and geophysical investigations.

Visual interpretation of Satellite Imagery, with emphasis on terrain analysis is being used widely for selection of sites suitable for recharge augmentation. Aspects, which are given special attention for the study, usually carried out with Satellite Imagery or False Colour Composites (FCC) on 1: 50,000 scale include stream course delineation, land form analysis, outcrop pattern analysis, fracture pattern analysis and land use analysis. These studies can provide valuable information on drainage density and lineament intensity, which helps in the identification of suitable sites for recharge. Various geomorphic units can also be delineated, which also help determine the type of recharge structures suitable for the area.

Apart from visual interpretation, digital image enhancement techniques are also being increasingly used for deriving geological, structural and geomorphological information. Digital Image Enhancement techniques are found to be extremely useful as they improve the feature sharpness and contrast for simple interpretation. Various thematic layers generated using remote sensing data such as lithology, structure, geomorphology, land use/land cover, lineaments etc. can be integrated with slope, drainage density and other collateral data in a Geographic Information System (GIS) framework and analysed using a model developed with logical conditions to arrive at suitable sites for artificial recharge.

Image rectification and preparation of a GIS file through visual interpretation of standard False Color Composite (FCC) data can be done to extract expressions of sub-surface moisture conditions. Techniques such as Edge Enhancement are Band Rationing are useful techniques for digital image interpretation.

Observations from satellite data must be complemented by field checks, existing geologic maps and topographic sheets.

5.2.2.2 Hydrometeorological Studies

Rainfall and evaporation are two of the most important parameters, which are required for proper planning of artificial recharge schemes.

Detailed information pertaining to the amount, duration and intensity of rainfall in a given area is a necessary pre-requisite for planning recharge schemes. Rainfall data is normally available at offices of India Meteorological Department (IMD), Revenue Offices such as Collectorates, Taluk/Block/Mandal offices, Irrigation project dam sites and Agricultural Universities/ Colleges etc.

Long-term average rainfall is an important parameter for assessing the storage capacity of various artificial recharge structures. On the other hand, daily and hourly rainfall data is essential for planning water conservation schemes such as farm ponds, contour trenches, roof top rainwater harvesting schemes and also for designing filters for runoff recharge schemes.

Long-term average rainfall, dependable average rainfall and probability of incidence of a particular amount of rainfall in a given area can be calculated using long-term rainfall data of IMD Stations for 100 to 150 years. For computations of daily and hourly rainfall intensity, data available with other agencies can be used.

Evaporation data is useful for assessing the potential losses from the free surfaces of ponds and other surface water storage structures. Data related to daily/seasonal/ monthly evaporation losses is helpful for identification of most effective recharge schemes in an area. The period/duration of ground water recharge with minimum evaporation losses can be determined from this data.

5.2.2.3 Hydrological Studies

Hydrological investigations are useful for ascertaining the availability of source water for recharge. These investigations are required to be carried out in the watershed, sub-basin or basin where the artificial recharge schemes are envisaged.

A detailed account of the hydrological investigations for artificial recharge schemes have already been discussed under the heading 'Source Water' earlier in the manual.

5.2.2.4 Geophysical Studies

Geophysical studies can provide useful information pertaining to the characteristics of sub-surface lithological formations, which influence the type of recharge mechanism suitable for a particular area. These studies are normally taken up to complement the data collected through hydrogeological investigations.

The main purpose of applying geophysical methods for the selection of appropriate sites for artificial recharge studies is to assess the unknown sub-surface hydrogeological conditions economically, adequately and unambiguously. They are usually employed to narrow down the target zone and to pinpoint the probable sites for artificial recharge structures. The application of geophysical techniques is also useful for bringing out a comparative picture of the sub-surface litho-environment and to correlate them with the hydrogeological setting. Besides defining the sub-surface structure and lithology, geophysical studies can also help in studies for identifying the brackish/fresh ground water interface, contaminated zones (saline) and area prone to seawater intrusion.

In the context of artificial recharge, Geophysical studies are particularly useful for gathering information pertaining to

i. Stratification of aquifer systems and spatial variability of hydraulic conductivity of different zones.
ii. Negative or non-productive zones of low hydraulic conductivity in unsaturated and saturated zones.
iii. Vertical hydraulic conductivity discontinuities such as dykes, faults etc.
iv. Moisture movement and infiltration.
v. Direction of ground water flow under natural/artificial recharge processes.
vi. Salinity changes in aquifers with depth / saline water ingress.

Surface Geophysical techniques such as Electrical Resistivity Surveys, Self Potential (SP) surveys, Very Low Frequency (VLF) Electromagnetic Surveys and Shallow Refraction Seismic Surveys are commonly used for identification of sites for artificial recharge structures. Physical parameters like rock resistivities, magnetic susceptibilities, shock wave velocities etc. are measured in these investigations and interpreted to gather information pertaining to sub-surface rock types, rock water content, structural controls on ground water movement and ground water salinity. Subsurface methods such as Spontaneous Potential, Neutron, Natural Gamma, and

Closed Circuit Television (CCTV) logging techniques are also useful for collecting valuable information from boreholes in the study area. As compared to surface methods, which measure parameter values representative of a combined subsurface layer sequence, subsurface methods measure the value of the physical parameter concerned for each individual layer.

5.2.2.5 Hydrogeological Studies

A detailed understanding of the hydrogeology of the area is of prime importance in ensuring successful implementation of any artificial recharge scheme. A desirable first step toward achieving this objective is to synthesize all available data on various hydrogeological parameters from different agencies. Regional geological maps indicate the location of different geological strata, their geological age sequence, boundaries/contacts of individual formations and structural expressions like strike, dip, faults, folds, fractures, intrusive bodies etc. These maps also indicate the correlation of topography and drainage to geological contacts.

Maps providing information on regional hydrogeological units, their ground water potential and general pattern of ground water flow and chemical quality of ground water in different aquifers are also necessary. Satellite imagery provide useful data on geomorphic units and lineaments, which govern the occurrence and movement of ground water, especially in hard rock terrain. A detailed hydrogeological study, aimed at supplementing the regional picture of hydrogeological set up available from previous studies, is imperative to have precise information about the promising hydrogeological units for recharge and to decide on the location and type of structures to be constructed.

5.2.2.5.1 Detailed Hydrogeological Mapping: The purpose of detailed hydrogeological mapping is to prepare the following maps, which facilitate an understanding of the ground water regime and its suitability to artificial recharge schemes

- i) Map showing the hydrogeological units demarcated on the basis of their water-bearing capabilities, both at shallow and deep levels.
- ii) Map showing ground water elevation contours to determine the form of the water table and the hydraulic connection between ground water and surface water bodies like rivers, tanks and canals.
- iii) Maps showing depths to water table, usually compiled for the periods of maximum, minimum and mean annual positions of water table.
- iv) Maps showing amplitudes of ground water level fluctuation.
- v) Maps showing piezometric heads of aquifers and their variations with time.
- vi) Maps showing ground water potentials of different hydrogeological units and the levels of ground water development.
- vii) Maps showing chemical quality of ground water in different aquifers.

The usage of the above interpretative maps is additive, i.e., their combined usage provides greater knowledge and understanding of an area than when a map is used in isolation. The maps mentioned above will help determine

a) whether any gaps exists in the data on sub-surface geology of the area
b) whether the available data on aquifer parameters is sufficient in case the area shows promise for artificial recharge to the deeper aquifers.
c) Whether the available ground water structures are sufficient to monitor the impacts of artificial recharge to ground water.

5.2.2.5.2 Aquifer Geometry: The data on sub-surface hydrogeological units and their thickness and depth of occurrence are necessary to bring out the disposition and hydraulic properties of the unconfined, semi-confined and confined aquifers in the area. For surface water spreading techniques, the area of interest is generally restricted to shallow depths. The main stress is on knowing whether the surface rock types are sufficiently permeable to maintain high rate of infiltration during artificial recharge.

5.2.2.6 Hydrochemical Studies

A detailed study of the quality of source water is vitally important whenever direct recharge techniques are contemplated. In cases where *in situ* precipitation or water supplied from canals are used for recharge, no constraints on account of water quality may arise. However, in cases where waters in the lower reaches of rivers or recycled municipal/industrial waste waters are proposed to be used, the quality of water requires to be precisely analysed and monitored to determine the type and extent of treatment required.

5.3 Appraisal of Economic Viability

Economic viability is another critical parameter to be ascertained before taking a decision to implement any artificial recharge scheme. The appraisal of economic viability has to be carried out after taking into account all possible expenses including those for investigation, source water (conveyance, treatment), construction of recharge structures, operation and maintenance etc. All benefits should be appropriately accounted for and assessed in order to decide the acceptability of the scheme as per its priority in the overall scheme of development.

Important guidelines for carrying out economic appraisal of ground water recharge projects are furnished below:

i. The inputs and outputs should be distinguished as ‘tradables’ and ‘non-tradables’.
ii. It is to be assumed that the project under consideration will not change the price of the output.
iii. Certain adjustments have to be made for converting financial prices to economic prices by applying appropriate conversion factors.
iv. The economic analysis should consider the effects of the project on both the producer and the user.
v. Labour and wages under skilled and unskilled categories have a special significance in the valuation for economic analysis. The real contribution to the economy probably varies according to region, type of labour and season. Hence, an extensive labour market survey is required for proper restructuring of the analysis.

vi. Although the computational part of the appraisal is rather straightforward, the essential purpose of the exercise is to ensure that the project has a positive impact on the efficient application of the resources of the nation.

vii. The outcome of the economic appraisal of a development project is decisive for the acceptance of the project.

viii. If the project is acceptable from the economic but not from the financial point of view, it implies that the project will contribute to an efficient application of the resources, but with additional financial support.

ix. If the project yields attractive returns to the Government but does not make a contribution to the efficient use of national resources, additional policy measures may be required to rectify the situation.

It is important to carry out the benefit cost analysis for all major public works before deciding the allocation of funds. The benefit cost analysis presents the quantifiable efforts and environmental and social aspects of any public projects in terms of money. Hence, it is an important instrument to guide investments for better planning and designing of the proposed layout.

The analysis of the financial benefits and costs requires the expression of cash flow elements under the non-financial operations in comparable terms. Costs are related to investments occurring during the lifetime of the project. Benefits, on the other hand, originate from the productive use of the projects. Both costs and benefits are, therefore, expressed in quantitative terms and translated into monetary terms by using market values of the inputs and outputs concerned. As the costs and benefits occur at different points of time, it is customary to express both in terms of their present value by applying appropriate discounting factors to make them comparable. After accounting for both costs and benefits against their market values, appropriate criteria are applied to determine the profitability of the project.

The benefit cost analysis of projects, also called Project Appraisal is done before the decision is taken to invest. The Project Appraisal includes financial, economic and social Benefit Cost analysis. The economic evaluation of the project, on the other hand, is done to analyse the performance and effects of the project after it has been executed.

The computational details of benefit cost analysis of artificial recharge projects are described in detail in the chapter on ‘Economic Evaluation of Recharge Projects’ in this manual.

5.4 Finalisation of Physical Plan

The finalization of physical plan for artificial recharge involves the following steps

i. Preparation of lay-out plan of the project area on an appropriate scale showing the locations of proposed structures and source water conveyance systems.
ii. Determination of the number of structures required for recharge.
iii. Identification of tentative locations of proposed structures
iv. Preparations of design specifications and drawings

v. Working out the time-schedules for completion of various stages of the scheme.
vi. Planning of financial aspects such as source of funds, allocations required at various stages, schedules of repayment etc
vii. Identification of the agency for executing the scheme.

5.5 Preparation of Report of the Scheme

Reports are to be prepared separately for each scheme or project, reflecting various considerations made during the planning process. The Detailed Report should essentially cover the following aspects;

- Detailed background, purpose, scope, technical feasibility and objectives of the scheme.
- All physical details of the work including layout plans, drawings, specifications of structures and materials etc.
- An execution plan, indicating various phases of the work, work and time schedules and agency-wise allocation of responsibilities.
- Financial allocations, mode of recoveries, repayment schedules etc.
- Details of monitoring systems and their operation.

6. ARTIFICIAL RECHARGE TECHNIQUES AND DESIGNS

The selection of a suitable technique for artificial recharge of ground water depends on various factors. They include:

a) Quantum of non-committed surface run-off available.
b) Rainfall pattern
c) Land use and vegetation
c) Topography and terrain profile
d) Soil type and soil depth
e) Thickness of weathered / granular zones
f) Hydrological and hydrogeological characteristics
g) Socio-economic conditions and infrastructural facilities available
h) Environmental and ecological impacts of artificial recharge scheme proposed.

6.1 Artificial Recharge Techniques

Techniques used for artificial recharge to ground water broadly fall under the following categories

I) Direct Methods

A) Surface Spreading Techniques
- a) Flooding
- b) Ditch and Furrows
- c) Recharge Basins
- d) Runoff Conservation Structures
  - i) Bench Terracing
  - ii) Contour Bunds and Contour Trenches
  - iii) Gully Plugs, *Nalah* Bunds, Check Dams
  - iv) Percolation Ponds
- e) Stream Modification / Augmentation

B) Sub-surface Techniques
- a) Injection Wells (Recharge Wells)
- b) Gravity Head Recharge Wells
- c) Recharge Pits and Shafts

II) Indirect Methods

- A) Induced Recharge from Surface Water Sources;
- B) Aquifer Modification
  - i) Bore Blasting.
  - ii) Hydro-fracturing.

III) Combination Methods

In addition to the above, ground water conservation structures like Subsurface dykes (*Bandharas*) and Fracture Sealing Cementation techniques are also used to arrest subsurface flows.

Aquifer disposition plays a decisive role in choosing the appropriate technique of artificial recharge of ground water (Todd and Mays, 2005) as illustrated in the **Fig. 6.1.**

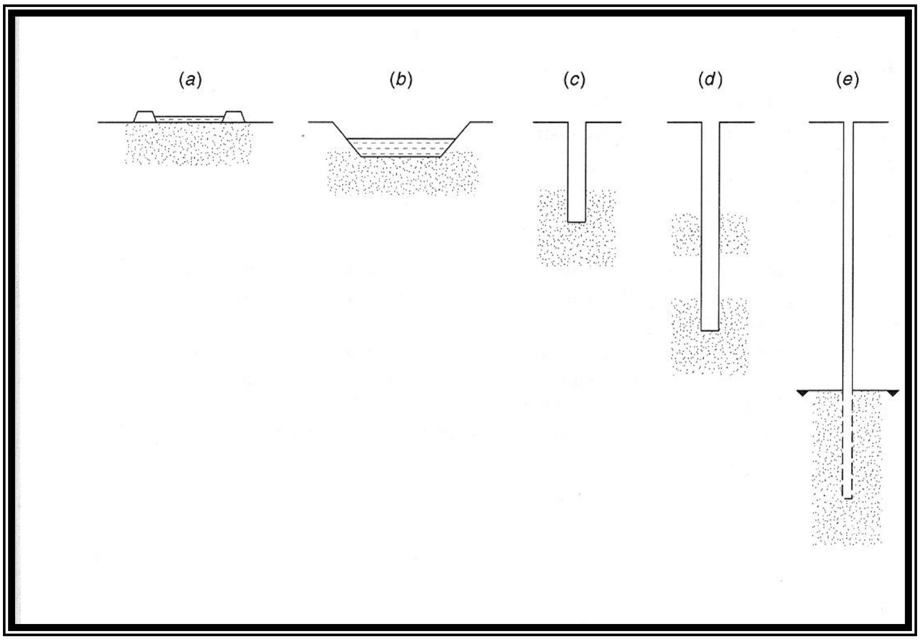


Figure 6.1 Recharge Systems for Increasingly Deep permeable materials: Surface Basin (a), Excavated Basin (b), Trench (c), Shaft or Vadose Zone Well (d) and Aquifer Well (e)

6.2 Direct Methods

6.2.1 Surface Spreading Techniques

These are aimed at increasing the contact area and residence time of surface water over the soil to enhance the infiltration and to augment the ground water storage in phreatic aquifers. The downward movement of water is governed by a host of factors including vertical permeability of the soil, presence of grass or entrapped air in the soil zone and the presence or absence of limiting layers of low vertical permeability at depth. Changes brought about by physical, chemical and bacteriological influences during the process of infiltration are also important in this regard.
Important considerations in the selection of sites for artificial recharge through surface spreading techniques include

i) The area should have gently sloping land without gullies or ridges.
ii) The aquifer being recharged should be unconfined, permeable and sufficiently thick to provide storage space.
iii) The surface soil should be permeable and have high infiltration rate.
iv) Vadose zone should be permeable and free from clay lenses.
v) Ground water levels in the phreatic zone should be deep enough to accommodate the recharged water so that there is no water logging.
vi) The aquifer material should have moderate hydraulic conductivity so that the recharged water is retained for sufficiently long periods in the aquifer and can be used when needed.

The most common surface spreading techniques used for artificial recharge to ground water are flooding, ditch and furrows and recharge basins.

6.2.1.1 Flooding

This technique is ideal for lands adjoining rivers or irrigation canals in which water levels remain deep even after monsoons and where sufficient non-committed surface water supplies are available. The schematics of a typical flooding system are shown in **Fig.6.2.** To ensure proper contact time and water spread, embankments are provided on two sides to guide the unutilized surface water to a return canal to carry the excess water to the stream or canal.

Flooding method helps reduce the evaporation losses from the surface water system, is the least expensive of all artificial recharge methods available and has very low maintenance costs

6.2.1.2 Ditch and Furrows method

This method involves construction of shallow, flat-bottomed and closely spaced ditches or furrows to provide maximum water contact area for recharge from source stream or canal. The ditches should have adequate slope to maintain flow velocity and minimum deposition of sediments. The widths of the ditches are typically in the range of 0.30 to 1.80 m. A collecting channel to convey the excess water back to the source stream or canal should also be provided. A typical system is shown in **Fig. 6.3(a)** and three common patterns *viz.* lateral ditch pattern, dendritic pattern and contour pattern are shown in **Fig.6.3 (b).** Though this technique involves less soil preparation when compared to recharge basins and is less sensitive to silting, the water contact area seldom exceeds 10 percent of the total recharge area.

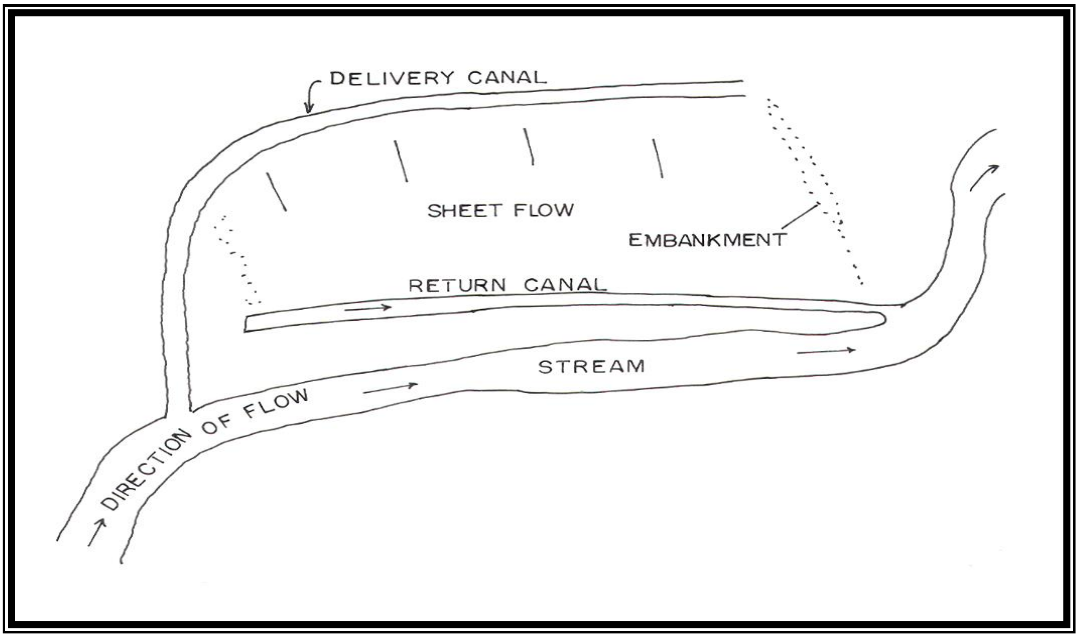


Fig.6.2 Schematics of a Typical Flood Recharge System

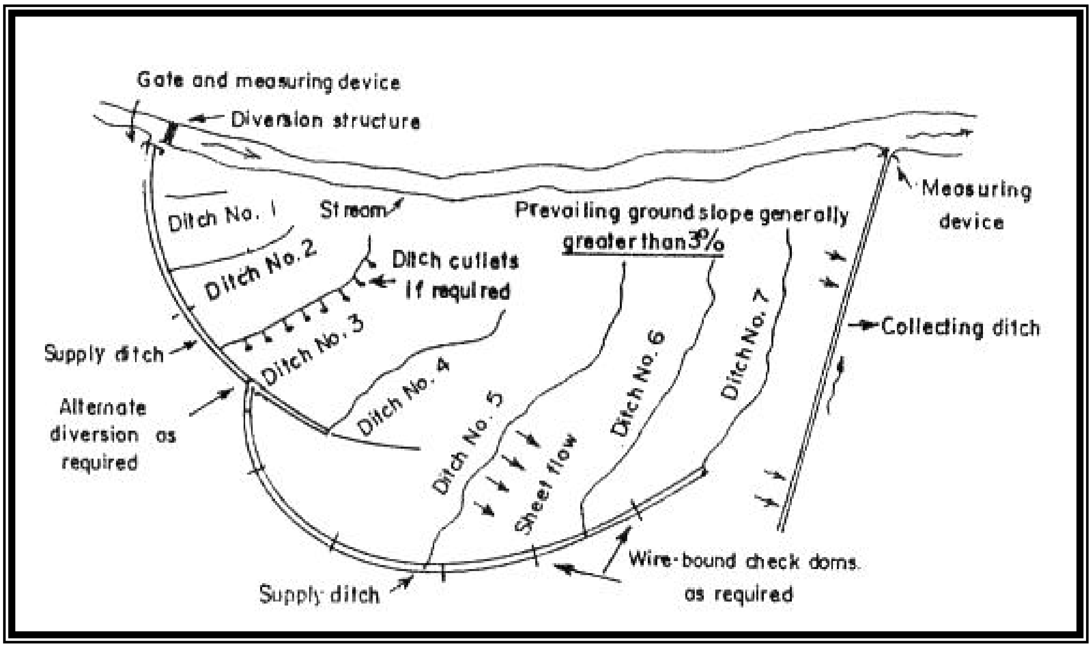


Fig 6.3(a) Schematics of a Typical Ditch and Furrows Recharge System

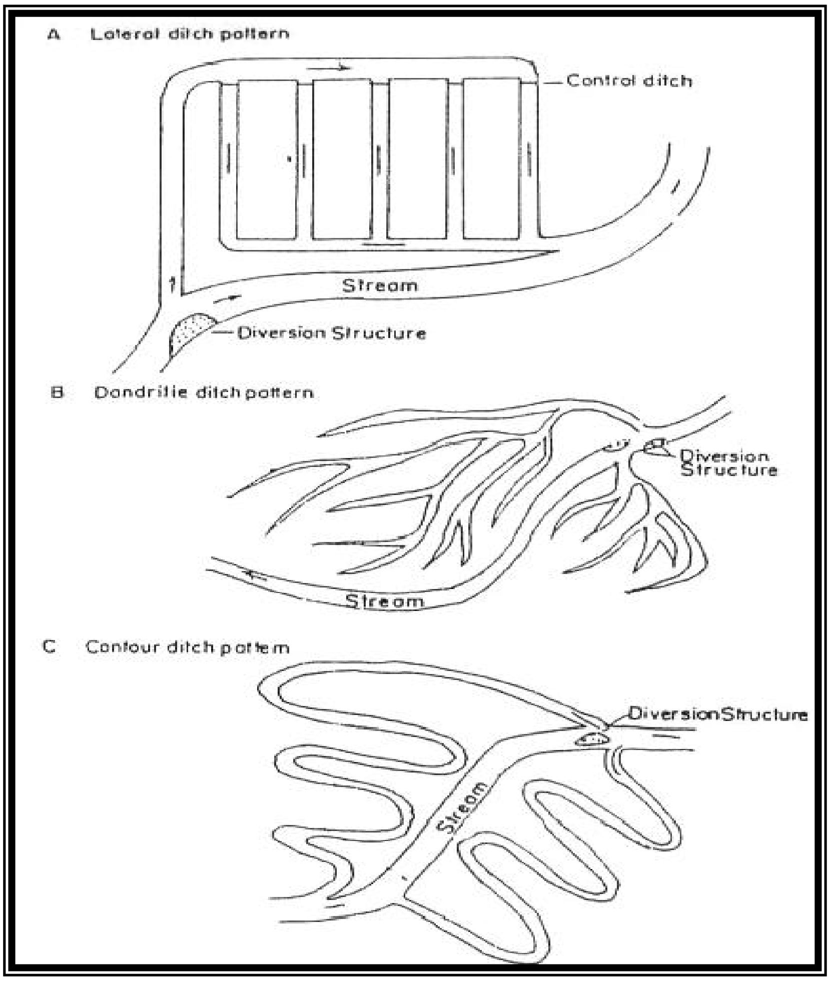


Fig 6.3(b) Common Patterns of Ditch and Furrow Recharge Systems.

6.2.1.3 Recharge Basins

Artificial recharge basins are commonly constructed parallel to ephemeral or intermittent stream channels and are either excavated or are enclosed by dykes and levees. They can also be constructed parallel to canals or surface water sources. In alluvial areas, multiple recharge basins can be constructed parallel to the streams **(Fig.6.4)**, with a view to a) increase the water contact time, b) reduce suspended material as water flows from one basin to another and c) to facilitate periodic maintenance such as scraping of silt etc. to restore the infiltration rates by bypassing the basin under restoration.

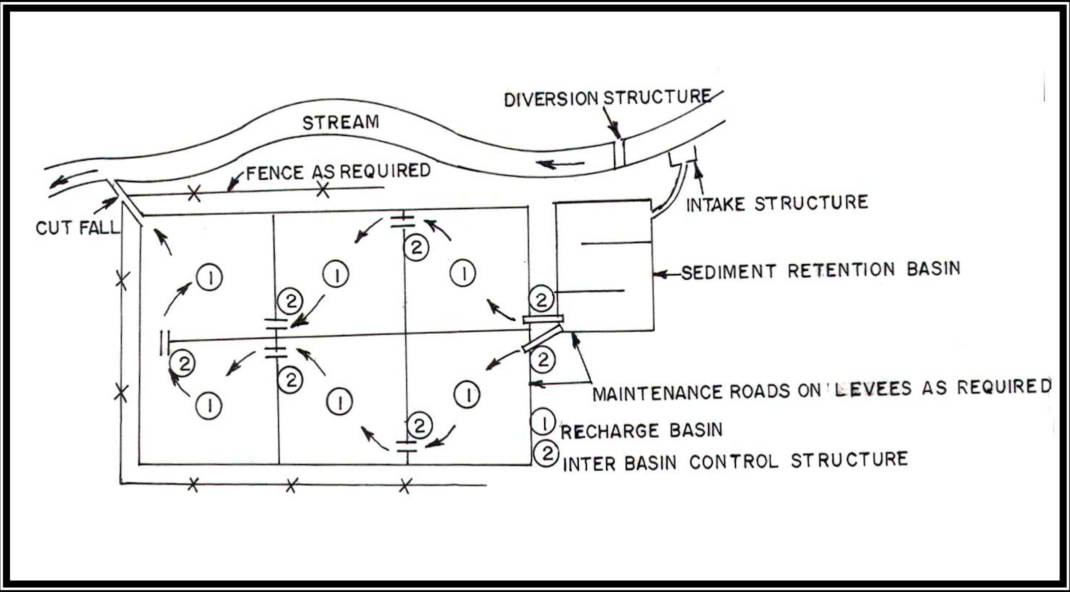


Fig 6.4 Schematics of a Typical Recharge Basin

In addition to the general design guidelines mentioned, other factors to be considered while constructing recharge basins include

a) area selected for recharge should have gentle ground slope.
b) the entry and exit points for water should be diagonally opposite to facilitate adequate water circulation in individual basins,
c) water released into the basins should be as sediment – free as possible and
d) rate of inflow into the basin should be slightly more than the infiltration capacity of all the basins.

6.2.2 Runoff Conservation Structures

These are normally multi-purpose measures, mutually complementary and conducive to soil and water conservation, afforestation and increased agricultural productivity. They are suitable in areas receiving low to moderate rainfall mostly during a single monsoon season and having little or no scope for transfer of water from other areas. Different measures applicable to runoff zone, recharge zone and discharge zone are available. The structures commonly used are bench terracing, contour bunds, gully plugs, *nalah* bunds, check dams and percolation ponds.

6.2.2.1 Bench Terracing

Bench terracing involves leveling of sloping lands with surface gradients up to 8 percent and having adequate soil cover for bringing them under irrigation. It helps in soil conservation and holding runoff water on the terraced area for longer durations, leading to increased infiltration and ground water recharge.

For implementing terracing, a map of the watershed should be prepared by level surveying and suitable benchmarks fixed. A contour map of 0.3 m contour interval is then prepared. Depending on the land slope, the width of individual terrace should be determined, which, in no case, should be less than 12 m. The upland slope between two terraces should not be more than 1:10 and the terraces should be leveled. The vertical elevation difference and width of terraces are controlled by the land slope. The soil and weathered rock thickness required, vertical elevation difference and the distance between the bunds of two terraces for different slope categories are furnished in **Table.6.1.**

In cases where there is a possibility of diverting surface runoff from local drainage for irrigation, as required in case of paddy cultivation in high rainfall areas, outlet channels of adequate dimensions are to be provided. The dimensions of the outlet channels depend on the watershed area as shown below in **Table6.2.** The terraces should also be provided with bunds of adequate dimensions depending on the type of soils as shown in **Table.6.3**

Table 6.1 Soil and Weathered Rock Thickness, Vertical Elevation Difference and the Distance between the Bunds of Two Terraces for Different Slope Categories

Land Slope (%)	Required Thickness of Soil and Weathered Rock (m)	Vertical Separation (m)	Distance Between Bunds of Two Terraces (m)
1	0.30	0.30	30
2	0.375	0.45	22
3	0.450	0.60	20
4	0.525	0.75	18.75
5	0.600	0.90	18
6	0.750	1.05	17.5
7	0.750	1.20	17
8	0.750	1.20	15

Table: 6.2 Dimensions of Output Channels for Different Watershed Areas

Area of watershed (ha)	Channel Dimensions (m)		
	Base Width	Top width	Depth
< 4	0.30	0.90	0.60
4 to 6	0.60	1.20	0.60
6 to 8	0.90	1.50	0.60
8 to 10	1.20	1.80	0.60
10 to 12	1.50	2.10	0.60

Table: 6.3 Dimensions of Terraces in Different Soil Types

Type of Soil	Soil Thickness (cm)	Base Width (m)	Top Width (m)	Height (m)	Side slope
Light	7.50 to 22.50	1.50	0.30	0.60	1:1
Medium	22.50 to 45.00	1.80	0.45	0.65	1:1
Medium Deep	45.00 to 90.00	2.25	0.45	0.75	1:1
Deep	> 90.00	2.50	0.50	0.80	1:1

In areas where paddy is cultivated, water outlets of adequate dimensions are to be provided to drain out excess accumulated water and to maintain water circulation. The width of the outlets may vary from 0.60 m for watersheds up to 2 ha to 3.0 m for watersheds of up to 8 ha for rainfall intensity between 7.5 and 10 cm. All the outlets should be connected to natural drainage channels.

6.2.2.2 Contour Bunds

Contour bunding, which is a watershed management practice aimed at building up soil moisture storage involve construction of small embankments or bunds across the slope of the land. They derive their names from the construction of bunds along contours of equal land elevation. This technique is generally adopted in low rainfall areas (normally less than 800 mm) where gently sloping agricultural lands with very long slope lengths are available and the soils are permeable. They are not recommended for soils with poor internal drainage e.g. clayey soils. Schematic of a typical system of contour bunds is shown in **Fig.6.5.**

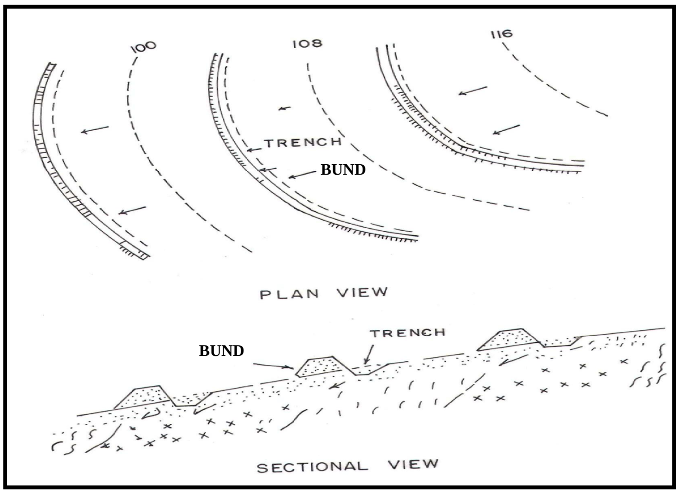


Fig.6.5 Schematics of a Typical Contour Bund

Contour bunding involves construction of narrow-based trapezoidal embankments (bunds) along contours to impound water behind them, which infiltrates into the soil and ultimately augment ground water recharge.

Field activities required prior to contour bunding include levelling of land by removing local ridges and depressions, preparation of map of the area through level surveying and fixing of bench marks. Elevation contours, preferably of 0.3 m interval are then drawn, leaving out areas not requiring bunding such as habitations, drainage etc. The alignment of bunds should then be marked on the map.

The important design aspects of contour bunds are i) spacing, ii) cross section and iii) deviation freedom to go higher or lower than the contour bund elevation for better alignment on undulating land.

6.2.2.2.1 Spacing of Bunds: Spacing of contour bund is commonly expressed in terms of vertical interval (V.I), which is defined as the difference in elevation between two similar points on two consecutive bunds. The main criterion for spacing of bunds is to intercept the water before it attains the erosive velocity. Spacing depends on slope, soil, rainfall, cropping pattern and conservation practices.

Spacing of contour bunds is normally calculated using the formula

Vertical Interval (V.I) = 0.305 (XS+Y), where

X is the rainfall factor,
S is the land slope (%) and
Y is the factor based on soil infiltration and crop cover during the erosive period of rains

The rainfall factor 'X' is taken as 0.80 for scanty rainfall regions with annual rainfall below 625 mm, as 0.60 for moderate rainfall regions with annual rainfall in the range of 625 to 875 mm and as 0.40 for areas receiving annual rainfall in excess of 875 mm. The factor 'Y' is taken as 1.0 for soils having poor infiltration with low crop cover during erosive rains and as 2.0 for soils of medium to good infiltration and good crop cover during erosive rains. When only one of these factors is favourable, the value of Y is taken as 1.50. Vertical spacing can be increased by 10 percent or 15 cm to provide better location, alignment or to avoid obstacles.

The horizontal interval between two bunds is calculated using the formula

Horizontal Interval (H.I) = V.I x 100/Slope

6.2.2.2.2 Cross Section of Contour Bunds: A trapezoidal cross section is usually adopted for the bund. The design of the cross section involves determination of height, top width, side slopes and bottom width of the bund.

The height of the bund depends on the slope of the land, spacing of the bunds and the rainfall excess expected in 24-hour period for 10-year frequency in the area. Once the height is determined, other dimensions can be worked out depending on the nature of the soil.

Height of the bund can be determined by the following methods

a) Arbitrary Design: The depth of impounding is designed as 30 cm. 30 cm is provided as depth flow over the crust of the outlet weir and 20 cm is provided as free board. The overall height of the bund in this case will be 80 cm. With top width of 0.50 m and base width of 2 m, the side slope will be 1:1 and the cross section, 1 sq m.

b) The height of bund to impound runoff from 24 hour rain storm for a given frequency can be calculated by the formula

$$H = \frac{\sqrt{Re \times V.I}}{50}, \quad \text{where}$$

H is the depth of impounding behind the bund (m),
Re is the 24 hour rainfall excess (cm) and
VI is the vertical interval (m)

To the height so computed, 20 percent extra height or a minimum of 15cm is added for free board and another 15 to 20 percent extra height is added to compensate for the settlement due to consolidation.

Top width of the bund is normally kept as 0.3 to 0.6 m to facilitate planting of grasses. Side slopes of the bund are dependent on the angle of repose of the soil in the area and commonly range from 1:1 for clayey soils to 2:1 for sandy soils. Base width of the bund depends on the hydraulic gradient of the water in the bund material due to the impounding water. A general value of hydraulic gradient adopted is 4:1. The base should be sufficiently wide so that the seepage line should not appear above the toe on the downstream side of the bund.

Size of the bund is expressed in terms of its cross-sectional area. The cross sectional area of bunds depends on the soil type and rainfall and may vary from 0.50 to 1.0 sq m in different regions. Recommended contour bund specifications for different soil depths are shown in **Table: 6.4**

Table 6.4 Recommended Contour Bund Specifications for Different Soil Depths

Soil Type	**Soil Depth (m)**	**Top Width (m)**	**Bottom Width (m)**	**Height (m)**	**Side Slope**	**Area of Cross section (sq m)**
Very Shallow Soils	< 7.5	0.45	1.95	0.75	1:1	0.09
Shallow Soils	7.50 to 23 .0	0.45	2.55	0.83	1.25:1	1.21
Medium Soils	23.0 to 45.0	0.53	3.00	0.83	1.50:1	1.48
Deep soils	45.0 to 80.0	0.60	4.20	0.90	2:1	2.22

The length of bunds per hectare of land is denoted by the Bunding Intensity, which can be computed as

$$\text{Bunding Intensity} = \frac{100\ S}{V.I}, \text{ where}$$

S is the land slope (%) and
V.I is the vertical interval (m)

The earthwork for contour bunding includes the main contour bund and side and lateral bunds. The area of cross-section of side and lateral bunds is taken equal to the main contour bund. The product of cross sectional area of the bund and the bunding intensity gives the quantity of earthwork required for bunding / hectare of land.

6.2.2.2.3 Deviation Freedom: Strict adherence to contours while constructing bunds is a necessary prerequisite for ensuring maximum conservation of moisture and soil. However, to avoid excessive curvature of bunds, which makes agricultural operations difficult, the following deviations are permitted

a) a maximum of 15 cm while cutting across a narrow ridge,
b) a maximum of 30 cm while crossing a gully or depression and
c) a maximum of 1.5 m while crossing a sharp, narrow depression not exceeding 5 m in width.

6.2.2.3 Contour Trenches

Contour trenches are rainwater harvesting structures, which can be constructed on hill slopes as well as on degraded and barren waste lands in both high- and low- rainfall areas. Cross section of a typical contour trench is shown in **Fig.6.6.**

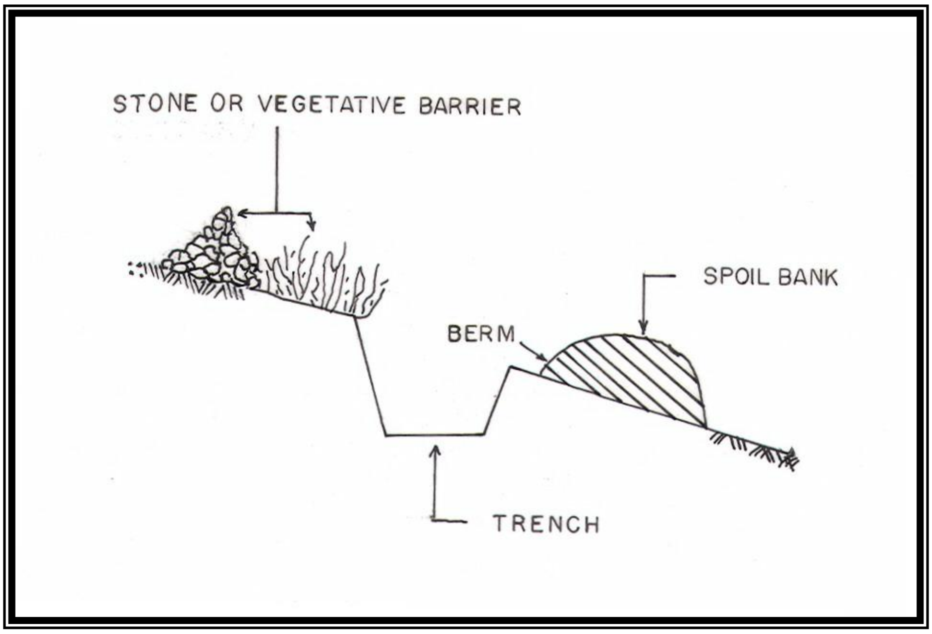


Fig.6.6 Schematics of a Contour Trench

The trenches break the slope at intervals and reduce the velocity of surface runoff. The water retained in the trench will help in conserving the soil moisture and ground water recharge.

The size of the contour trench depends on the soil depth and normally 1000 to 2500 sq. cm cross sections are adopted. The size and number of trenches are worked out on the basis of the rainfall proposed to be retained in the trenches. The trenches may be continuous or interrupted and should be constructed along the contours. Continuous trenches are used for moisture conservation in low rainfall area whereas intermittent trenches are preferred in high rainfall area.
The horizontal and vertical intervals between the trenches depend on rainfall, slope and soil depth. In steeply sloping areas, the horizontal distance between the two

trenches will be less compared to gently sloping areas. In areas where soil cover is thin, depth of trenching is restricted and more trenches at closer intervals need to be constructed. In general, the horizontal interval may vary from 10 m in steep slopes to about 25 m in gentle slopes.

6.2.2.4 Gully Plugs, *Nalah* Bunds and Check Dams

These structures are constructed across gullies, *nalah*s or streams to check the flow of surface water in the stream channel and to retain water for longer durations in the pervious soil or rock surface. As compared to gully plugs, which are normally constructed across 1st order streams, *nalah* bunds and check dams are constructed across bigger streams and in areas having gentler slopes. These may be temporary structures such as brush wood dams, loose / dry stone masonry check dams, Gabion check dams and woven wire dams constructed with locally available material or permanent structures constructed using stones, brick and cement. Competent civil and agro-engineering techniques are to be used in the design, layout and construction of permanent check dams to ensure proper storage and adequate outflow of surplus water to avoid scours on the downstream side for long-term stability of the dam.

The site selected for check dam should have sufficient thickness of permeable soils or weathered material to facilitate recharge of stored water within a short span of time. The water stored in these structures is mostly confined to the stream course and the height is normally less than 2 m. These are designed based on stream width and excess water is allowed to flow over the wall. In order to avoid scouring from excess runoff, water cushions are provided on the downstream side. To harness maximum runoff in the stream, a series of such check dams can be constructed to have recharge on a regional scale. The design particulars of a cement *nalah* bund are shown in **Fig.6.7.**

The following parameters should be kept in mind while selecting sites for check dams / *nalah* bunds:

- i) The total catchment area of the stream should normally be between 40 and 100 ha. Local situations can, however, be a guiding factor in this regard.
- ii) The rainfall in the catchment should be preferably less than 1000 mm / annum.
- iii) The stream bed should be 5 to 15 m wide and at least 1m deep.
- iv) The soil downstream of the bund should not be prone to water logging and should have a pH value between 6.5 and 8.
- v) The area downstream of the Check Dam / bund should have irrigable land under well irrigation.
- vi) The Check dams / *Nalah* bunds should preferably be located in areas where contour or graded bunding of lands have been carried out.
- vii) The rock strata exposed in the ponded area should be adequately permeable to cause ground water recharge.

Check dams / *Nalah* bunds are normally 10 to 15 m long, 1 to 3 m wide and 2 to 3 m high, generally constructed in a trapezoidal form. Detailed studies are to be made in the watershed prior to construction of the check dam to assess the current erosion condition, land use and water balance. The community in the watershed should also be involved in the planning and selection of the type and location of the structure.

For construction of the check dam, a trench, about 0.6 m wide in hard rock and l.2 m wide in soft impervious rock is dug for the foundation of core wall. A core brick cement wall, 0.6 m wide and raised at least 2.5m above the *nalah* bed is erected and the remaining portion of trench back filled on upstream side by impervious clay. The core wall is buttressed on both sides by a bund made up of local clays and stone pitching is done on the upstream face. If the bedrock is highly fractured, cement grouting is done to make the foundation leakage free.

6.2.2.5 Percolation Tanks

Percolation tanks, which are based on principles similar to those of *nalah* bunds, are among the most common runoff harvesting structures in India. A percolation tank can be defined as an artificially created surface water body submerging a highly permeable land area so that the surface runoff is made to percolate and recharge the ground water storage. They differ from *nalah* bunds in having larger reservoir areas. They are not provided with sluices or outlets for discharging water from the tank for irrigation or other purposes. They may, however, be provided with arrangements for spilling away the surplus water that may enter the tank so as to avoid over-topping of the tank bund.

It is possible to have more than one percolation tank in a catchment if sufficient surplus runoff is available and the site characteristics favour artificial recharge through such structures. In such situations, each tank of the group takes a share in the yield of the whole catchment above it, which can be classified as

(i) 'free catchment', which is the catchment area that only drains into the tank under consideration and
(ii) 'combined catchment', which is the area of the whole catchment above the tank.

The difference between the combined and free catchment gives the area of the catchment intercepted by the tanks located upstream of any tank. The whole catchment of the highest tank on each drainage shall be its free catchment. Moreover, each tank will receive the whole runoff from its free catchment, but from the remainder of its catchment it will receive only the balance runoff that remains after the upper tanks have been filled.

6.2.2.5.1 Site Selection Criteria: The important site selection criteria for percolation ponds include

i) The hydrogeology of the area should be such that the litho-units occurring in the area of submergence of the tank should have high permeability. The soils in the catchment area of the tank should be sandy to avoid silting up of the tank bed.
ii) The availability of non-committed surplus monsoon runoff should be sufficient to ensure filling of the tank every year.

iii) As the yield of catchments in low rainfall areas generally varies between 0.44 to 0.55 M Cu m/sq km, the catchment area may be between 2.50 and 4.0 sq km for small tanks and between 5.0 and 8.0 sq km for larger tanks.

iv) Selection of the size of a percolation tank should be governed by the percolation capacity of the strata rather than the yield of the catchment. In order to avoid wastage of water through evaporation, larger capacity tanks should be constructed only if percolation capacity is proven to be good. If percolation rates are low to moderate, tanks of smaller capacity may be constructed. Percolation tanks are normally designed for storage capacities of 8 to 20 M cft. (2.26 to 5.66 M Cu m).

v) The depth of water impounded in the tank provides the recharge head and hence it is necessary to design the tank to provide a minimum height of ponded water column of 3 to 4.5 m and rarely 6 m above the bed level. This would imply construction of tanks of large capacity in areas with steep gradient.

vi) The purpose of construction of percolation tanks is to ensure recharge of maximum possible surface water runoff to the aquifer in as short a period as possible without much evaporation losses. Normally, a percolation tank should not retain water beyond February.

vii) The percolation tank should be located downstream of runoff zone, preferably toward the edge of piedmont zone or in the upper part of the transition zone. Land slope between 3 and 5 percent is ideal for construction of percolation tanks.

viii) There should be adequate area suitable for irrigation and sufficient number of ground water abstraction structures within the command of the percolation tank to fully utilise the additional recharge. The area benefited should have a productive phreatic aquifer with lateral continuity up to the percolation tank. The depth to water level in the area should remain more than 3 m below ground level during post-monsoon period.

6.2.2.5.2 Investigations Required: An area, preferably the entire watershed, needing additional ground water recharge is identified on the basis of declining water level trends both during pre and post monsoon, increase in the demand of ground water and water scarcity during lean period etc. Areas having scarcity of water during summer in spite of incidences of flood during monsoons may also be considered for artificial recharge.

A base map, preferably on 1:50,000 scale showing all available geological, physiographical, hydrogeological and hydrological details along with land use, cropping pattern etc. is a pre-requisite for the scientific planning. Survey of India toposheets, aerial photographs and satellite imagery of the area may be consulted to gather preliminary information about the area under study. The nature of catchment as regards to the general slope, land use, forest cover, cropping pattern, soils, geology etc. should be understood to assess their influence on runoff.

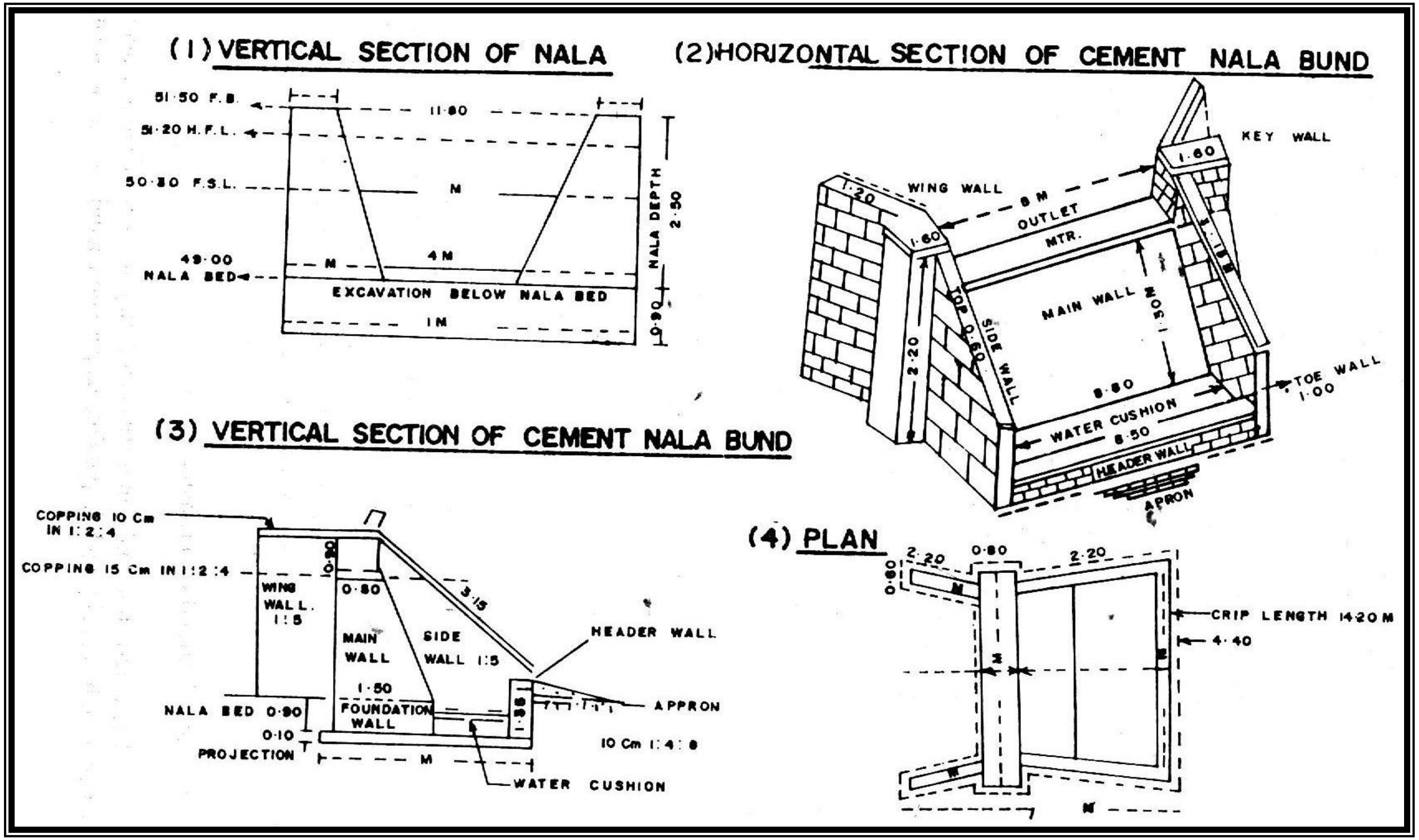


Fig.6. 7 Design Aspects of a Cement *Nalah* Bund

The rainfall data of rain gauge stations located in the watershed or in its immediate vicinity is to be collected during the preliminary investigations. The intensity and pattern of rainfall, number of rainy days and duration of dry spells during the monsoon are to be analyzed. The dependability of normal monsoon rainfall and the departure of actual rainfall from normal rainfall are also worked out along with other weather parameters.

Percolation tanks are to be normally constructed on second or third order streams, as the catchment area of such streams would be of optimum size. The location of tank and its submergence area should be in non-cultivable land and in natural depressions requiring lesser land acquisition. There should be cultivable land down stream of the tank in its command with a number of wells to ensure maximum benefit by such efforts. Steps should be taken to prevent severe soil erosion through appropriate soil conservation measures in the catchment. This will keep the tank free from siltation which otherwise reduces the percolation efficiency and life of the structure.

Detailed geological and hydrogeological mapping is to be carried out in the area of submergence, at the tank site and also downstream of the site to find out the permeability of vadose zone and aquifer. The potential of additional storage and capacity of aquifer to transmit the ground water in adjoining areas is also assessed based on aquifer geometry. Infiltration rates of soils in the probable area of submergence are to be determined through infiltration tests. Aquifer parameters of water-bearing formations in the zone of influence may also be determined to assess the recharge potential and number of feasible ground water structures in the area. Periodic water level measurements along with ground water sampling for water quality may be done before and after the construction of percolation tanks. Detailed geological investigations may be carried out to study the nature and depth of formation at the bund (dam) site for deciding the appropriate depth of cut off trench (COT). This will help in reducing the visible seepage and also ensure safety and long life of the structure. The depth of foundation and its treatment should be considered on the basis of nature of formation while designing and constructing the dam wall and waste weir.

6.2.2.5.3 Engineering Aspects: A percolation tank is essentially an earthen structure with a masonry spill way. It should be designed with maximum capacity utilisation, long life span, cost-effectiveness and optimum recharge to ground water in mind. Storage capacity, waste weir, drainage arrangements and cut off trench (COT) are the important features of percolation tank that need proper design. The overall design of the percolation tank is similar to that of an earthen dam constructed for minor irrigation.

Detailed topographical survey to demarcate the area of submergence in natural depression and alignment of dam line in the valley is to be taken up prior to construction of the structure. A number of sections along and across the drainage are prepared and the best suitable site is identified. The land availability and possibility of land acquisition is explored during the survey. The spillway site is demarcated and is designed in such a way that it allows the flow of surplus water based on single day maximum rainfall after the tank is filled to its maximum capacity. The depth of foundation for masonry work of waste weir etc. is decided depending on the nature of formation. Cut Off Trench (COT) is provided to minimize the seepage losses across the streambed. The depth of COT is generally 2-6 m below ground level depending upon the subsurface strata. In order to avoid erosion of bund due to ripple action, stone pitching is provided in the upstream direction up to High Flood Level (HFL). The sources for availability of constructional material, especially clay and porous soil for earthwork and stone rubble for pitching are to be identified.

a) Design of Storage Capacity: The storage capacity of a percolation tank may be defined as the volume of water stored in the tank up to the Full Tank Level (FTL). The storage capacity can be computed by using the contour plan of the water-spread area of the tank. The total capacity of the tank will be the sum of the capacities between successive contours. The smaller the contour interval, the more accurate the capacity computation will be. The summation of all the volumes between successive contours will be required for computing the storage capacity of the tank. When contour plan is not available and only the area of the tank at FTL is known, then the effective volume of the tank may be roughly computed as the area multiplied by one-third of the depth from FTL to the deep bed of the tank.

The tank is designed to ensure maximum utilisation of its capacity. A structure of optimum capacity is the most cost effective. An under-utilized structure leads to unproductive expenditure incurred on extra earthwork. The design of storage capacity of a tank depends mainly upon the proper estimation of catchment yield, which is calculated as,

$$\mathbf{Q = A * Strange's\ Coefficient}$$

Where, Q is yield at site and A is area of the catchment.

Strange's coefficients for various amounts of monsoon rainfall for three categories of catchments, i.e. good, average and bad are available from Strange's tables provided in standard Hydrology text books. The rainfall data of 40-50 years, collected from the nearest rain gauge station, may be used for design purposes. The percolation tanks are to be designed for a realistic percentage of the yield of the catchment considering the temporal distribution of monsoon rainfall. Another important consideration is the fact that water stored in a percolation tank starts percolating immediately and the terminal storage in the tank is not the cumulative storage from different spells of rain. The concept of storage capacity of percolation ponds thus differs significantly from that of an irrigation tank.

The catchment yield and basin configuration drawn from topographic surveys at site determine the height of the percolation tank. The top of dam wall is normally kept 2-3 m wide. Upstream and downstream slopes of the dam wall are normally taken as 2.5:1 and 2:1 respectively as recommended in design manual for minor irrigation tanks. The design particular of a typical percolation tank is shown in **Fig.6.8** along with all relevant details.

b) Design of Tank Bund: The tank bund, for all practical purposes, is a small-sized earthen dam and its design and construction should be carried out in accordance with the principles applicable to earthen dams.

The bunds of a percolation pond may be of three types, i.e.

Type A: Homogeneous embankment type **(Fig. 6.9(a))**
Type B: Zoned Embankment Type **(Fig. 6. 9(b))**
Type C: Diaphragm Type **(Fig. 6.9(c))**

Tank bunds in India are mostly of Type A and are constructed with soils excavated from pits in the immediate vicinity of the bund and transported to the bund.

The most commonly adopted standards used for fixing the dimensions of tank bunds, particularly in South India are given in **Table 6.5.** In favourable soils such as gravels, black loams etc., the side slopes of the bund may be kept at 0.5:1 for smaller tanks with water depths not exceeding 2.50 m and 2:1 for larger tanks up to 5.0 m deep. In light sandy or black clayey soils, on the other hand, the slopes may be kept between 2:1 or 2.5:1.

Table 6. 5 Common Dimensions of Bunds of Percolation Tanks

Sl.No	Maximum Water Depth (m)	Free Board (m)	Width of Top of Bund (m)
1	1.5 to 3.0	0.90	1.20
2	3.0 to 4.5	1.20	1.50
3	4.5 to 6.0	1.50	1.80
4	Over 6.0	1.80	2.70

The upstream face of the tank bund is generally riveted with stone apron or riprap (**Fig.6.10**) so as to protect it against erosion and if this is done, then the upstream slope generally adopted is 1.5:1, even up to 6 m depth. For inferior soils or greater depths, however, the riveted slope may be made flatter, say 2:1.

In this way, for average cases, a 1.5:1 slope will generally be adopted for upstream face and 2:1 slope for downstream face.

This practice is contrary to the standard recommendations adopted in many countries where the upstream slope, even when riveted, is kept flatter than the downstream slope because of the soil being saturated. There are, however, thousands of tanks in Tamil Nadu with slopes of 1.5:1 and failure by slipping of this slope is rare. Hence, the prevailing practice can be easily adopted. In very small tanks and in cases where the upstream slope is heavily riveted, upstream faces have been given 1:1 or even steeper slopes in actual practice, but such steeper slopes are not recommended.

c) Waste/ Surplus Weir: The waste/surplus weirs are constructed for discharging the excess water from the tank into the downstream channel after it is filled so as to avoid the rise of water in the tank above the Maximum Water Level (MWL). The water will start spilling over the crest of this escape weir as and when it rises above the FTL and the discharging capacity of this weir will be so designed as to pass the full flood discharge likely to enter the tank with a depth over the weir equal to the difference between FTL and MWL.

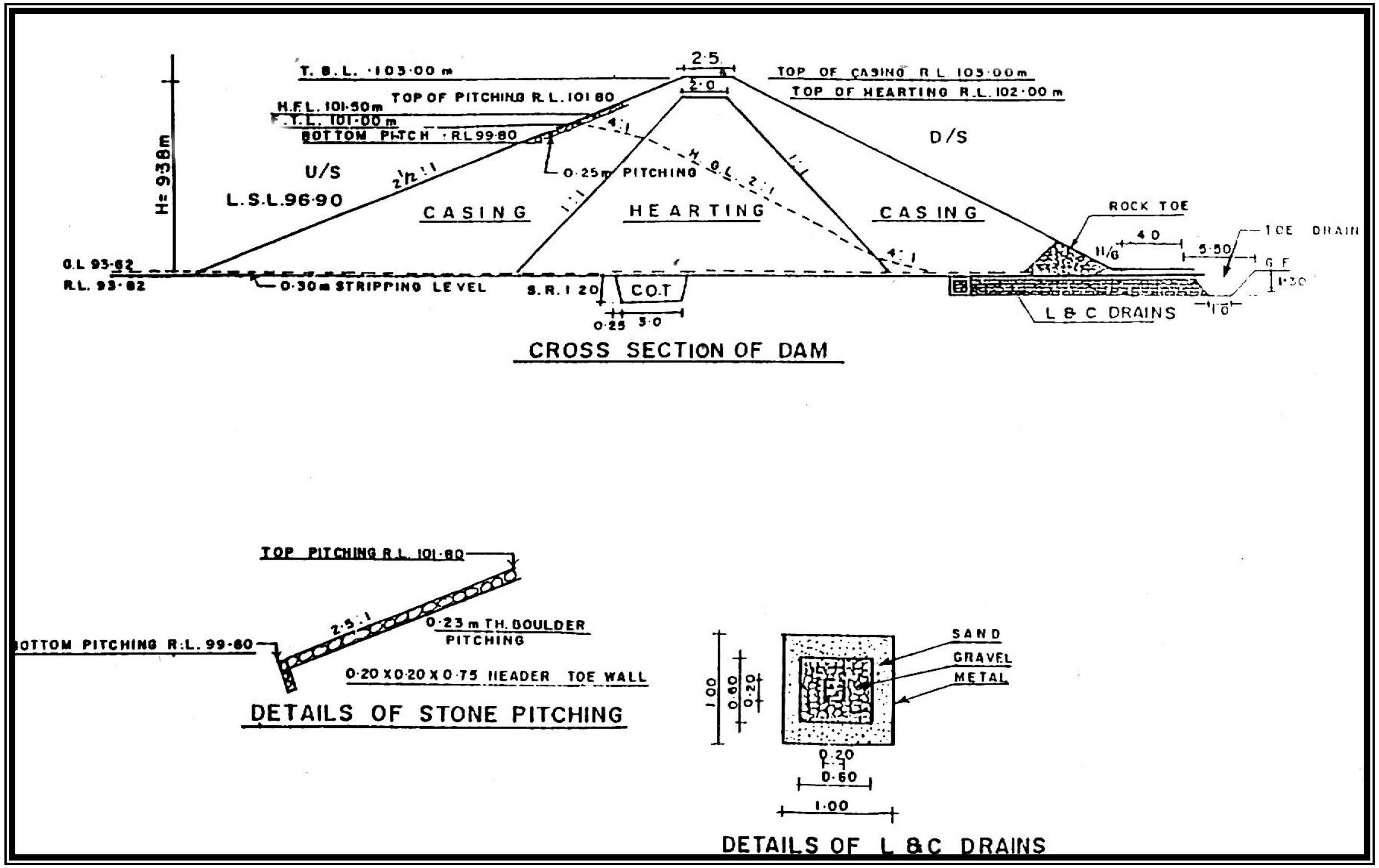
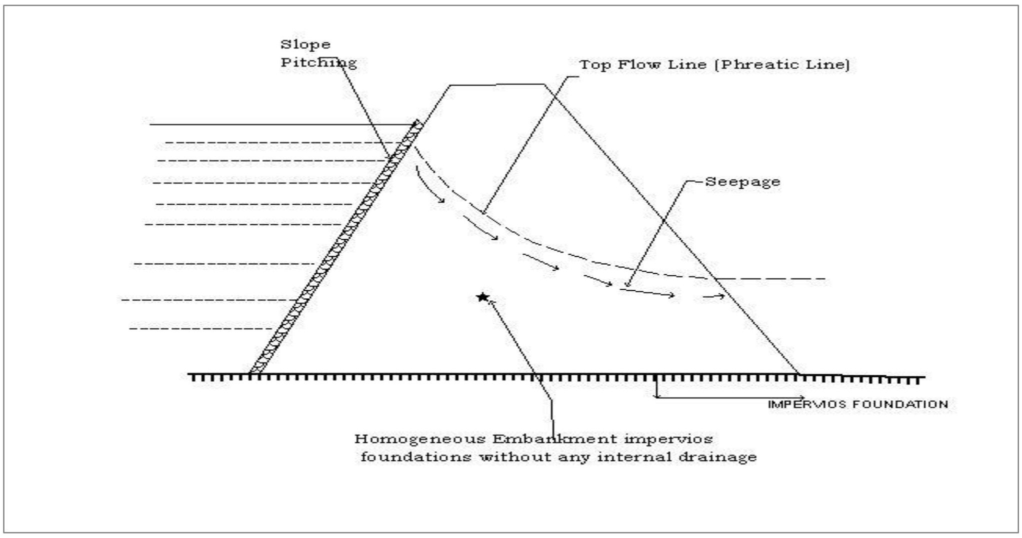


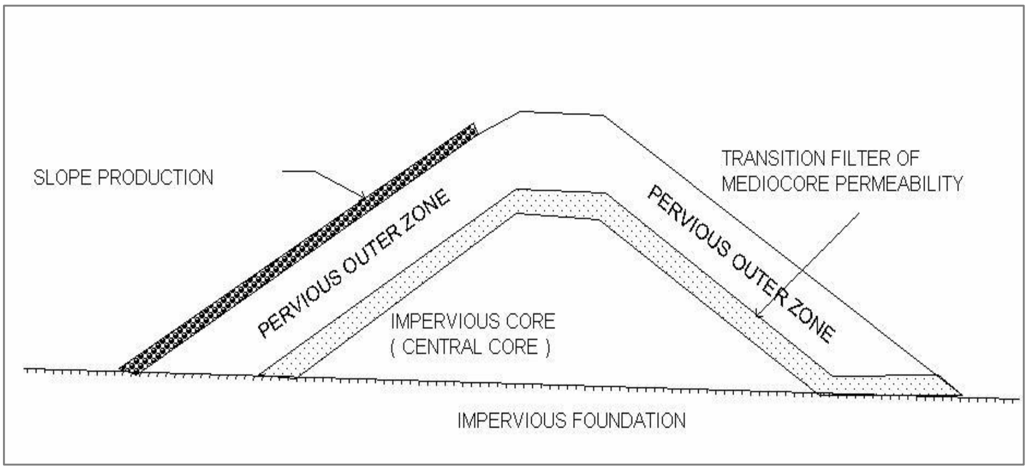
Fig.6.8 Design Aspects of a Typical Percolation Pond

a)



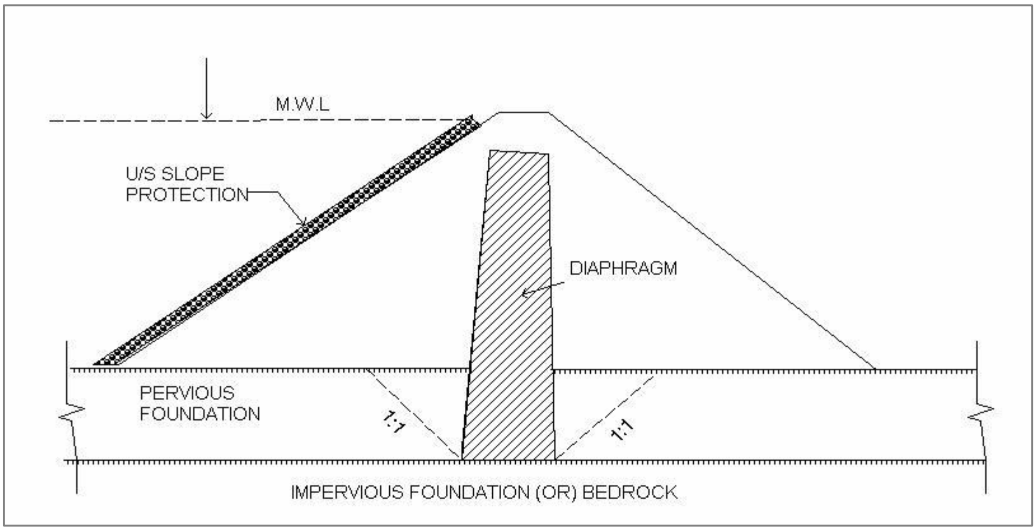
Homogeneous Type

b)



Zoned Type

c)



Diaphragm Type

Fig. 6.9 Common Types of Bunds of Percolation Ponds

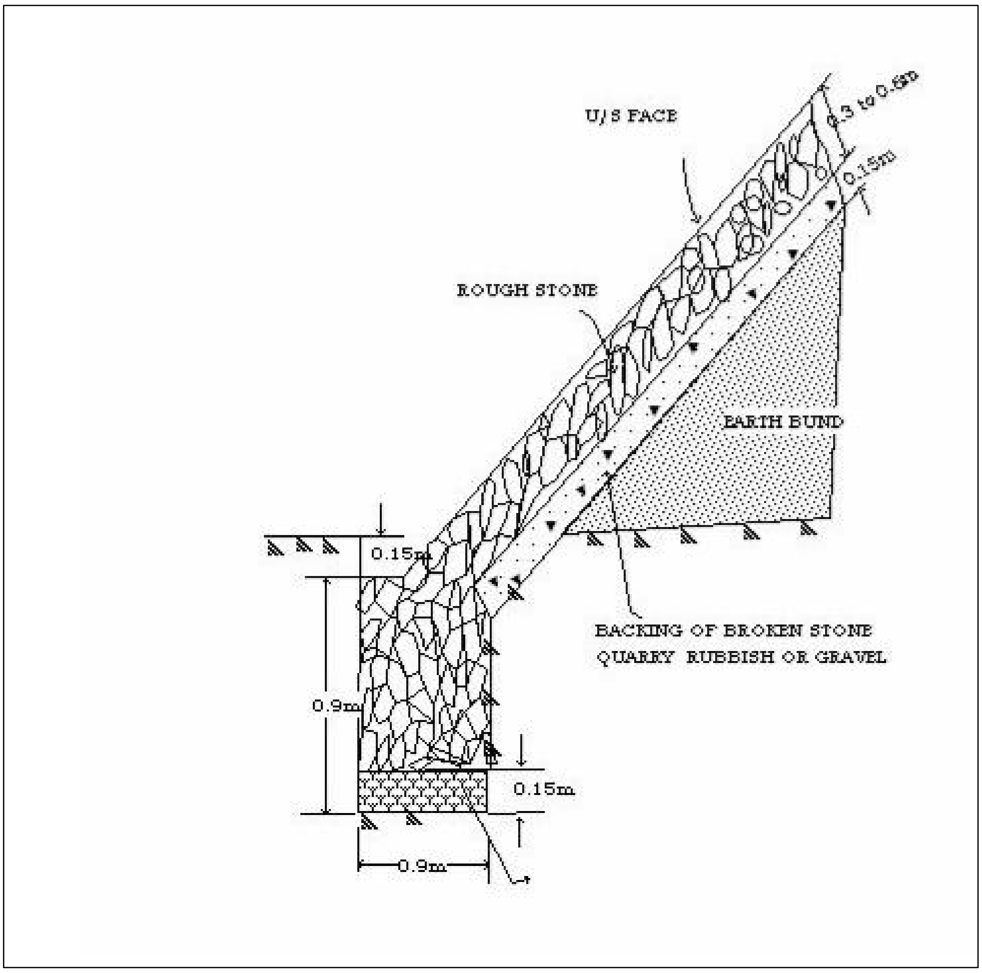


Fig. 6.10. Upstream Revetment of Tank Bunds

Although the effective storage capacity of a percolation tank is limited by FTL, the area submerged by the tank bund and revetment is dependent on MWL. Hence, in order to restrict the dimensions of these, it is desirable to keep the difference between FTL and MWL as small as possible. On the other hand, the smaller the difference, the longer will be the surplus escape required in order to enable it to pass the given discharge. Hence, the difference (H) between FTL and MWL is fixed on a compromise basis in each particular project so as to obtain maximum economy and efficiency. In small and medium sized tanks, the usual difference between FTL and MWL is kept between 0.30 and 0.60 m and is rarely allowed to exceed 0.90 m.

Surplus weirs are similar to river weirs (i.e. Diversion weirs or *anicuts*) and are classified into the following three general types

Type A: Masonry weirs with a vertical drop
Type B: Rock fill weirs with a sloping apron and
Type C: Masonry weirs with a sloping masonry apron (glacis)

i) **Masonry Weirs with Vertical Drop (Type A):** A typical cross section of such a weir is shown in **Fig. 6.11(a).** This weir consists of a horizontal floor and a masonry crest with vertical or near-vertical downstream face. The raised masonry crest does the maximum ponding of water but a part of it is usually carried out by shutters at the top of the crest. The shutters can be dropped down during floods so as to reduce the afflux (the rise in the Maximum Flood Level (HFL) upstream of the weir caused due to the construction of the weir) by increasing the waterway opening. This type of weir is particularly suitable for hard clay and consolidated gravel foundations. However, these weirs are fast becoming obsolete and are being replaced by modern concrete weirs.

ii) **Rock-fill Weirs with Sloping Aprons (Type B):** These weirs are also known as 'Dry Stone Slope Weirs'. A typical cross section of such a weir is shown in **Fig. 6.11(b).** It is the simplest type of construction and is suitable for fine sandy foundations like those encountered in alluvial areas in North India. Such weirs require huge quantities of stone and are economical only when stone is easily available. The stability of such weirs is not amenable to theoretical treatment. With the development of concrete glacis weirs, these weirs are also becoming obsolete.

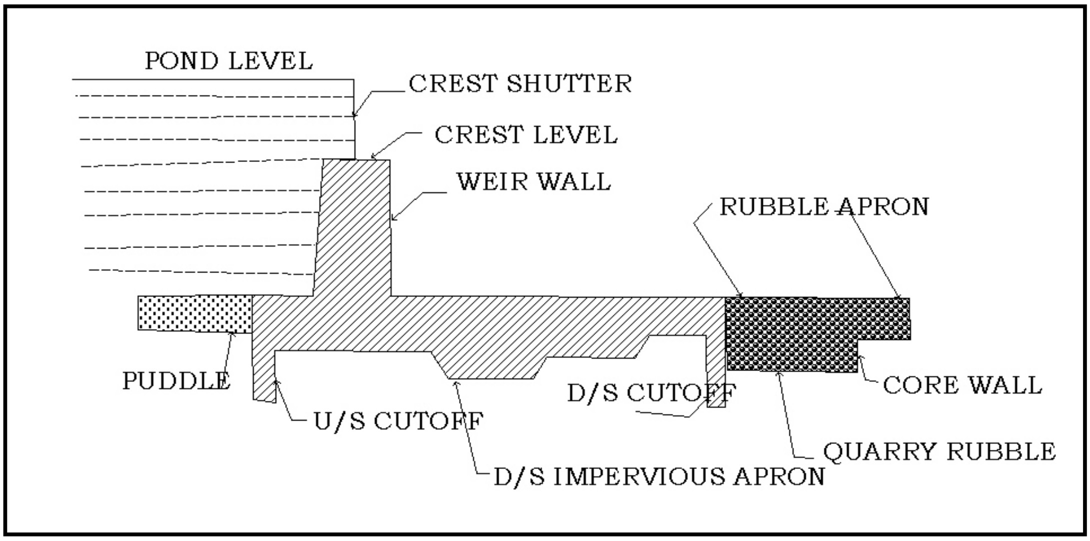


Fig. 6.11(a) A Typical Masonry Tank Weir with a Vertical Drop.

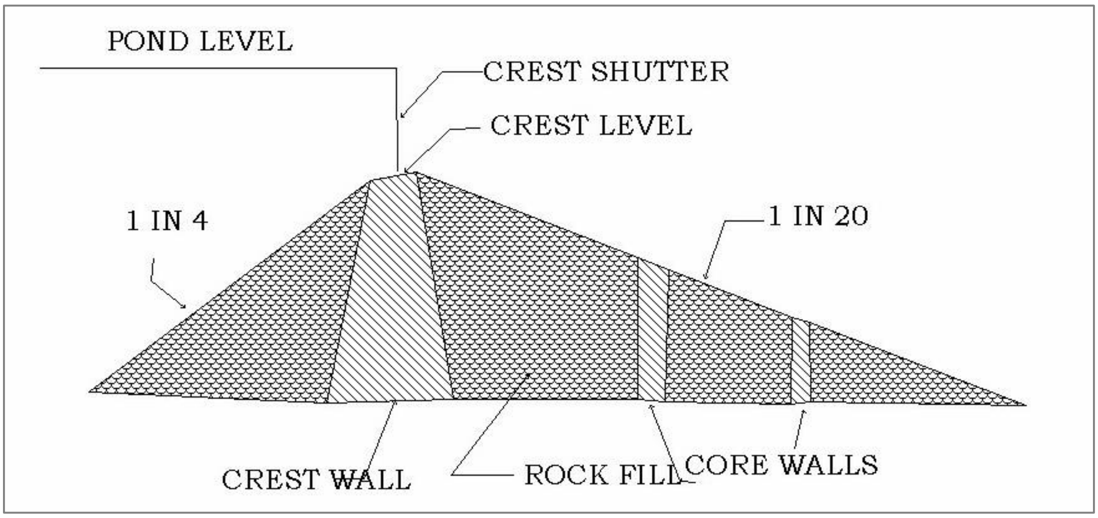


Fig. 6.11(b) A Typical Rock-filled Weir with Sloping Aprons.

iii) **Modern Concrete Weirs with Sloping Downstream Glacis (Type C):** Weirs of this type are of recent origin and their design is based on modern concepts of sub-surface flow. A typical cross-section of such a weir is shown in F**ig: 6.11(c)**. Sheet piles of sufficient depths are driven at the ends of upstream and downstream floor. Sometimes, an intermediate pile line is also provided. The hydraulic jump is formed on the downstream sloping glacis so as to dissipate the energy of the flowing water.

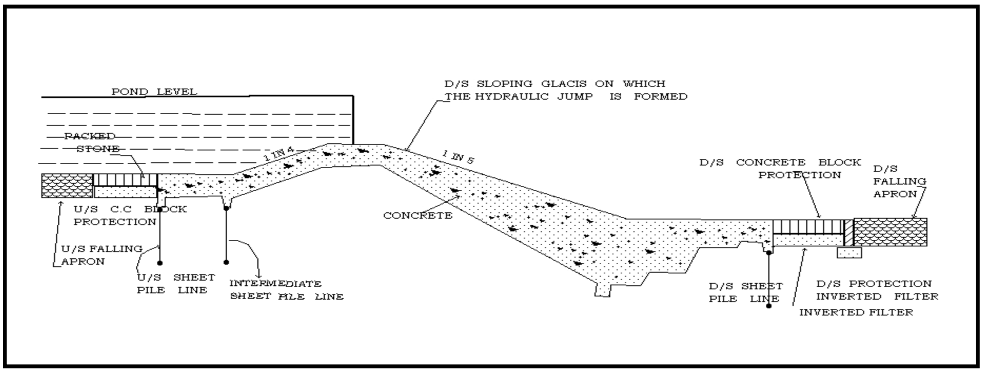


Fig. 6.11(c) Typical Cross-section of a Modern Concrete Weir with Permeable Foundation.

Besides these three important types of weirs, a combination of type A and type C may also be used. In such weirs, a number of vertical steps are made instead of providing a horizontal or sloping downstream apron. Such weirs are called Type D weirs or 'weirs with stepped aprons' and is shown in **Fig.6.11(d)**. A & D types are most commonly used in percolation tanks.

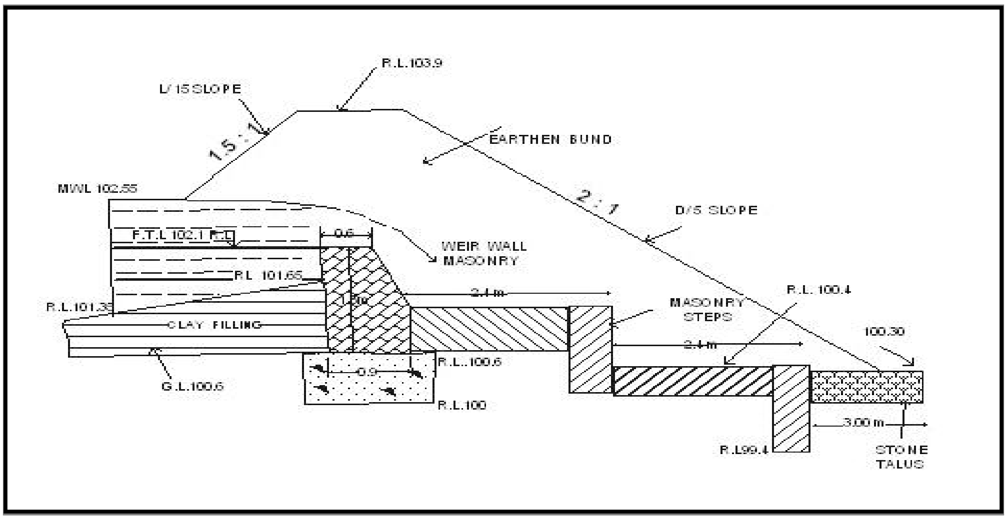


Fig. 6.11(d) A Typical Stepped – Apron Tank Weir

d) Design Aspects of Waste Weirs

i) Width of floors of Weirs: The widths of horizontal floors of type A and D weirs from the foot of the drop wall to the downstream edge of the floor should never be less than 2(D+H) where D is the height of the drop wall and H is the maximum water head over the wall. In major works, this width may be increased to 3(D+H). The rough stone apron forming a talus below the last curtain wall may be of varying widths depending on the nature of the soil, velocity and probable quantity and intensity of annual runoff. It would generally vary from 2.5(D+H) to 5(D+H) depending on local conditions.

ii) Length of Surplus weirs: In order to determine the length of surplus weirs, it is necessary to determine the maximum flood discharge that may enter the tank after it is filled up to full tank level. If the tank is an independent one, the flood discharge can be estimated using Ryve's formula

$$Q = CM^{2/3}$$

Where 'Q' is the estimated flood discharge in cubic meters/second, 'M' is the area of the catchment in square kilometers and 'C' is known as 'Ryve's coefficient' usually ranging from 6.8 to 15 depending upon the topography of the catchment and intensity of rainfall over the catchment. If the tank is part of a group of tanks, the flood discharge likely to enter such a tank is calculated using the formula

$$Q = \mathbf{CM^{2/3} - cm^{2/3}}$$

Where 'Q' is the estimated flood discharge in cubic meters/second that is likely to enter the tank in question, 'M' is the combined catchment area of all tanks above the surplus of the tank in square kilometers, 'm' is the intercepted catchment area in square kilometers by the upper tanks, 'C', Ryve's coefficient varying from 6.8 to 15 and 'c', modified coefficient which generally varies from 1/5 to 1/3 of C.

In case of catchments of less than two square kilometers, it is better to adopt discharges obtained by calculating the runoff from the catchment with a precipitation of 2.54 cm/hour (Equivalent to 1 Inch/Hour). The flood discharge obtained from a catchment with 2.54 cm precipitation can be calculated from the following formula:

$$Q = 7M^{2/3}$$

Where 'Q' is the discharge in cubic meters/second obtained due to a precipitation of 2.54 cm/hr and 'M', the area of the catchment in square kilometers.

After assessing the flood discharge and fixing the FTL and MWL with reference to the storage requirements, the length of the surplus weir can be calculated from the formula

$$Q = 2/3\ C_d\, LH2\ gH \quad \text{or} \quad 2.95C_d\, LH^{3/2}, \text{ where}$$

'Q' is the quantum of flood water in cubic meter/second to be discharged, 'L', the length of the weir in meter, 'H', the head over the weir or the difference between MWL and FTL in meters and 'C_d, the coefficient of discharge, which varies depending upon the type of weir as given below in **Table.6. 6.**

Table.6.6. Coefficient Discharge for Various Types of Weirs

Sl. No	Type of weir	Value of C_d	Reduced Formula for Discharge per Meter Length of Weir
1	Weirs with crest width up to 1 m	0.625	$1.84H^{2/3}$
2	- do- with width >1m	0.562	$1.66H^{2/3}$
3	Rough Stone sloping escapes	0.50	$1.48H^{2/3}$
4	Flush escapes	0.437	$1.29H^{2/3}$

iii) Scouring Depth: This is controlled by the type of formation and also on discharge and is calculated by using following formula

$$\mathbf{D = 0.47\,(Q / f)^{1/3}}, \text{ where}$$

D is depth of scouring in meter,
Q is maximum discharge in m^3/sec. (silt factor)
f is coefficient of rugocity, which is taken as
f = 1.0 for hard rock
= 0.75 for soft rock
= 0.45 for gravel (*Murrum)*
= 0.30 for soil

e) Design of Water Cushion: Depth of water cushion is calculated by using following formula.

$$\mathbf{D = c\sqrt{d} \times 3\,\sqrt{h}}$$

Where, D is depth of water column in m,
h is difference between level of water passing over the weir and that of tail water (m)
d is vertical drop (m) and
c is a constant (coefficient of rugocity)

Length of water column (L) is calculated using following formula,

$$\mathbf{L = 6\,\sqrt{d}}$$

f) Design of Spill Channel: The Spill channel is designed on the basis of Maximum flood discharge (Q), bed width (L), maximum flood lift (H) and bed slope. The area of cross section (A) of waste weir is worked out as L * H (Sq m) and wetted perimeter (P) is worked out as L + (H*D) in metres.

Hydraulic mean depth (R) is calculated as, R = A/P

Velocity (V) = (1/N) * $R^{2/3}$ x $\sqrt{S}$ (m/sec),

Where S is the slope and N is taken as 0.03

Capacity of discharge Q = A * V (m^3 / sec)

The section capable of discharging floods equal to Q value estimated is adopted for the spill and approach channel.

g) Design of Cut off Trench (COT): A trench excavated below the ground surface along the bund line is known as cut off trench (COT). The depth of excavation depends upon the type of subsurface strata. A trial pit is excavated and dug wells and stream sections are also studied to determine the maximum depth of COT. It is recommended to dig COT down to the hard strata, or down to the depth equal to H (height of water column), whichever is less. The COT is then filled up to the ground by clayey soil. Clay is commonly used for filling. If COT of appropriate depth is not provided, the chances of visible seepage losses from the structure become high.

h) Design of Hearting and Casing: Hearting is the impervious core of the percolation tank bund, which is constructed of clayey material. The slopes of the hearting are 1:1 both on upstream and downstream sides. Its height is up to the highest flood level (HFL) of the dam. The hearting is covered with casing from all the sides. The material used for casing should be porous and devoid of clay content.

i) Stone Pitching: Stone pitching is done on the upstream face of the bund to protect the structure from erosion, which may be caused by the wave ripple action of water stored in the tank. The pitching is done using boulder and stone pieces of 20-30 cm size. It is done on the upstream side from bottom to the HFL. In some cases, strip pitching is also done below the HFL for few meters.

j) Dam Drainage Arrangement: Longitudinal and cross drains are provided below the bund in casing zone to drain out the water seeping into the structure during different stages of filling to prevent formation of sludge around the structure. For this, excavation is made down to 1m along the dam line beneath the casing zone. Cross drains are also excavated to ultimately drain out the water of longitudinal drains. These drains are filled with porous material in layered sequence of sand, and gravel

Toe drains are constructed at the downstream of dam wall to drain out water away from the structure.

k) Rock Toe: A rubble hump is normally provided over the ground surface on downward side of the tank, to protect the dam from slippage and sliding of casing zone.

6.2.2.6. Modification of Village Tanks as Recharge Structures

The existing village tanks, which are normally silted and damaged, can be modified to serve as recharge structures. Unlike in the case of properly designed percolation tanks, cut-off trenches or waste weirs are not provided for village tanks. Desilting of village tanks together with proper provision of waste weirs and cut off trenches on the upstream side can facilitate their use as recharge structures. As such tanks are available in plenty in rural India, they could be converted into cost-effective structures for augmenting ground water recharge with minor modifications.

6.2.2.7 Stream Channel Modification / Augmentation

In areas where streams zigzag through wide valleys occupying only a small part of the valley, the natural drainage channel can be modified with a view to increase the infiltration by detaining stream flow and increasing the streambed area in contact with water. For this, the channel is so modified that the flow gets spread over a wider area, resulting in increased contact with the streambed. The methods commonly used include a) widening, leveling, scarifying or construction of ditches in the stream channel, b) construction of L – shaped finger levees or hook levees in the river bed at the end of high stream flow season and c) Low head check dams which allow flood waters to pass over them safely.

Stream channel modification can be employed in areas having influent streams that are mostly located in piedmont regions and areas with deep water table such as arid and semi arid regions and in valley fill deposits. The structures constructed for stream channel modification are generally temporary, are designed to augment ground water recharge seasonally and are likely to be destroyed by floods. These methods are commonly applied in alluvial areas, but can also be gainfully used in hard rock areas where thin river alluvium overlies good phreatic aquifers or the rocks are extensively weathered or fractured in and around the stream channel. Artificial recharge through stream channel modifications could be made more effective if surface storage dams exist upstream of the recharge sites as they facilitate controlled release of water.

6.2.3. Subsurface Techniques

Subsurface techniques aim at recharging deeper aquifers that are overlain by impermeable layers, preventing the infiltration from surface sources to recharge them under natural conditions. The most common methods used for recharging such deeper aquifers are a) Injection wells or recharge wells, b) Recharge pits and shafts, c) Dug well recharge, d) Borehole flooding and e) Recharge through natural openings and cavities.

6.2.3.1 Injection Wells or Recharge Wells

Injection wells or recharge wells are structures similar to bore/tube wells but constructed for augmenting the ground water storage in deeper aquifers through supply of water either under gravity or under pressure. The aquifer to be replenished is generally one with considerable desaturation due to overexploitation of ground water. Artificial recharge of aquifers by injection wells can also be done in coastal regions to arrest the ingress of seawater and to combat problems of land subsidence in areas where confined aquifers are heavily pumped.

In alluvial areas, injection wells recharging a single aquifer or multiple aquifers can be constructed in a manner similar to normal gravel packed pumping wells. However, in case of recharge wells, cement sealing of the upper section of the wells is done to prevent the injection pressure from causing leakage of water through the annular space of the borehole and the well assembly. Schematics of a typical injection well in alluvial terrain are shown in **Fig.6.12.** In hard rock areas, injection wells may not require casing pipes and screens and an injection pipe with an opening against the fractures to be recharged may be sufficient. However, properly designed injection wells with slotted pipes against the zones to be recharged may be required for recharging multiple aquifer zones separated by impervious rocks.

The effectiveness of recharge through injection wells is limited by the physical characteristics of the aquifers. Attempts to augment recharge may prove to be counter-productive in cases where the aquifer material gets eroded due to the speed of ground water flow, especially in unconsolidated or semi-consolidated aquifers. Failure of confining layers may also occur if excessive pressure is applied while injecting water. These may result in clogging and/or even collapse of the bore/tube well.

6.2.3.1.1 Site Selection and Design Criteria

i) A proper understanding of the aquifer geometry is the most important factor in implementation of successful recharge schemes through injection or recharge wells. Detailed studies of the vertical and lateral extents of the aquifer and its characteristics are necessary prerequisites for such schemes. Grain size distribution of granular aquifers is another important parameter in the case of sedimentary aquifers.

ii) Recharge through injection wells increases chances of clogging of well screens and aquifer material, resulting in decreased injection rates. Clogging may be caused by suspended particles and air bubbles in the source water, formation of chemical precipitates in the well, source water or aquifer material, proliferation of bacteria in and around the injection well and swelling and dispersion of clay in the aquifer being recharged. Clogging may be minimized by proper treatment and removal of suspended material from source water, chemical stabilization and bacterial control. Using non-corrosive materials for pipelines and well casings may minimize clogging by corrosion products. Chlorination of source water prevents development of bacterial growth. Acid treatment helps in removing calcium carbonate precipitates from the gravel packs and aquifers. Periodic development of wells through surging, swabbing and pumping can considerably improve the efficiency and life of injection wells.

iii) As clogging increases the well losses considerably, the efficiency of injection wells should be taken as 40 to 60 percent as compared to pumping wells of similar design in the same situation.

iv) Adequate care should be taken to ensure that the water being used for recharge is not contaminated. The water being recharged should be compatible with the formation water to avoid any precipitation and resultant clogging. The relative temperatures of source and formation waters also affect the recharge rate.

v) For optimum benefits, it is advisable to have injection – cum – pumping wells to be used both for ground water recharge and extraction under favourable conditions.

vi) The following considerations are important in the design of an injection well

   a) The permissible pressure head of hydraulic injection in terms of water column may be worked out as 1.2 times the depth to the top of the confined aquifer, which represent the hydrofracturing pressure of the confining layer. In consolidated strata, however, this pressure is likely to be much higher. Injection of water at pressures exceeding this limit can result in the rupture of the confining layer.

b) The rate of recharge likely to be accepted by the aquifer may be worked out on the basis of observed discharge-drawdown relation of the existing pumping wells tapping the same aquifer. If the aquifer parameters are known, the recharge rates may be worked out from theoretical considerations using appropriate formulae. However, it is always desirable to determine the actual intake rates through injection/recharge tests in the wells.

c) The diameter of the conductor and casing pipes and the bore/tube well are to be worked out from the rate of recharge estimated. Usually, pipes with nominal diameters of 100mm, 150mm, 200mm and 250mm can handle flows up to 50 Cum/hr, 150 Cu m/hr, 250 Cu m/hr and 400 Cu m/hr respectively.

d) In case the well is being proposed as an injection – cum – pumping well, the well assembly should be so designed to accommodate higher flows while pumping.

e) The inner diameter of the housing pipe has to be two nominal diameters higher than the pump bowl size and the length of the housing pipe should be adequate to accommodate seasonal and long-term fluctuations, interference effects of surrounding wells in addition to expected drawdown and desired pump submergence.

f) The casing material used for the well must be similar to the one used for production wells and should have adequate tensile strength and collapsing pressure. In case chemical treatment is anticipated during development, the casing pipe and screens should be made of corrosion-resistant material.

g) The recharge well should be designed to fully penetrate the aquifer to avoid additional head losses due to partial penetration. In hard rocks, the top casing should be adequate to cover the unconfined zone.

h) Artificial gravel packs should be provided around screens in case of screened wells in unconsolidated and semi-consolidated formations. The gravel pack should be so designed to arrest the inflow of aquifer particles into the well.

i) It is advisable to achieve exit velocity comparable to entrance velocity recommended (0.03m/sec) for pumping wells to reduce incrustation and corrosion by providing appropriate open area for passage of water into the aquifer. The desired open area can be achieved for a given thickness of aquifer by adjusting well casing diameter and percent open area of the screen using the relation

Total area of the screen x Percent open area = Volume x Entrance Velocity.

j) Injection wells may be designed to recharge a single aquifer or multiple aquifers.

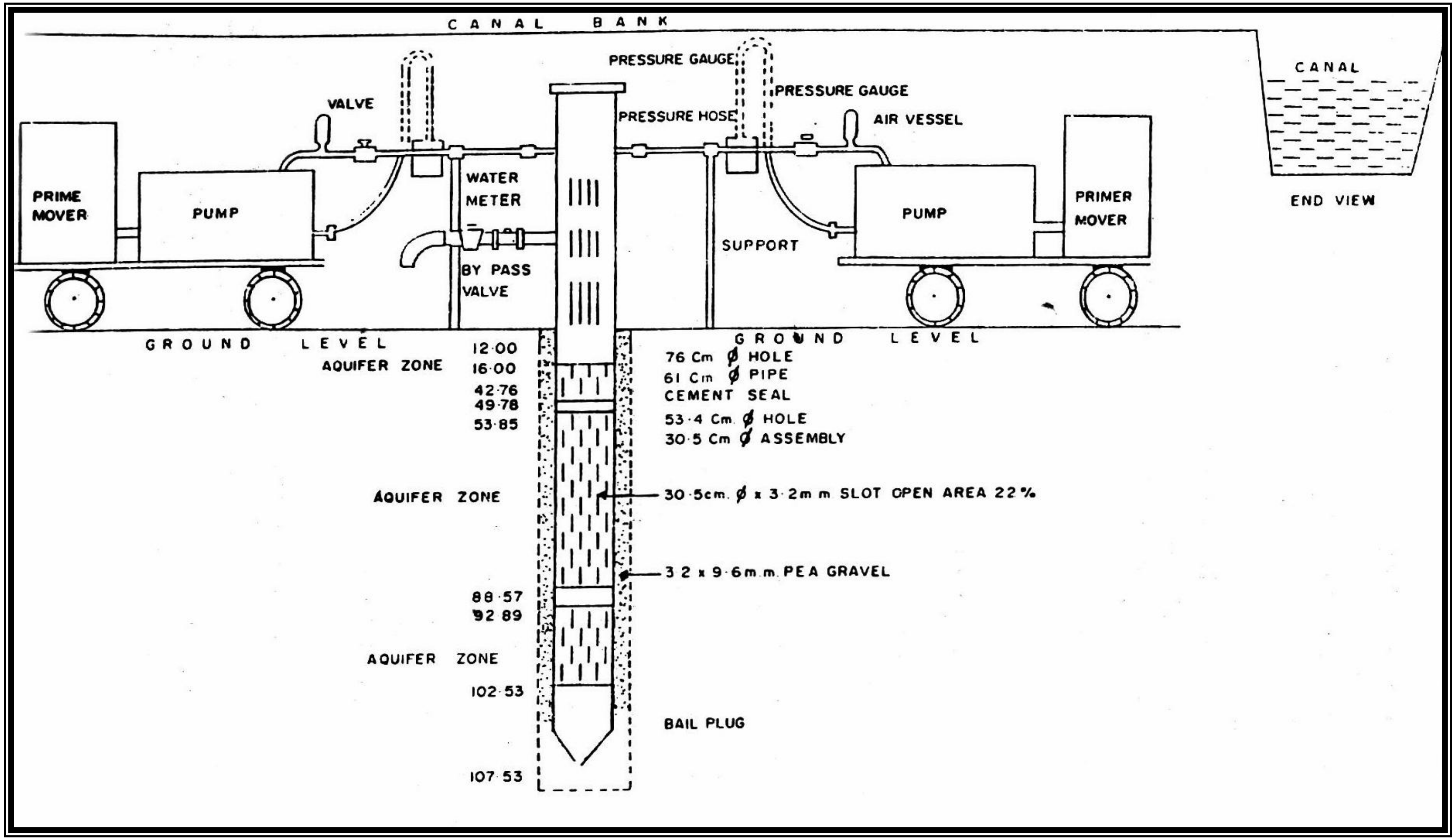


Fig.6.12 Schematics of a Typical Injection Well in Alluvial Terrain

k) For pressure injection, conductor pipes of suitable diameter should be used to reach the aquifer with an inflatable packer to be placed around the pipe just above the screen. In a dual injection well, the inflatable packer is a must.

vii) Injection of water into the well should be started at rates below the pre-estimated injection rate, which is then progressively increased, taking care to ensure that the pressure build-up remains below the permissible limit. Once the maximum permissible injection rate is attained, the well should be regularly monitored for injection rate, injection head and quality of water.

viii) The Specific Injection Capacity of the well, computed as the ratio of the quantum of water applied to the head build-up in the well is determined on commissioning a recharge well. The Specific Injection Capacity of the well reduces with time due to clogging. When the injection rate falls below accepted economic limits, the well is required to be redeveloped.

6.2.3.2 Gravity Head Recharge Wells

In addition to specially designed injection wells, existing dug wells and tube/bore wells may also be alternatively used as recharge wells, as and when source water becomes available. In areas where considerable de-saturation of aquifers have already taken place due to over-exploitation of ground water resources resulting in the drying up of dug wells and lowering of piezometric heads in bore/tube wells, existing ground water abstraction structures provide a cost-effective mechanism for artificial recharge of the phreatic or deeper aquifer zones as the case may be. Schematics of a typical system for artificial recharge through dug wells are shown in **Fig.6.13.**

6.2.3.2.1 Site Characteristics and Design Guidelines

i) In areas where excess surface water is available during rainy season and the phreatic aquifers remain unsaturated, surface water can be pumped into the dug wells for augmentation of ground water resources.

ii) Wells with higher yields before getting dried up due to the de-saturation of aquifers should be selected for recharge as they prove to be more suitable for ground water recharge when compared to low-yielding wells.

iii) The recharge head available in gravity head recharge wells is the elevation difference between the surface water level in the feeder reservoir /tank and the elevation of water table or piezometric head. The recharge rates in such cases are likely to be much less when compared to pressure injection and will also keep on reducing with build-up of the water table in the aquifer.

iv) Pumping of wells during periods of non-availability of recharge water helps in removing the silt that may enter the well during recharge. However, more rigorous development may be essential in the case of deep bore/tube wells

v) Care should be taken to ensure that the source water is adequately filtered and disinfected when existing wells are being used for recharge. The recharge water should be guided through a pipe to the bottom of well, below the water level to avoid scouring of bottom and entrapment of air bubbles in the aquifer.

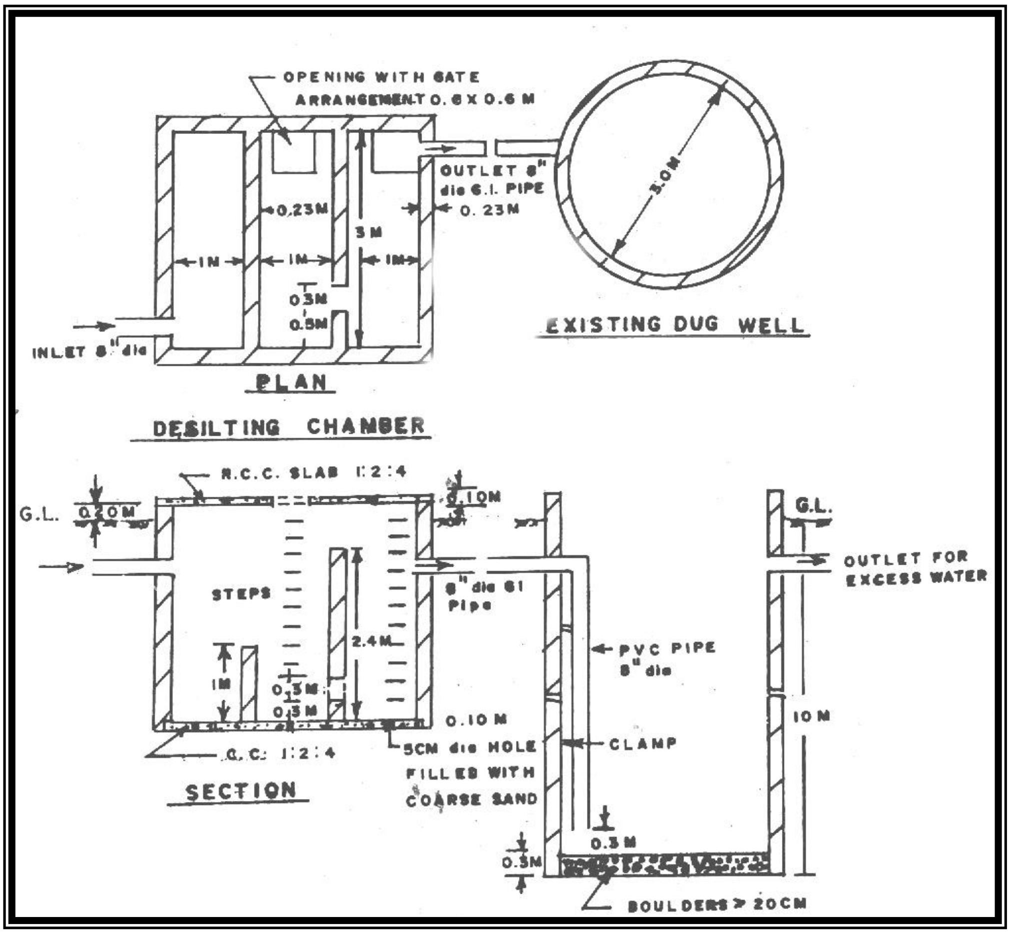


Fig.6.13 Schematics of a Typical System for Artificial Recharge through Dug Well.

6.2.3.3 Recharge Pits and Shafts

Recharge pits and shafts are artificial recharge structures commonly used for recharging shallow phreatic aquifers, which are not in hydraulic connection with surface water due to the presence of impermeable layers. They do not necessarily penetrate or reach the unconfined aquifers like gravity head recharge wells and the recharging water has to infiltrate through the vadose zone.

6.2.3.3.1 Recharge Pits: Recharge pits are normally excavated pits, which are sufficiently deep to penetrate the low-permeability layers overlying the unconfined aquifers (**Fig.6.14)**. They are similar to recharge basins in principle, with the only difference being that they are deeper and have restricted bottom area. In many such structures, most of the infiltration occurs laterally through the walls of the pit as in most layered sedimentary or alluvial material the lateral hydraulic conductivity is considerably higher than the vertical hydraulic conductivity. Abandoned gravel quarry pits or brick kiln quarry pits in alluvial areas and abandoned quarries in basaltic areas can also be used as recharge pits wherever they are underlain by permeable horizons. *Nalah* trench is a special case of recharge pit dug across a streambed. Ideal sites for such trenches are influent stretches of streams. Contour trenches, which have been described earlier also belongs to this category.

6.2.3.3.1.1 Site Characteristics and Design Guidelines

i) The recharging capacity of the pit increase with its area of cross section. Hence, it is always advisable to construct as large a pit as possible.
ii) The permeability of the underlying strata should be ascertained through infiltration tests before taking up construction of recharge pits.
iii) The side slopes of recharge pits should be 2:1 as steep slopes reduce clogging and sedimentation on the walls of the pit.
iv) Recharge pits may be used as ponds for storage and infiltration of water or they may be back-filled with gravel sand filter material over a layer of cobbles/boulders at the bottom. Even when the pits are to be used as ponds, it is desirable to provide a thin layer of sand at the bottom to prevent the silt from clogging permeable strata.
v) As in the case of water spreading techniques, the source water being used for recharge should be as silt-free as possible.
vi) The bottom area of the open pits and the top sand layer of filter-packed pits may require periodic cleaning to ensure proper recharge. Recharge pits located in flood-prone areas and on streambeds are likely to be effective for short duration only due to heavy silting. Similar pits by the sides of

streambeds are likely to be effective for longer periods.

vii) In hard rock areas, streambed sections crossing weathered or fractured rocks or sections along prominent lineaments or intersection of lineaments form ideal locations for recharge pits.

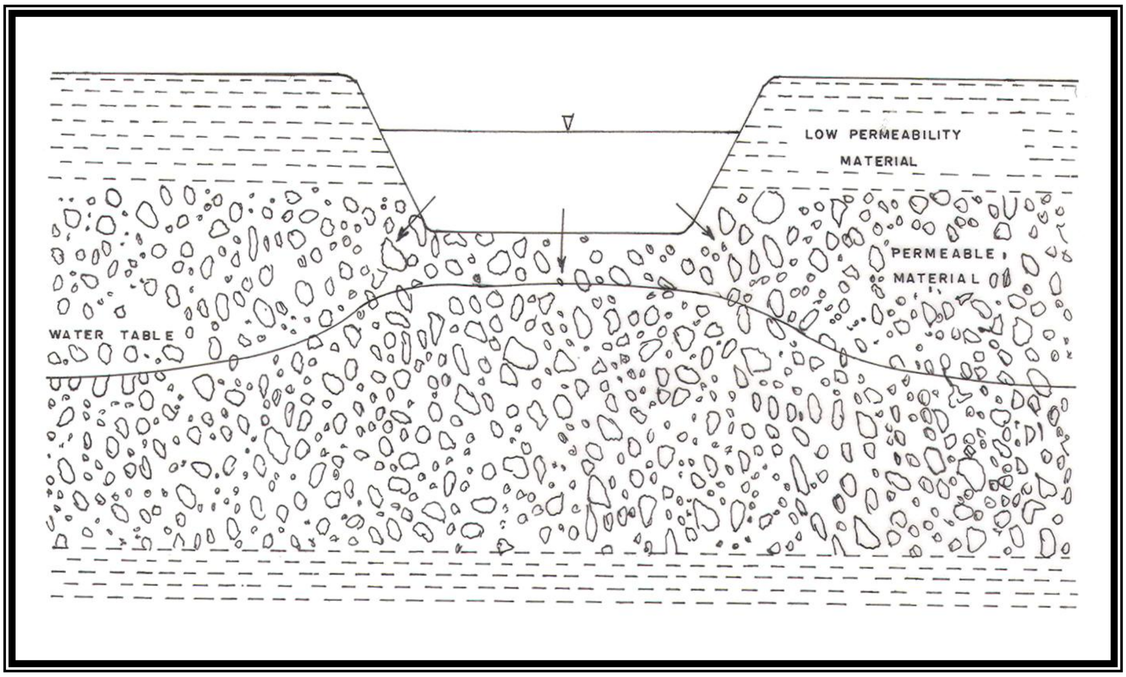


Fig.6.14 Schematics of a Recharge Pit.

6.2.3.3.2 Recharge Shafts

Recharge Shafts are similar to recharge pits but are constructed to augment recharge into phreatic aquifers where water levels are much deeper and the aquifer zones are overlain by strata having low permeability **(Fig.6.15)**. Further, they are much smaller in cross section when compared to recharge pits. Detailed design particulars of a recharge shaft are shown in **Fig.6.16**

6.2.3.3.2.1 Design Guidelines

i) Recharge shafts may be dug manually in non-caving strata. For construction of deeper shafts, drilling by direct rotary or reverse circulation may be required.

ii) The shafts may be about 2m in diameter at the bottom if manually dug. In case of drilled shafts, the diameter may not exceed 1m.

iii) The shaft should reach the permeable strata by penetrating the overlying low permeable layer, but need not necessarily touch the water table.

iv) Unlined shafts may be back-filled with an inverse filter, comprising boulders/cobbles at the bottom, followed by gravel and sand. The upper sand layer may be replaced periodically. Shafts getting clogged due to biotic growth are difficult to be revitalized and may have to be abandoned.

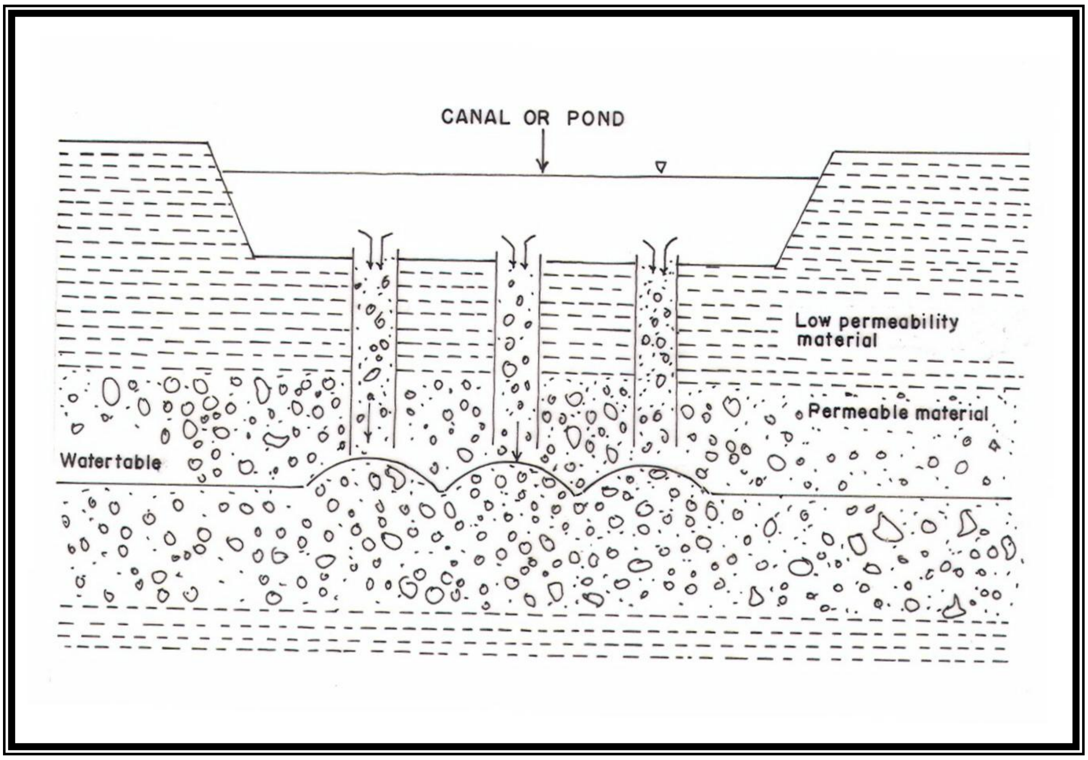


Fig.6.15 Schematics of Recharge Shafts

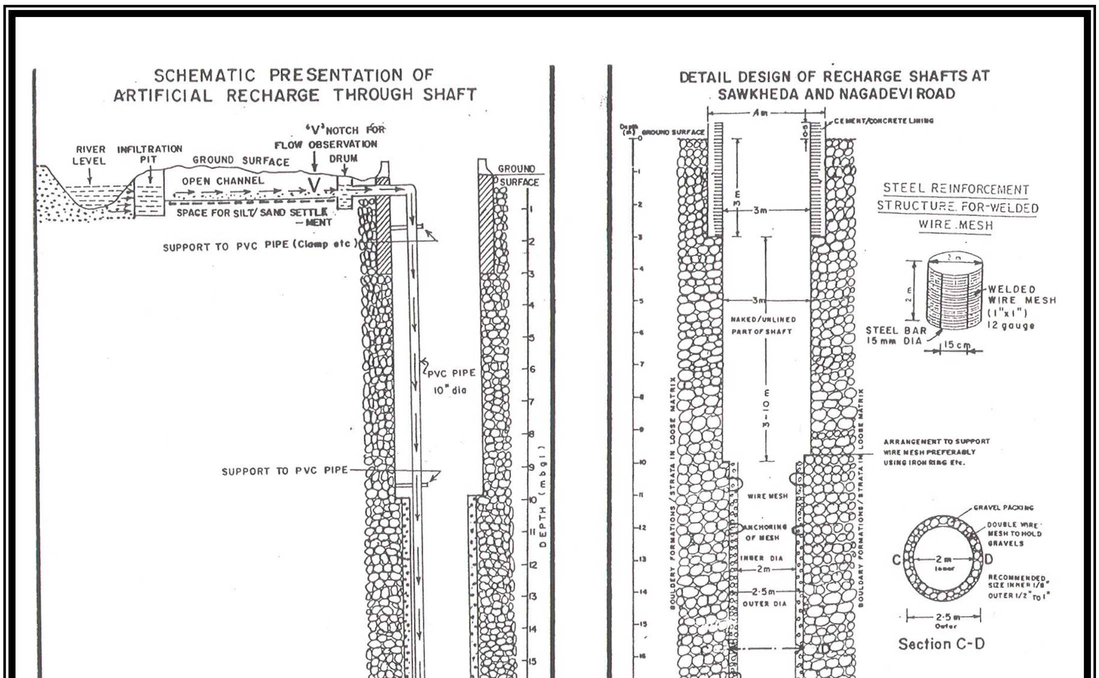
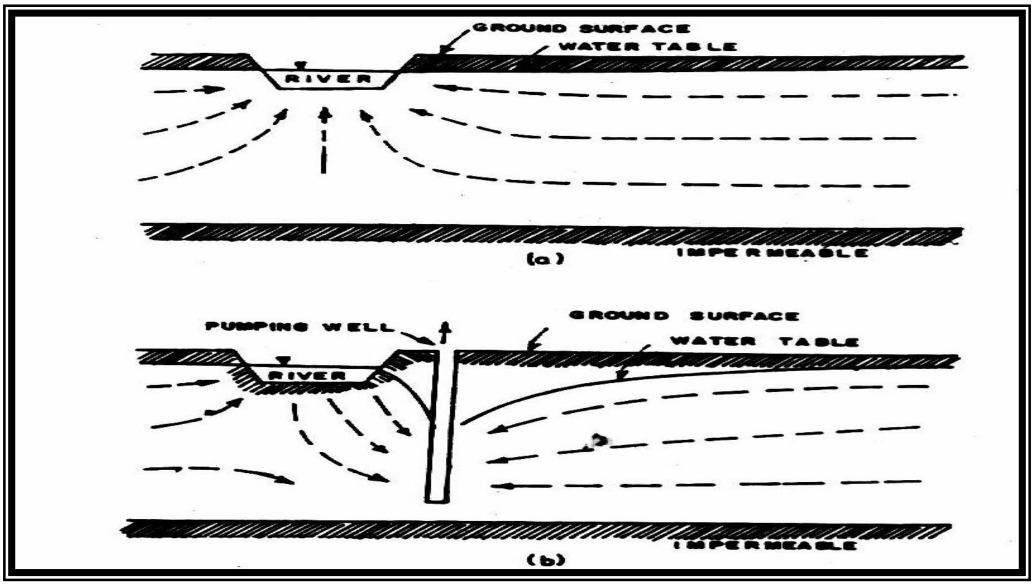


Fig.6.16 Design Particulars of a Typical Recharge Shaft

v) Deeper shafts constructed in caving strata may require lining or casing. In such cases, the shafts need not be completely back-filled and a reverse gravel-sand filter, a few meters thick, at the bottom of the shaft will suffice. In such cases, the water from the source may be fed through a conductor pipe reaching down to the filter pack.



vi) The source water should be made as silt-free as possible before letting into the shaft by providing suitable filters.

6.3 Indirect Methods

Indirect methods for artificial recharge to ground water does not involve direct supply of water for recharging aquifers, but aim at recharging aquifers through indirect means. The most common methods in this category are induced recharge from surface water sources and aquifer modification techniques.

6.3.1 Induced Recharge

Induced recharge involves pumping water from an aquifer, which is hydraulically connected with surface water to induce recharge to the ground water reservoir. Once hydraulic connection gets established by the interception of the cone of depression and the river recharge boundary, the surface water sources starts providing part of the pumping yield (**Fig.6.17).** Induced recharge, under favorable hydrogeological conditions, can be used for improving the quality of surface water resources due to its passage through the aquifer material. Collector wells and infiltration galleries, used for obtaining very large water supplies from riverbeds, lakebeds and waterlogged areas also function on the principle of induced recharge.

Fig.6.17 Principle of Induced Recharge through Pumping of Wells near a Stream a) Natural Flow Pattern b) Change in Flow Pattern Due to Pumping.

In hard rock areas, abandoned buried channels often provide favorable sites for the construction of structures for induced recharge. Check dams constructed in the river channel upstream of the channel bifurcation can help in high infiltration to the channel when wells located in the channels are pumped with high discharge for prolonged periods.

6.3.1.1 Design Guidelines

i) Quality of source water, hydraulic characteristics and thickness of aquifer material, distance of the pumping wells from the river and their pumping rates are the important factors controlling the design of schemes for induced recharge.
ii) For implementation of successful induced recharge schemes from stream channels, pumping wells should be selected at sites where water in the streams has sufficient velocity to prevent silt deposition.
iii) Dredging of channel bottom in the vicinity of the existing pumping wells may have to be carried our periodically to remove organic matter and impervious fine material from the beds of the channel.
iv) For wells constructed in unconfined alluvial strata for induced recharge, the lower one-third of the wells may be screened to have optimum drawdowns. In highly fractured consolidated rocks, dug wells penetrating the entire thickness of the aquifer should be constructed with lining above the water table zone and the curbing height well above the High Flow Level (HFL) of the stream.

6.3.2 Aquifer Modification Techniques

These techniques modify the aquifer characteristics to increase its capacity to store and transmit water through artificial means. The most important techniques under this category are bore blasting techniques and hydrofracturing techniques. Though they are yield augmentation techniques rather than artificial recharge structures, they are also being considered as artificial recharge structures owing to the resultant increase in the storage of ground water in the aquifers.

6.4 Combination Methods

Various combinations of surface and sub-surface recharge methods may be used in conjunction under favorable hydrogeological conditions for optimum recharge of ground water reservoirs. The selection of methods to be combined in such cases is site-specific. Commonly adopted combination methods include a) recharge basins with shafts, percolation ponds with recharge pits or shafts and induced recharge with wells tapping multiple aquifers permitting water to flow from upper to lower aquifer zones through the annular space between the walls and casing (connector wells) etc.

6.5 Ground Water Conservation Techniques

Ground water conservation techniques are intended to retain the ground water for longer periods in the basin/watershed by arresting the sub-surface flow. The known techniques of ground water conservation are a) Ground water dams / sub-surface dykes / Underground ‘Bandharas’ and b) Fracture sealing Cementation techniques.

6.5.1 Sub-Surface Dykes / Ground Water Dams / Underground '*Bandharas*'

A sub-surface dyke / ground water dam is a sub-surface barrier constructed across a stream channel for arresting/retarding the ground water flow and increase the ground water storage. At favorable locations, such dams can also be constructed not only across streams, but in large areas of the valley as well for conserving ground water. Schematics of a typical sub-surface dyke are shown in F**ig.6.18**

6.5.1.1 Site Characteristics and Design Guidelines

i) The primary objective of a sub-surface dyke is the creation of a subsurface storage reservoir with suitable recharge conditions and low seepage losses. Valley shapes and gradients are important considerations for site identification.

ii) Optimally, a valley should be well defined and wide with a very narrow outlet (bottle necked). This reduces the cost of the structure and makes it possible to have a comparatively large storage volume. This implies that the gradient of the valley floor should not be steep since that would reduce the storage volumes behind a dam of given height.

iii) The limitations on depth of underground construction deem that the unconfined aquifer should be within a shallow to moderate depth (down to 10 m bgl) and has a well-defined impermeable base layer. Such situations occur in hard rock areas and shallow alluvial riverine deposits.

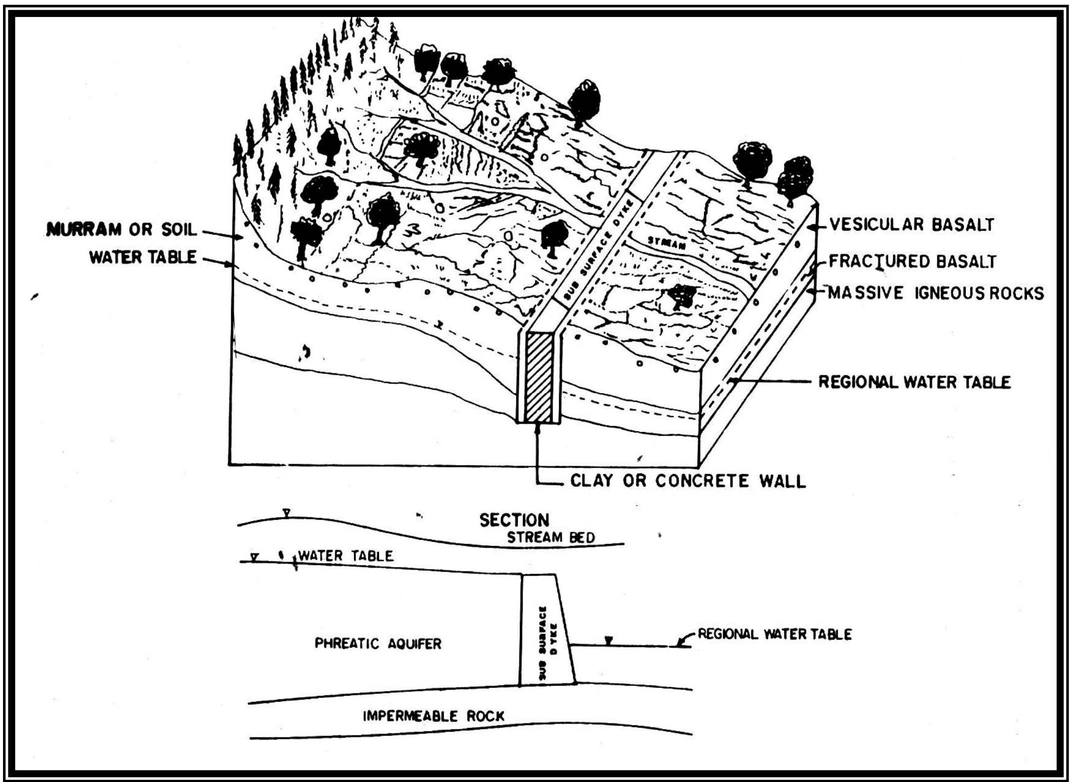


Fig.6.18 Schematics of a Subsurface Dyke in Basaltic Terrain.

iv) The dyke is ideally constructed across narrow ground water valleys, generally not exceeding 150 to 200 m in width. On the basis of a thorough study of a water table contour map of the area, a narrow ground water valley section where the flow lines tend to converge from up-gradient direction, usually coinciding with a surface drainage line should be identified. The requirement of narrow flow section is usually fulfilled in watersheds in hard rock terrain having rolling topography where relatively narrow depressions separate hard rock spurs.

v) The drainage valley across which the subsurface dyke is constructed should carry a seasonal stream that goes dry in winter and summer and the water table should be located well below the riverbed, preferably throughout the year (The stream should be preferably influent or may be effluent for a very limited period during rainy season). The valley section should preferably have a moderate gradient (less than 1%) so that the benefit spreads sufficiently in the up-gradient direction.

vi) The thickness of aquifer underlying the site should be adequate (more than 5 m) so that the quantity of ground water stored is commensurate with the effort and investment. Normally, in hard rock watersheds, the drainage courses have a limited thickness of alluvial deposits underlain by a weathered rock or fractured aquifer, which in turn passes into consolidated unaltered aquitard. This forms an ideal situation.

vii) The sub-surface dyke directly benefits the up-gradient area and hence should be located at a sufficient distance below the storage zone and areas benefiting from such recharge. This implies construction of ground water conservation structures in lower parts of the catchments but sufficiently upstream of watershed outlet.

viii) A sub-surface dyke may potentially deprive the downstream users the benefit of ground water seepage, which they received under a natural flow regime. Care should therefore be taken to see that a large number of users are not located immediately downstream and those affected are duly compensated through sharing of benefits. Care should also be taken to ensure that the water levels in the upstream side of the dyke are deep enough not to cause any water logging as a result of the dyke.

ix) For construction of ground water dam/ sub-surface dykes, a trench should be dug out across the ground water depression (streambed) from one bank to the other. In case of hilly terrain in hard rocks, the length of the trench generally may be less than 50 m. In more open terrain, the length may be usually less than 200 m but occasionally even more. It should be wide enough at the bottom to provide space for construction activity. In case of shallow trenches down to 5 m depth, the width at the bottom should be 2 m. For deeper trenches down to 15-20 m, deployment of mechanical equipment may be required. In such cases, width of 5 m at the bottom is recommended. The side slopes within alluvial strata should be 2:1 to make them stable. In case of more consolidated substrata, the slope could be steeper. The width at the surface should be planned accordingly.

x) The bottom of the trench should reach the base of the productive aquifer. In case of hard rock terrain, below a limited thickness of alluvial fill, weathered zone and underlying fractured aquifer may occur. The trench should be deep enough to penetrate both highly weathered and fractured strata. In case of more open terrain in consolidated or semi-consolidated strata, the alluvial thickness may be larger and the trench should end below the alluvial fill deposit. In order to minimize or avoid problem of dewatering during construction, the work should be taken up by the end of winter and completed well before the onset of rains, as water table is at lower elevation in this period.

xi) The cut-out dyke could be either of stone or brick masonry or an impermeable clay barrier. For ensuring total imperviousness, PVC sheets of 3000 PSI tearing strength and 400 to 600 gauge or low density polyethylene film of 200 gauge is also used to cover the cut out dyke faces. In the case of relatively shallow trenches within 5 m depth, where good impermeable clay is available within an economic distance (3 km), the cut-out dyke could be entirely be made of clay. In case good impermeable clay is not available, a stone masonry wall of 0.45 metre thickness or a brick wall of 0.25 m thickness may be constructed on a bed of concrete. Cement mortar of 1: 5 proportion and cement pointing on both faces is considered adequate. In the case of very long trenches, for economic considerations, it may be necessary to provide masonry wall only in the central part of dyke and clay dyke suitably augmented by tar felting, PVC sheet etc. on the sides.

xii) In case of clay dykes, the width should be between 1.5 and 2m depending on the quality of clay used. The construction should be in layers and each fresh layer should be watered and compacted by plain sheet or sheep foot rollers of 1 to 2 ton capacity. In absence of roller, the clay should be manually compacted by hand ramets. Where the core wall is a masonry structure, the remaining open trench should be back-filled by impermeable clay. The underground structures should be keyed into both the flanks of stream for one meter length to prevent leakage from sides.

xiii) The top of the underground structures should be located between 1 to 1.5 m below the streambed to permit overflow in high water table stage for flushing of salinity of ground water stored behind the dyke. The alignment of the dyke should be shown by fixing marker stones on the banks and whenever there is change of alignment in between. Before back-filling the sub-surface trench, piezometric tubes should be installed on both the faces of the dyke for measuring water levels. Such piezometers should be located in the central part, and in case of wider dykes at additional one or two locations.

xiv) Sites for construction of subsurface dykes have to be located in areas where there is a great scarcity of water during the summer months or where there is need for additional water for irrigation. Some emphasis also needs to be laid on finding sites where land ownership conditions would make constructions more feasible. Single ownership is ideal in the absence of which it has to be implemented on a cooperative basis.

6.6. Suitability of Artificial Recharge Structures under Combinations of Factors

Based on the discussions regarding various artificial recharge methods and structures, an attempt has been made to prescribe structures suitable for different slope categories, aquifer types and amount of precipitation received.

A matrix **(Table 6.7**) has been developed for easy visualization of these combinations and their possible variations. Three broad columns represent three distinct hydrogeologic settings normally encountered in nature. Each of these columns is split further to represent areas based on the adequacy of rainfall received. Areas receiving annual precipitation of less than 1000 mm and not having access to any surface inflow source are taken as areas with limited source water availability.

Four different slope categories have been considered in the matrix, representing runoff zone, piedmont zone, transition zone and storage zone. Indirectly, this classification also takes into account the status of ground water flow in the aquifer. Within each row the upper box represents the unconfined aquifers and the lower one represents for the leaky confined and confined aquifers.

The matrix thus tries to separate out 48 different combinations, all of which may not be relevant or suitable for effecting artificial recharge. Further, it is to be remembered that in a natural situation there are smooth transitions of conditions stipulated from one column or row to the other. Hence this tabulation will serve the purpose of broadly identifying recommended method or structure. The final choice should be governed by actual relevance of factors at a given site.

Table 6.7 Artificial Recharge Structures Suitable Under Combination of Different Topographic Slopes, Hydrogeologic Groups and Rainfall Distribution.

Topographic slope	**Hydrogeologic Group**						**Aquifer situation**
	Consolidated		**Semi Consolidated**		**Un-consolidated**		
	Rainfall						**Unconfined /Confined**
	Adequate	**Limited**	**Adequate**	**Limited**	**Adequate**	**Limited**	
1	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Steep Slope (20 - 10%) Runoff zone	Bench Terrace Contour Trench	Gully Plug	Bench Terrace Contour Trench	Gully Plug	-	-	Unconfined
Moderate Slope (10 to 5%) Piedmont zone	Bench Terrace Contour Trench Gravity Head Recharge Well*	*Nalah* Bunds Contour Bunding Percolation Tanks *Nalah* Trench Gravity Head Recharge well* Bore Blasting	Bench Terrace Contour Trench Gravity Head Recharge Well*	*Nalah* Bund Contour Bund Percolation Tanks *Nalah* Trench Gravity Head Recharge Well	Ditch & Furrow Recharge Basin Pits & Shafts Contour Trench Gravity Head Recharge Well	Recharge Basin Pits* & Shafts* Contour Trench Gravity Head Recharge Well	Unconfined
	Deep Gravity Head Recharge Well Hydro fracturing Fracture Seal Cementation		Injection Well* Recharge Shafts*				Confined

Topographic slope	**Hydrogeologic Group**						**Aquifer situation**
	Consolidated		**Semi Consolidated**		**Un-consolidated**		
	Rainfall						**Unconfined Confined**
1	**Adequate**	**Limited**	**Adequate**	**Limited**	**Adequate**	**Limited**	
	2	**3**	**4**	**5**	**6**	**7**	**8**
Moderate to Gentle Slope (2 to 5%) Transition zone	*Nalah* Bunds Contour Bunding Percolation Tanks Recharge Pits Canal Irrigation* Induced Recharge Ground Water Dams Fracture Seal cementation	*Nalah* Bunds Contour Bunding Percolation Tanks Recharge Pits Ground Water Dams Canal Irrigation*	Recharge Basin Canal Irrigation* Induced recharge Stream Channel Modification Recharge Pits	Recharge Pits Stream Channel Modification	Flooding Recharge Basin Stream Channel Modification Induced Recharge Gravity Head Recharge Well* Canal Irrigation*	Stream Channel Modification Gravity Head Recharge Well* Ditch & Furrow Recharge Basin* Recharge Shaft* Ground Water Dam (In shallow alluvium)	Unconfined
	Gravity Head Recharge Well* Hydrofracturing Deep Fracture Seal Cementation		Recharge Shaft* Gravity Head Recharge Wells* Injection Wells* Hydrofracturing		Recharge Shafts* Gravity Head Recharge Wells* Injection Wells*		Confined
Gentle Slope (< 2%) Storage Zone	Surface Irrigation Recharge Basin Recharge Pits Gravity Head Recharge Wells	Induced Recharge Recharge Basin Recharge Pits Gravity Head Recharge Wells	Recharge Pits	Flooding Canal Irrigation* Induced Recharge Surface Spreading Infiltration Gallery	Flooding* Surface Spreading* Infiltration Gallery		Unconfined
	Gravity Head Recharge Wells (On Lineaments or their intersections)		Injection Wells		Injection Wells Connector Wells		

Note: Rainfall is considered 'adequate' if annual precipitation is more than 1000 mm.
* Indicate availability of source water supply through canals, trans-basin transfer or treated wastewater.
(Modified After: Manual on Artificial Recharge of Ground Water, CGWB (1994).

7. ROOF TOP RAINWATER HARVESTING

Drinking water supply in urban areas is mostly from surface sources like natural or impounded reservoirs and from ground water sources. As the population density and usage levels are comparatively high in urban areas, Government agencies construct, operate and maintain huge surface water dams and reservoirs for meeting their water demands. These sources are planned and constructed to take care of the water requirements of the population throughout the year. Ground water is in use in areas where the surface water supplies are either not reaching or are not adequate.

In most of the rural areas, ground water is the major source of drinking water. In earlier days, open wells and ponds that belonged to the community were the source of drinking water supply. With the advent of bore well technology and progress made in rural electrification, the scenario of rural water supply has considerably changed. The traditional methods and practices have given way to hand pumps and power pump schemes. Government organisations have given priority to provide protected water supply to the villages through rural water supply schemes. Bore wells are drilled and water from over-head tanks is distributed through supply mains. Statistics reveal that more than 85% of rural water supply is from the ground water sources at present.

Indiscriminate exploitation of ground water and the decline in ground water levels have rendered many bore wells dry either seasonally or through out the year. To overcome such a situation, bore wells and tube wells are now being drilled to greater depths, often tapping ground water from deep aquifers hitherto considered ‘static’. Discharge of untreated effluents into surface water streams and lakes by industries has resulted not only in contaminating the surface water resources, but also the ground water bodies. In coastal areas, over exploitation of ground water has resulted in seawater intrusion, rendering ground water sources saline in some areas.

Identification and promotion of simple, reliable and environmental friendly technologies for augmentation of ground water resources are necessary to overcome the above problems and to ensure the long-term sustainability of our precious ground water resources. Reviving the traditional practices of rainwater harvesting along scientific lines can go a long way in preventing a serious water crisis in the major part of our country in the years to come.

7.1 Concept of Roof Top Rainwater Harvesting

The concept of rainwater harvesting involves ‘tapping the rainwater where it falls’. A major portion of rainwater that falls on the earth’s surface runs off into streams and rivers and finally into the sea. An average of 8-12 percent of the total rainfall recharge only is considered to recharge the aquifers. The technique of rainwater harvesting involves collecting the rain from localized catchment surfaces such as roofs, plain / sloping surfaces etc., either for direct use or to augment the ground water resources depending on local conditions. Construction of small barriers across small streams to check and store the running water also can be considered as water harvesting.

Among various techniques of water harvesting, harvesting water from roof tops needs special attention because of the following advantages:

a) Roof top rainwater harvesting is one of the appropriate options for augmenting ground water recharge/ storage in urban areas where natural recharge is considerably reduced due to increased urban activities and not much land is available for implementing any other artificial recharge measure. Roof top rainwater harvesting can supplement the domestic requirements in rural areas as well.
b) Rainwater runoff which otherwise flows through sewers and storm drains and is wasted, can be harvested and utilized.
c) Rainwater is bacteriologically safe, free from organic matter and is soft in nature.
d) It helps in reducing the frequent drainage congestion and flooding during heavy rains in urban areas where availability of open surfaces is limited and surface runoff is quite high.
e) It improves the quality of ground water through dilution.
f) The harnessed rainwater can be utilized at the time of need.
g) The structures required for harvesting rainwater are simple, economical and eco-friendly.
h) Roof catchments are relatively cleaner and free from contamination compared to the ground level catchments.
i) Losses from roof catchments are much less when compared to other catchments.

Collection of rainwater from roof tops for domestic needs is popular in some parts of India. The simplest method of roof top rainwater harvesting is the collection of rainwater in a large pot/vessel kept beneath the edge of the roof. The water thus collected can meet the immediate domestic needs. Tanks made of iron sheets, cement or bricks can also be used for storing water. In this method, water is collected from roofs using drain pipes/gutters fixed to roof edge.

Though the practice of roof top rainwater harvesting is an age-old one, systematic collection and storage of water to meet the drinking water needs has become popular only recently. The popularity of this practice is limited by the costs involved in collection of water by gutters/pipes and its storage in underground tanks made of iron or brick. Use of Ferro-cement technology in construction and maintenance of storage tanks has become popular in recent years as the strength and durability of ferro-cement structures have been found to make the schemes cost-effective.

Rainwater harvesting practices vary widely in size, type of construction material used and methods of collection and storage. Easy availability of know-how on systematic and economic methods of construction will encourage the user households to adopt this practice. There is also a need for creating awareness and for development of simple techniques of construction/fabrication of the components of rainwater harvesting system for popularising this technique as a potential alternative source of drinking water, at least for part of the year.

7.2 Components of Roof Top Rainwater Harvesting System

In a typical domestic roof top rainwater harvesting system, rainwater from the roof is collected in a storage vessel or tank for use during periods of scarcity. Such systems are usually designed to support the drinking and cooking needs of the family and comprise a roof, a storage tank and guttering to transport the water from the roof to the storage tank. In addition, a first flush system to divert the dirty water, which contains debris, collected on the roof during non-rainy periods and a filter unit to remove debris and contaminants before water enters the storage tank are also provided. Therefore, a typical Roof top Rainwater Harvesting System (**Fig.7.1**) comprises following components:

- Roof catchment.
- Drain pipes
- Gutters
- Down pipe
- First flush pipe.
- Filter unit
- Storage tank.
- Collection sump.
- Pump unit

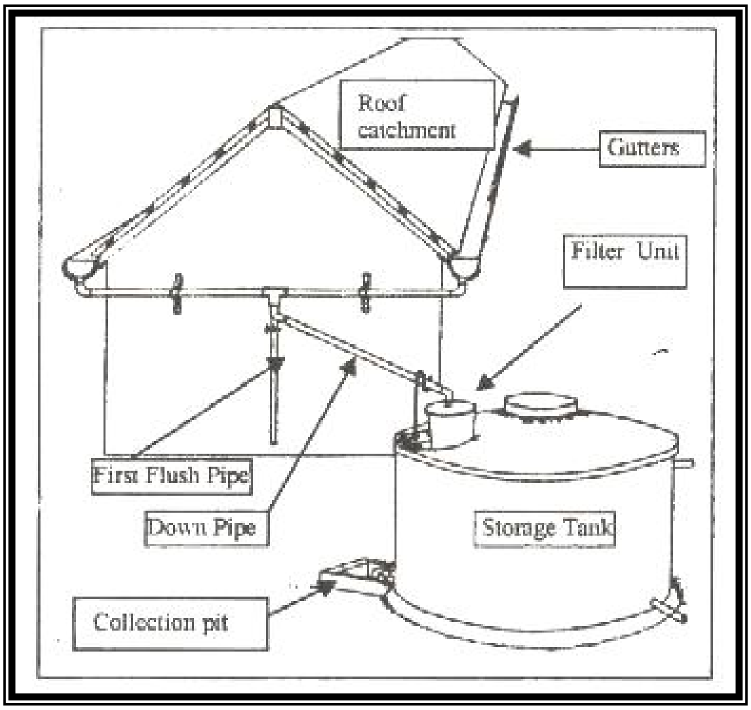


Fig. 7.1 A Typical Rainwater Harvesting System

Among the above components, storage tank and filter unit are the most expensive and critical components. The capacity of the storage tank determines the cost of the system as well as its reliability for assured water supply whereas the filter unit assures the quality of the supplied water. Brief descriptions of each of the components are given below:

7.2.1 Roof Catchment

The roof of the house is used as the catchment for collecting the rainwater. The style, construction and material of the roof determine its suitability as a catchment. Roofs made of corrugated iron sheet, asbestos sheet, tiles or concrete can be utilized as such for harvesting rainwater. Thatched roofs, on the other hand, are not suitable as pieces of roof material may be carried by water and may also impart some colour to water.

7.2.2 Drain Pipes

The drain pipes of suitable size, made of PVC / Stoneware are provided in RCC buildings to drain off the roof top water to the storm drains. They are provided as per the building code requirements.

7.2.3 Gutters

Gutters are channels fixed to the edges of roof all around to collect and transport the rainwater from the roof to the storage tank. Gutters can be prepared in rectangular shapes (**Fig.7.2**) and semi-circular (**Fig.7.3**).

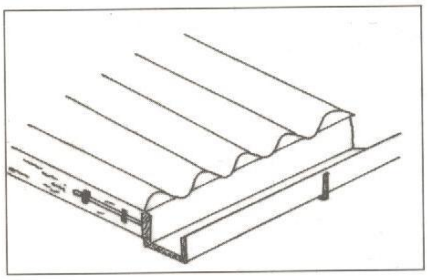


Fig. 7.2 Rectangular Gutter

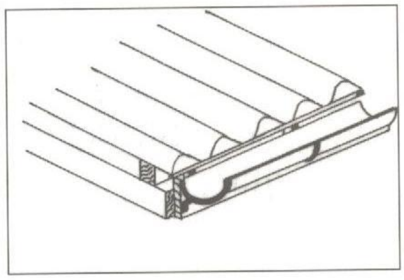


Fig. 7.3 Semi-circular Gutter

Gutters are channels made of either plain Galvanized Iron sheets or cut PVC pipes or split Bamboo. These channels are fixed to the roof ends to divert the rainwater into the storage tank. Semi-circular or rectangular shaped channels can be made using GI sheet. Cut PVC pipes and Bamboos will be semi-circular in shape. These channels are made at the site of construction and fixed to the roof by using mild steel supports. As the preparation of gutters from GI sheet involves cutting and bending the sheet to the required size and shape, certain amount of skill is required. Gutters from PVC pipes or bamboos are easily made. Use of locally available materials reduces the overall cost of the system.

7.2.4 Down Pipe

Down pipe is the pipe that carries the rainwater from the gutters to the storage tank. Down pipe is joined with the gutters at one end, whereas the other end is connected to the filter unit of the storage tank as shown below (**Fig.7.4**). PVC or GI pipes of 50 mm to 75 mm (2 inch to 3 inch) diameter are commonly used for down pipe. In the case of RCC buildings, drain pipes themselves serve as down pipes. They have to be connected to a pipe to carry water to the storage tank.

The down pipe and first flush pipe can be of either GI or PVC material of diameter 7.5 cm. Joining of pipes will be easy if both are of same material.

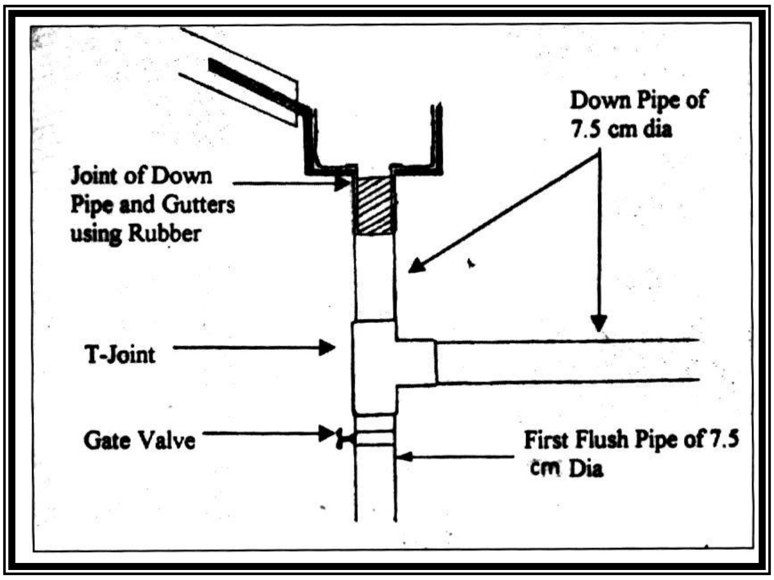


Fig. 7.4 Down Pipe

The orientation and arrangement of the down pipe depends on relative locations of tank and roof. The shape of the roof and type of the roof also determine the arrangement of down pipes. The most common type of down pipe arrangement is shown in **Fig.7.5.**

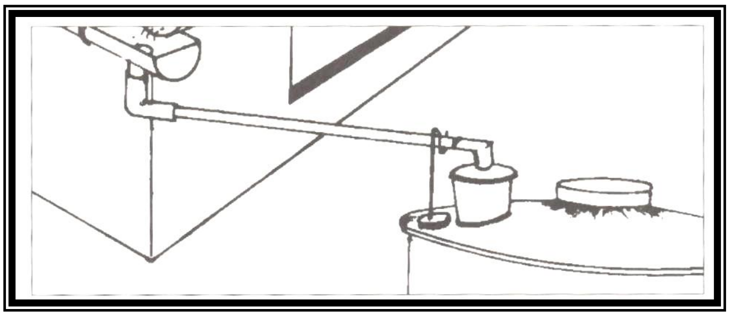


Fig. 7.5 Most Common Arrangement of Down Pipe

7.2.5 First Flush Pipe

Debris, dirt and dust collect on the roofs during non-rainy periods. When the first rains arrive, these unwanted materials will be washed into the storage tank. This causes contamination of water collected in the storage tank, rendering it unfit for drinking and cooking purposes.

A first flush system can be incorporated in the roof top rainwater harvesting systems to dispose off the 'first flush' water so that is does not enter the tank. There are two such simple systems. One is based on a simple, manually operated arrangement, whereby the down pipe is moved away from the tank inlet and replaced again once the first flush water has been disposed. In another semi-automatic system, a separate vertical pipe is fixed to the down pipe with a valve provided below the 'T' junction (**Fig.7.6**).

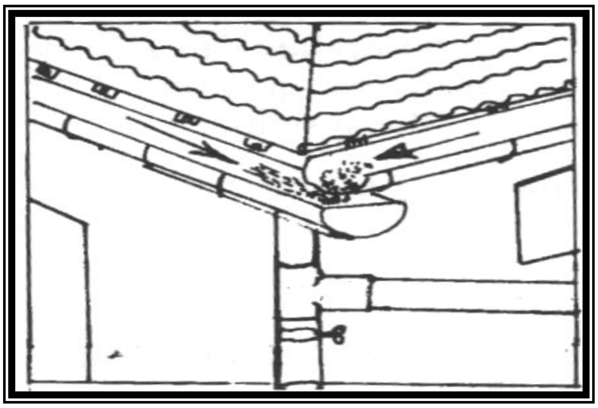


Fig.7.6 First Flush Pipe

After the first rain is washed out through first flush pipe, the valve is closed to allow the water to enter the down pipe and reach the storage tank.

7.2.6 Filtration of Water

7.2.6.1 Process of Filtration

Filtration forms the most important process in the purification of water. It usually involves allowing water to pass through a filter media e.g. sand. Filtration essentially involves removal of suspended and colloidal impurities present in water. Depending on the type of filtration, the chemical characteristics of water may be altered and the bacterial content may be considerably reduced. These effects take place due to various processes such as mechanical straining, sedimentation, biological metabolism and electrolytic changes.

Mechanical straining involves removal of suspended particles, which are unable to pass through the voids of the filter media. Sedimentation of particles of impurities occurs in the voids between sand grains in the filter unit. Such particles also adhere to the sand grains due to i) presence of a gelatinous film or coating developed on sand grains by previously trapped bacteria and colloidal matter and ii) physical attraction between particles. Biological metabolism in filter units involves the formation of a zoological jelly or film containing large colonies of bacteria around the sand grains, which feed on the organic impurities in the water and convert them into harmless compounds by complex biochemical reactions. Electrolytic changes involve the neutralization of ionic charges of particles of suspended and dissolved impurities when they come into contact with sand particles having opposite charge. When this happens, they neutralize each other, which ultimately results in the alteration of chemical characteristics of water.

7.2.6.2 Filter Sand

The sand being used for filter in roof top rainwater harvesting systems should be free from clay, loam, vegetable matter, organic impurities etc. and should also be uniform in nature and grain size. In place of sand, ‘anthrafilt’, made from anthracite (stone-coal) can also be used as filter medium. This material is found to possess many advantages such as low cost, high rate of infiltration and better efficiency. However, as sand is readily available almost everywhere, the usual practice is to use it as filter medium.

7.2.6.3 Classification of Filters

Filters are classified into two main categories *viz.* Slow sand filters and Rapid sand filters based on the rate of filtration. They have also been categorised as gravity filters and pressure filters depending on the forces aiding the process of filtration. Based on these, filters can be classified into i) Slow Sand Filters, ii) Rapid Sand Filters (gravity type) and iii) Pressure Filters.

i) Slow Sand Filters

A typical slow sand filter (**Fig.7.7 & 7.8**) consists of an enclosure tank, under-drainage system, base material, filter media (sand) and appurtenances.

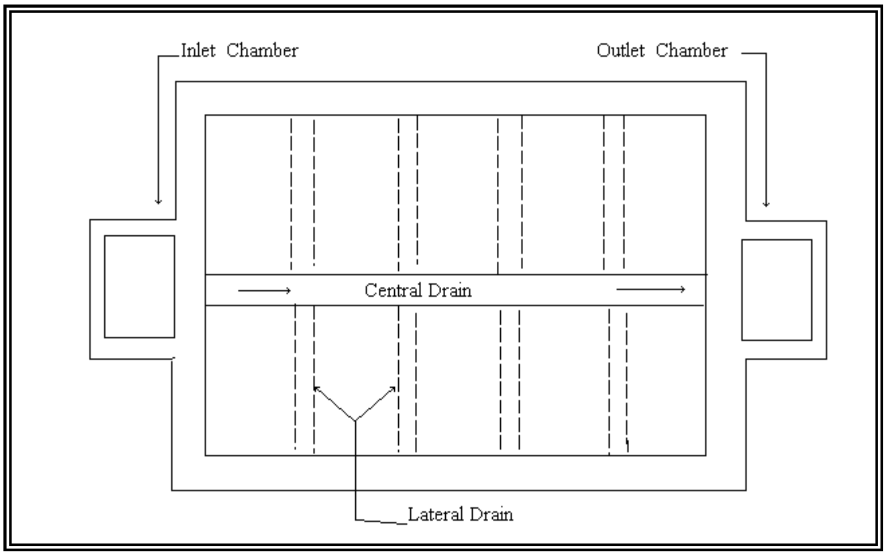


Fig.7.7 Plan of a Slow Sand Filter

One of the major disadvantages of a slow sand filter is the requirement of considerable space for its installation. This makes it uneconomical for places where land is not available, or it is costly. This has led engineers and scientists to find out means for increasing the rate of filtration. It is observed that rate of filtration can be increased either by increasing the grain size of sand used as filter media so that frictional resistance to water passing through it is reduced, or by allowing water to pass under pressure through the filter media.

ii. Rapid Sand Filters (Gravity type)

Rapid sand filters have been developed to achieve increased filtration rates by increasing the grain size of the filter media. These types of filters are preferred for rainwater harvesting schemes implemented over larger areas

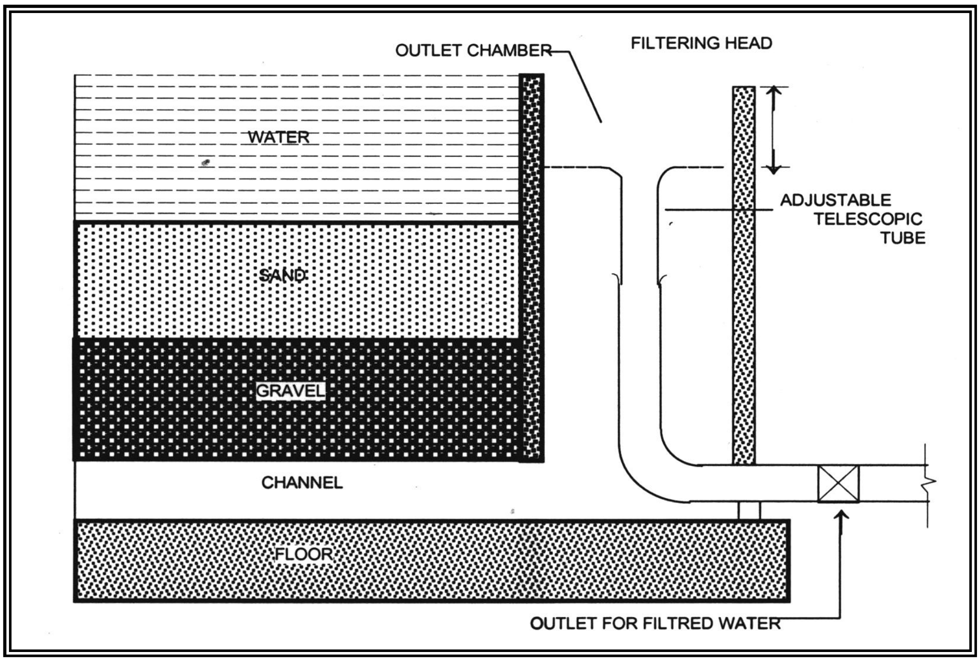


Fig.7.8 Cross section of a Slow Sand Filter

Essential Parts

The layout of a typical rapid sand filter (gravity type) is shown in **Fig. 7.9**.

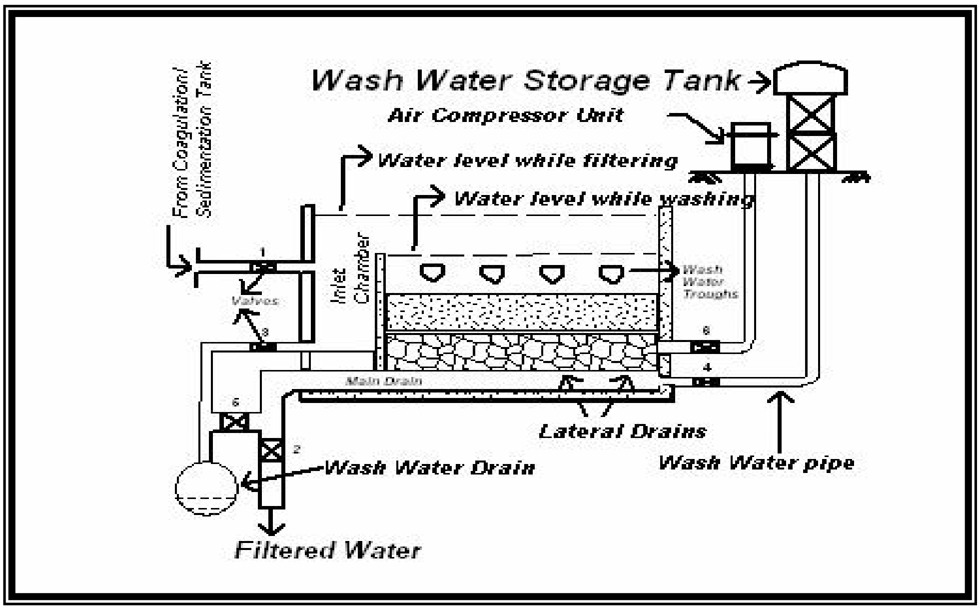


Fig. 7.9 Lay out of a Typical Rapid Sand Filter

The essential parts are the same as those of the slow sand filter, *viz.* Enclosure tank, Under-drainage system, Base material, Filter media (sand) and appurtenances.

Enclosure Tank: A watertight tank, either of masonry or concrete is used. The sides and floor are coated with waterproof material. Depth of the tank is about 2.50 to 3.0 m. The surface area of a unit of rapid sand filter varies from 10 to 50 m^2. The units are arranged in series.

Under-Drainage System: There are various types of under-drainage systems, most of which are patented by the manufacturers. Two of the common types of under-drainage systems are Perforated Pipe Systems and Pipe and Strainer Systems.

In a Perforated Pipe System, a number of lateral drains are attached to a central drain or manifold (**Fig. 7.10**). The drains are usually made of cast iron. The lateral drains are placed at distances of 15 to 30 cm. They are also provided with holes at the bottom in such a way that the holes are at 30° with the vertical (**Fig. 7.11**). The holes are about 10 mm in diameter and are sometimes staggered on either side instead of being continuous. The holes are usually drilled with a centre to centre distance of 75 to 200 mm. Brass bushings are sometimes inserted in the holes to prevent rusting of surface of holes. Concrete blocks, 40 to 50 mm thick, are placed on the floor of the filter for supporting lateral drains.

Perforated Pipe Systems are economical and simple in operation. They, however, require large quantities of about 700 litres of wash water per square metre of filter area for washing purpose. The wash water for this 'high velocity wash' is provided from a wash water overhead tank as shown in Fig. 7.9.

Pipe and Strainer Systems also have a central drain or manifold with lateral drains attached on either side (**Fig.7.12**). However, unlike in Perforated Pipe System, strainers in this system are placed on lateral drain. A strainer (**Fig.7.13**) is a small pipe of brass, closed at the top and having perforations on its surface. They are either screwed or fixed on the lateral drains. In some cases, the strainers are fixed on the central drain as well. Two lateral drains / strainers are normally placed at a distance of 15 cm from each other. It is desirable to place all the strainers at the same elevation.

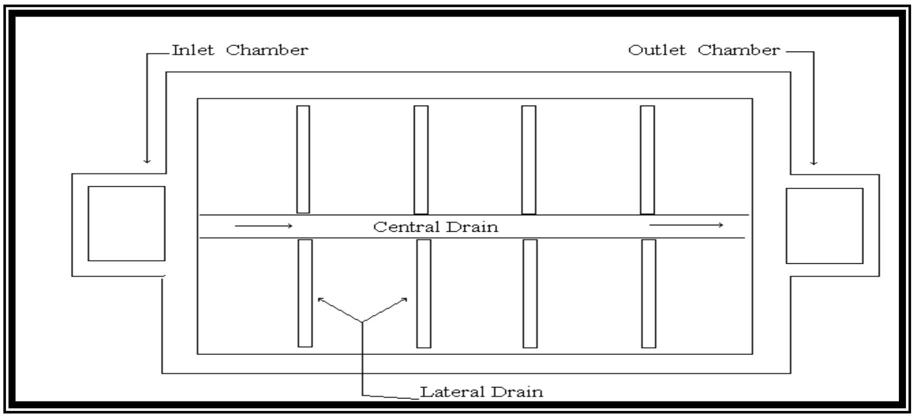


Fig.7.10 Plan of Under-drainage System in a Rapid Sand Filter

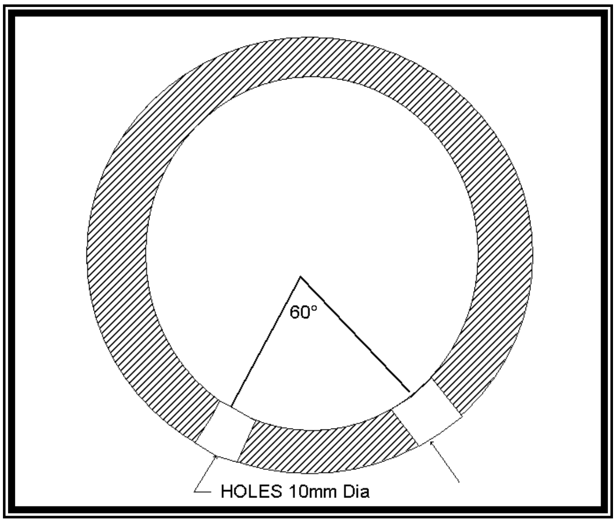


Fig.7.11 Plan of a Perforated Lateral Drain

In Pipe and Strainer Systems, compressed air is used for cleaning the filter, which results in saving of wash water. It still requires about 250 litres of wash water per minute per square metre of filter area for washing purpose. This process of washing is known as ‘low velocity wash’.

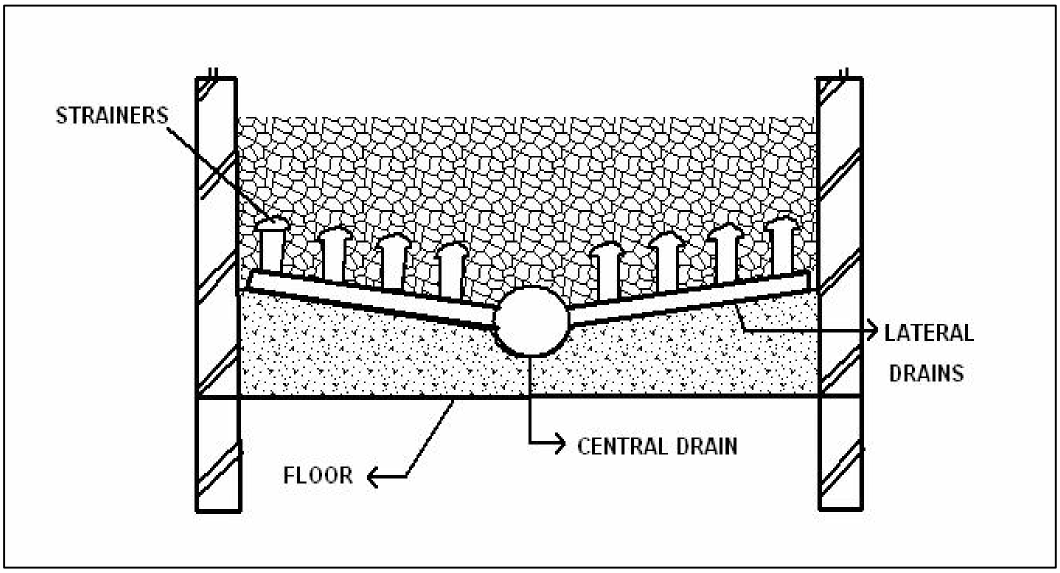


Fig.7.12 Cross Section of Pipe and Strainer System

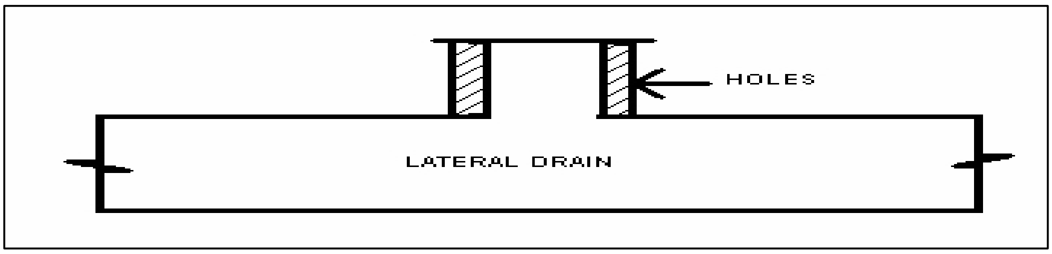


Fig. 7.13 Cross Section of a Strainer

Under-Drainage System: The under-drainage system in Pipe and Strainer Systems are designed in such a way that the ratio of length of lateral drain to its diameter does not exceed 20 and the cross-sectional area of the central drain is about twice the cross-sectional area of the lateral drains. Care should be taken to ensure that the total cross-sectional area of the perforations is about 0.20 percent of the total filter area and the cross-sectional area of a lateral is about two to four times the total cross-sectional area of the perforations in it.

Base Material: Gravel is used as the base material, which is placed on the top of the under-drainage system. The gravel being used should be clean and free from clay, dust, silt and organic matter and should be durable, hard, round and strong. The thickness of the layer of base material varies from 45 to 60 cm. It is usually graded and laid in layers of 15 cm. The size of the gravel increases from top to bottom.

Filter Media (Sand): A layer of coarse sand, ranging in thickness from 60 to 90 cm is placed above the gravel. The effective size of sand used varies from 0.35 to 0.60 mm and its uniformity coefficient is between 1.20 and 1.70, ensuring increased rate of filtration on account of the increase in the void space between particles when compared to slow sand filters.

Appurtenances: In addition to the usual appurtenances required in the case of slow sand filters, the following special devices are to be provided in case of rapid sand filters:

> **i) Air Compressors:** The agitation of sand grains during washing of filters is achieved by compressed air, water jet or mechanical rakes. When compressed air is to be used, an air compressor of suitable capacity should be installed. Generally, it should have the capacity of supplying compressed air at the rate of 0.60 to 0.80 m^3 per minute per square metre of filter area for 5 minutes. The pressure of compressed air should be sufficient to overcome frictional resistance in air pipes and water above the air distribution system. The compressed air may be supplied through laterals or through a dedicated pipe system.
>
> **ii) Wash water Troughs:** The water used for washing the filters is collected in wash water toughs or gutters, which are placed above sand bed level. The troughs may be of cast-iron, concrete, steel or wrought iron. They are placed at a distance of 130 to 180 cm from edge to edge. The bottom of the trough is about 45 to 75 cm above the sand bed. For efficient working, the troughs should be large enough and should be laid at a suitable slope.
>
> **iii) Rate Control Devices:** There are various devices to control the rate of flow of water, which may be fitted at the outlet of the filter. They work on the principle of Venturi meter.

Cleaning of Filters: When water passes through the filter, there is head loss due to frictional resistance. The difference in water levels in the filter and in the outlet pipe provides an idea about the loss of head **(Fig. 7.14)**. When the filter is clean, the head loss is negligible (about 15 to 30 cm). It goes on increasing with time until the frictional resistance offered by the filter media exceeds the static head of water above the sand bed, due to the deposition of suspended matter in the top layer. The lower part of the filter media then acts more or less like a vacuum and water is sucked through the filter media rather than getting filtered by passing through it. The fall of liquid level in the piezometric tubes below the centre line of the under-drainage system indicates negative head. The negative head thus formed tends to release dissolved air and other gases present in the water, which sticks to the sand grains, seriously impairing the functioning of the filter. In case of rapid sand filters, the permissible loss of head is about 3.0 to 3.5 m and the permissible negative head is about 120 cm. The filter is to be cleaned when this limit is reached.

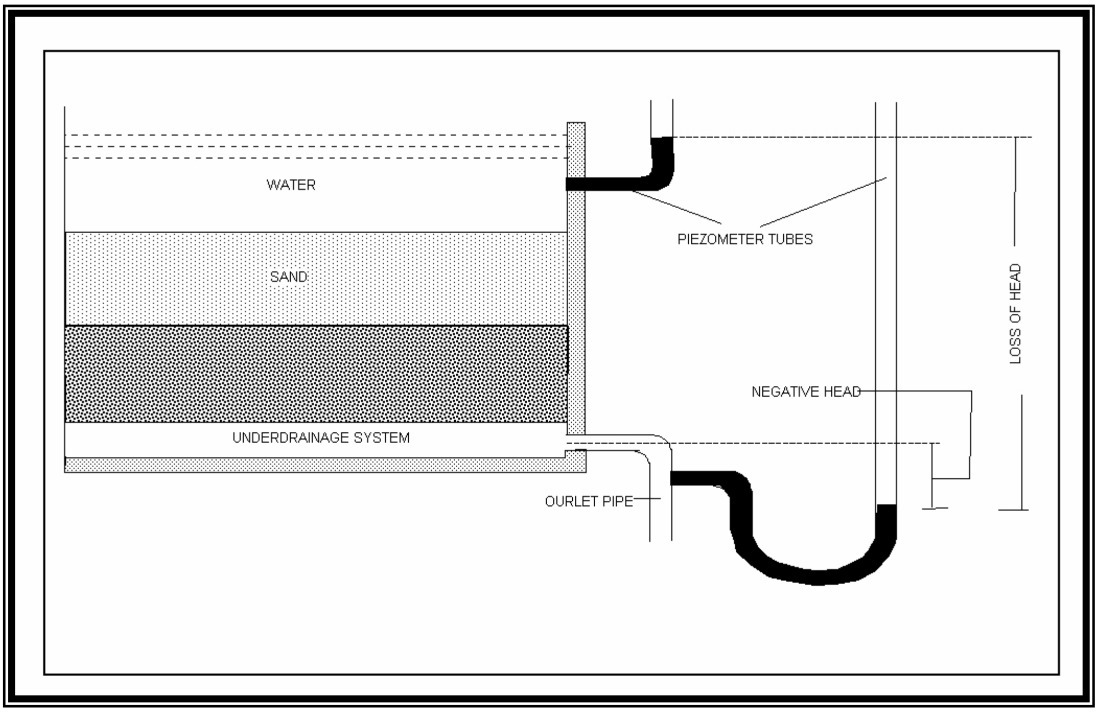


Fig. 7.14 Loss of Head and Negative Head

Operational Problems: The most commonly encountered operational problems of rapid sand filters are i) Mud balls and ii) Cracking of filters.

> i) Mud balls: Mud balls are normally formed near the top of the filter media, though it is not uncommon to have them distributed throughout the filter. These are formed as a consequence of insufficient washing of sand grains resulting in gelatinous film formed during filtration not being separated from the sand grains during washing. They range in size from 25 to 50 mm and seriously impair the functioning of the filter. Mud balls are broken and removed using rakes or other suitable instruments. Cleaning of filters with water at high velocity will also help in getting rid of the problem.
>
> ii) Cracking of Filters**:** Shrinkage of fine material contained in the top layer of the filter may result in the formation of cracks in the filter. Such cracks are normally prominent near wall junctions. The damaged portion of the filter media has to be replaced to get rid of this problem.

Rate of Filtration: The main advantage of a rapid sand filter is its high rate of filtration, which is about 3000 to 6000 litres per hour per square metre of filter area. The high rate of filtration results in considerable saving of space for the installation of filter.

Efficiency of Rapid Sand Filters

i) Bacterial Load: Rapid sand filters are less effective in the removal of bacterial load when compared to slow sand filters. They are expected to remove between 80 and 90 percent of bacterial load present in water.
ii) Colour: These filters are highly efficient in removal of colour and the intensity of colour can be brought down below 10 on cobalt scale.
iii) Turbidity: Rapid sand filters are capable of removing turbidity to the extent of 35 to 40 ppm. As water entering rapid sand filter is invariably given treatment in coagulation/sedimentation tank, it normally has low turbidity to begin with. The turbidity still present in water is easily brought down to permissible limits by rapid sand filters.

Problem

Find the area of rapid sand filters required for a town having a population of 80,000 with an average demand of 200 litres per head per day.

Solution:

Maximum daily demand (Litres)
(assumed as 1.5 times average demand) = 80,000 x 200 x 1.50

= 240, 00,000

Assuming rate of filtration as 5000 litres/hour/m^2 of filter area,
Area of filter required (m^2) = 24000000/ (5000 x 24)
= 200

Assuming size of individual filter units as 8.00 m x 5.00 m, 6 such units, including one as stand-by will be sufficient to cater to the water requirements of the town.

Comparison Between Slow and Rapid Sand Filters

Slow and rapid sand filters have their own advantages and disadvantages. A comparative analysis of their relative merits and de-merits are shown in **Table 7.1.**

iii. Pressure Filters

In pressure filters, which are more or less similar to rapid sand filters, the filter is enclosed in a container, through which water passes under pressures greater than the atmospheric pressure. This pressure, varying from 3 to 7 kg/cm^2, can be developed by pumping.

Table.7.1 Comparative Analysis of Merits and De-merits of Slow and Rapid Sand Filters

Sl. No	Particulars	Slow Sand Filters	Rapid Sand Filters (gravity Type)
1	Base material (Gravel)	Size : 3 to 65 mm Depth : 30 to 75 cm	Size : 3 to 40 mm Depth : 60 to 90 cm
2	Pre-treatment (Sedimentation / Coagulation)	Not required	Essential
3	Space require-ments for installation	Large	Small
4	Construction	Simple	Complicated
5	economy	High initial cost of land and material	Cheap and economical
6	Efficiency	Very efficient in the removal of bacteria but less efficient in the removal of colour and turbidity	Less efficient in the removal of bacteria but more efficient in the removal of colour and turbidity.
7	Filter media (sand)	Effective size: 0.2 to 0.3mm Uniformity coefficient: 2 -3	Effective size: 0.35 to 0.60mm Uniformity coefficient: 1.2 – 1.7
8	Flexibility	Not flexible for meeting variations in demand	Quite flexible for reasonable fluctuations in demand.
9	Loss of head	15 to 75 mm	3 to 3.5 m
10	Method of cleaning	Scraping of top layer of 15 to 25 mm thickness. Long and laborious method	Agitation and back washing with or without the help of compressed air. Short and speedy method.
11	Period of cleaning	1 to 3 months	2 to 3 days
12	Rate of filtration	100 to 200 lph/m^2 of filter area	3000 to 6000 lph/m^2 of filter area
13	Supervision	Not essential	Essential
14	Suitability	Filter can be constructed using local labour and material. Suitable for small town and villages where land is cheap.	Suitable for big cities where land cost is high and variation in demand of water is considerable.

Pressure filters are closed steel cylinders, either riveted or welded, with manholes provided at the top for inspection. They may be or horizontal or vertical types. The diameter of pressure filters varies from 3.50 to 8.0 mm.

Water mixed with the coagulant is directly pumped into the pressure filter through the inlet and flocculation takes place inside the filter itself. In normal working condition, all valves except those for raw water and filtered water are kept closed. The rate of filtration is higher than that of rapid sand filters and is normally in the range of 6000 to 15000 litre per hour per square metre of filter area. They are found to be less efficient when compared to rapid sand filters.

Pressure filters can be cleaned by agitating sand grains using compressed air. The valves for raw water and filtered water are closed and those for wash water and wash water drain are kept open during the process. Frequency of cleaning is more for pressure filters when compared to rapid sand filters. Automatic pressure filters are also available now in the market in which washing of the filter is done automatically at pre-determined intervals of time or loss of head.

Pressure filters are not suitable for public water supply projects, but can be installed for colonies of houses, industrial plants, private estates, swimming pools, railway stations etc.

d) Double Filtration

Double filtration, involving filtration of water twice, can be used for better results. This can be achieved either by allowing water to pass through two or more slow or rapid sand filters arranged in series, or by passing the water through a raid sand filter before sending it to a slow sand filter. In practice, the latter alternative is most commonly adopted to increase the filtration rate of an existing slow sand filter. The rapid sand filter used in such a way is known as a ‘roughing’ filter. Coarse materials are used in the construction of roughing filters, resulting in rates of filtration as high as 7000 litre per hour per square metre of filter area. They generally do not necessarily require water pre-treated with coagulant. Double filtration is usually adopted at places where there are constraints of land availability for construction of slow sand filters. Installation of roughing filters practically doubles the capacity of slow sand filters.

Screen filters or micro filters, which are readily available in the market, can also be used for filtration. Silt and other contaminants present in the roof top rainwater can be removed efficiently using these filters. The size of the filter can be decided based upon roof top area and the rainfall amount.

Locally fabricated filters consisting of buckets or other containers filled with filter media such as coarse sand, charcoal, coconut fiber, pebbles and gravels may also be used to remove the debris and dirt form water that enters the tank in small scale domestic roof top rainwater harvesting systems. The container is provided with a perforated bottom to allow the passage of water. The filter unit is placed over the storage tank. Another simple way of filtering the debris and dust particles in the water is to use a fine cloth as filter media. The cloth, in 2 or 3 layers, can be tied to the top of a bucket or vessel with perforation at the bottom.

7.2.7 Storage Tank

Storage tank is used to store the water that is collected from the Roof tops. Common vessels used for small-scale water storage are plastic bowls, buckets, jerry cans, clay of ceramic jars, cement jars, old oil drums etc. For storing larger quantities of water, the system will usually require a bigger tank with sufficient strength and durability.

Different types of storage tanks feasible for storing roof top rainwater are given below.

Storage tanks

- RCC
- Masonry
- Ferro Cement
- PVC

There are unlimited numbers of options for the construction of these tanks with respect to the shape (cylindrical, rectangular and square), the size (capacity from 1,000 - 15,000 L. or even higher) and the material of construction (brick, stone, cement bricks, Ferro-cement, concrete and reinforced cement concrete). For domestic water needs, taking the economy and durability of tanks into consideration, ferro-cement tanks of cylindrical shape in capacities ranging between 4,000 and 15,000 L are most suitable. Brick, stone or cement brick may be used for capacities ranging between 15,000 to 50,000 L. Cement concrete and reinforced cement concrete are used for tank capacities exceeding 50,000 L

Storage tanks are usually constructed above ground level to facilitate easy detection of structural problems/leaks, easy maintenance and cleaning and easy drawal of stored water. They are provided with covers on the top to prevent contamination of water from external sources. They are also provided with pipe fixtures at appropriate places for drawing water, cleaning the tank and for disposal of excess water. They are called tap or outlet, drain pipe and over flow pipe respectively. PVC or GI pipes of diameter 20 to 25 mm are generally used for the purpose.

7.2.7.1 Size of Storage Tanks for Rural Areas

Size of the storage tank needs to be carefully selected considering various factors such as number of persons in the household, water use, duration of water scarcity, rainfall, type and size of house roof and the status of existing water sources in the area. In general, the period of water scarcity for domestic purposes is found to be in the range of 90 days to 200 days depending upon the quantity and distribution of rainfall and water sources existing in the area.

The water use of the household should first be studied, considering the local culture and habits of the people influencing the water use. Availability of water at the doorstep, as is the case with RRHS, is likely to increase the water use of the household. This results in increase in required size of storage tank and its cost. It is found that the per capita water use varies over a range of 3 litres to 10 litres per day. A per capita water consumption of 5 litres per day for the domestic drinking and cooking purposes is found optimum. Adding 20% towards additional water requirement for visitors, festivals and wastage, a per capita water requirement of 6 litres per day may be considered for selecting the size of water storage tank.

The size of water storage tank may be determined using the following relation and approximating to the nearest thousand:

Size of Storage tank (in litres)	=	No. of persons in the household x Period of water scarcity (in days) x Per capita water requirement (in liters per day)

The capacity of storage tank, which reflects the total household water requirement during the period of water scarcity, need to be checked with the amount of water available from house rooftop during rains. If the amount of water available from roof is less than the required capacity of storage tank, then the household shall use the water available from roof only for a part of the water scarcity period.

Water available from roof is obtained from the following relation:

Water available (in litres) =	Annual rainfall (in mm) x Roof area (in sq.m) x Runoff Coefficient

Area of a roof shall be measured as the area projected on a horizontal surface. For practical purpose, it is measured on the ground surface and the area calculated as the product of length and breadth.

The coefficient of runoff varies depending on the type of roof and indicates the fraction of rainwater that can be collected from roof. Run-off coefficients for common types of roofs are shown in **Table 7.2.**

Table.7.2 Runoff Coefficients of Common Types of Roofs

GI Sheet	0.9
Asbestos	0.8
Tiled	0.75
Concrete	0.7

Example

Selection of size for storage tank

No. of persons in the selected household (4 adults and 4 children)	= 8
Period of water scarcity for the domestic needs	= 120 days
Per capita water requirement	= 6 L/day
Annual average rainfall	= 1000 mm
Area of roof made of country tiles	= 20 sq. m
Runoff coefficient for tiled roof	= 0.75

Size of storage tank (in litres) = No. of persons in the household x
Period of water scarcity (in days) x
Per capita water requirement (in lt./day)
= 8 x 120 x 6
= 5,760 L
Say 6,000 L

Check with water availability from roof top
Water available from roof top = Annual rainfall (in mm) X
Area of roof (in sq.m) X
Coefficient of runoff for the roof
= 1000 X 20 X 0.75 = 15000 liters

7.2.7.2 Size of Storage Tank for Urban Area

In urban areas covering major cities and towns where regular water supply is already existing throughout the year, the size of the storage tank for harvesting roof top rainwater can be decided as per the quantum of rainfall occurring in a single event. For this, it is assumed that serious water scarcity days are not counted and collection of roof top rainwater is voluntary/mandatory as an effort towards saving our natural resources. Moreover, it is difficult to create enough storage space due to various constraints. In such a situation, it is ideal to create the storage space to collect the rainfall per spell and utilize the same before the next spell. In this case,

Water available from roof = Annual rainfall (in mm) X Area of roof (in sq m) X Runoff coefficient for the roof

Water available per spell of rainfall = Rainfall per spell X Area of roof (in sq m) X Runoff coefficient for the roof

7.2.8 Collection Sump

A small pit is normally dug in the ground beneath the tap of the storage tank and constructed in brick masonry to make a chamber, so that a vessel could be conveniently placed beneath the tap for collecting water form the storage tank. A small hole is left at the bottom of the chamber, to allow the excess water to drain-out without stagnation. Size of collection pit shall be 60 cm x 60 cm x 60 cm.

7.2.9 Pump Unit

A hand pump or a power pump fitted to the storage sump facilitates lifting of water to the user. The size of the pump has to be decided depending upon the consumption of the stored water.

7.3 Data Requirements for Planning Rainwater Harvesting Systems

7.3.1 Amount of Rainfall (mm/year): The total amount of water available is the product of total available rainfall and the surface area from which it is collected. There is usually a runoff coefficient included in the computation to account for evaporation and other losses. Mean annual rainfall data may be used for obtaining rainfalls in an average year.

7.3.2 Rainfall Distribution: Rainfall pattern as well as the total rainfall determines the feasibility of a rainwater harvesting system in an area. A climate where rainfall is received regularly throughout the year will mean that the storage requirement and hence the system costs will be correspondingly low. On the other hand, in areas which receive a major part of the annual rainfall during a couple of months, the water collected during the rainy season has to be stored for as long as possible, requiring huge storage tanks coupled with provision for treatment. In such cases, it may be more economical to use rainwater to recharge ground water aquifers so that it can be extracted at times of need. Long-term rainfall records are necessary to ascertain the rainfall pattern.

7.3.3 Intensity of Rainfall: The maximum intensity of rainfall in mm/hr for a short duration (normally 20 min) will decide the peak flow to be harvested by the roof top rainwater harvesting system. The size of the gutter and diameter of down-take pipes have to be estimated on the basis of the peak flow.

7.3.4 Surface Area: The scope of any roof top rainwater harvesting system is restricted by the size of the roof forming the catchment. Other surfaces can also be included to supplement the roof top catchment area wherever feasible. Accurate estimate of total surface area of the catchment is a necessary prerequisite for planning the scheme.

7.3.5 Storage Capacity: The storage tank is usually the most expensive component of a rainwater harvesting system and hence, a careful analysis of storage requirements against the cost has to be carried out prior to implementation of the scheme.

7.3.6 Daily Demand: This varies considerably from 10-15 litre per-capita per day (lpcd) in some parts of the world to several hundred lpcd in some industrialized countries. This will have obvious impacts on system specification.

7.3.7 Number of Users: This will greatly influence the requirements and design specifications of the rainwater harvesting system.

7.3.8 Cost: Cost is a major factor in any rainwater-harvesting scheme.

7.3.9 Alternative Water Sources: Availability of alternative water sources can make a significant difference to the usage pattern of the water collected using rainwater harvesting systems. If a sustainable and safe ground water source is available within economic distances, a rainwater harvesting system may provide a reliable supply of water for a house/community for the majority of the year. On the other hand, where rainwater is to be stored and used only for domestic use, the pattern of use will depend on the quality of water.

7.3.10 Water management Strategy: A judicious water management strategy is required for proper and optimum use of harvested water. In situations where there is strong reliance on stored rainwater, there is need to control or manage the amount of water being used so that it is available for a longer period.

7.4 Feasibility of Roof Top Rainwater Harvesting Systems

7.4.1 Urban Area

In Urban area, water is mainly required for domestic and related uses, and is mostly drawn from surface water bodies, rivers, streams and/or ground water sources. Roof top rainwater harvesting is an ideal alternative in such areas. Appropriate storage facilities can be created to store roof top rainwater depending on availability of space. Rainwater harvesting in urban areas helps not only in meeting at least a part of the water requirement but also prevents storm runoff and flooding of roads during heavy rains. It also reduces the pumping costs and reduces the stress on ground water resources.

7.4.2 Rural Area

In rural area, ponds, streams and wells have traditionally been used as sources of water for drinking and other domestic uses. In recent years, bore wells with hand pumps and small water supply schemes have almost replaced these traditional sources of water. Yet, in many rural habitations, these sources have not been able to supply water to the rural households round the year, due to various reasons. Domestic Roof top Rainwater Harvesting System (RRHS) provides a viable solution to bridge the gap between demand and supply of water in such areas, especially during periods of water scarcity. Specifically, RRHS is applicable in:

- Areas where traditional water sources like ponds, streams and wells dry up during summer.
- Areas with problems of ground water salinity such as coastal areas.
- Areas where ground water has high concentration of harmful chemical constituents such as fluoride, iron and arsenic.
- Areas where water sources are contaminated due to pollution from various sources.

The advantages of RRHS over conventional water supply systems in rural areas are:

- It can provide a dependable, economical and durable source of water for drinking and cooking purposes to the rural households, especially during periods of water scarcity.
- Water is made available at the doorstep of the house.
- Easy access to the source of water improves the health and hygiene of family.
- Time spent in fetching water from distant water sources is considerably reduced. This generally being the responsibility of women, the time saved could be productively used for themselves and their family.
- Rainwater from roof top is free from contamination and pollution, and generally found clean and potable.
- Requires simple maintenance, which could be carried out by the users easily.
- Construction and maintenance are simple and does not require sophisticated tools or technology.

The planning of roof top rainwater harvesting systems in an area needs to be done in terms of its technical suitability, social acceptance and economic viability.

7.5 Technical Suitability

Assessing technical suitability involves the study of factors which influence the need and reliability of RRHS. Important considerations in this regard are described briefly in the following sections.

7.5.1 Existing Water Sources

Existing water source such as community wells, hand pumps, small water supply schemes, ponds and streams shall be studied. The availability of water, its quality and accessibility of these sources during different seasons of the year should be looked into for determining the period for which water may be required from RRHS.

7.5.2 Roof Catchment

The type of roof determines the quality of water that is collected in the storage tank. Among the commonly seen roof types in rural areas, concrete, tiled, asbestos sheet and galvanized iron sheet are most suitable as roof catchments. The roof should be away from big trees to avoid accumulation of leaf litter and bird droppings. Thatched roofs are not suitable as roof catchments because the water collected from these roofs gets brownish colour and carries pieces of roof material.

The slope and shape of the roof are also important in planning a roof top rain top rainwater harvesting system. Water flows with high velocity on steep-sloped roofs, causing overflow or wastage of water form gutters and filter. Gentle slopes in the range of 10 to 30 degrees are most suitable for smooth flow of water into the storage tank. Roofs having slope more than 30 degrees are to be avoided wherever possible.

The size of roof is another important factor which determines the amount of water available for storage in the RRHS. Generally, a roof area of 15-20 square meters is required for collecting sufficient water required for a household. Roof catchments of lesser sizes could become a limiting factor in designing RRHS to the required capacity.

7.5.3 Rainfall

Roof top rainwater harvesting systems collect rainwater from the roof catchments during rainy days. Therefore, the amount and distribution of rainfall are major factors influencing the dependability of such systems.

In India, a major part of the rainfall is received during monsoons. The southwest monsoon reaches India in June and extends normally up to September. The northeast monsoon extends from October to December. Kerala and north-eastern states benefit from both the monsoons and hence receive rainfall extending over a period of 7-8 months. Rest of the country receives rains during southwest monsoon, except Tamil Nadu, which receives comparatively more rains during northeast monsoon. The rainfall is limited to 3-4 month in these regions. The average annual rainfall received also varies across the country, from 200-250 mm in western Rajasthan to >3000 mm in Kerala and north-eastern States.

The amount and period of rainfall at a given place are important factors that determine the period for which water will be required from RRHS. For example, at places where rainfall is received only during southwest monsoon (3-4 months), the period of water requirement form RRHS could be longer than that of places in Kerala and north-eastern States. Also, in low rainfall areas such as western Rajasthan and northern Gujarat, roof size could become a limiting factor as households in these areas may require water for an extended dry period of 7-8 months. Here, traditional water harvesting systems make use of open lands, adjacent to the house, as catchments areas for domestic water harvesting systems.

7.5.4 Space

Among all the components of roof top rainwater harvesting systems, storage tank is the component occupying most space, and hence the space required for the system depends on the size of the storage tank. For a typical 10,000 litre tank, the minimum space required is 3.0 x 3.0m. Therefore, assessment of availability of space adjacent to the house shall be done giving due importance to the preferences of the household. Storage tanks located near the roof reduce the cost of down pipes. The site should be clean, hygienic and away from cattle sheds to avoid contamination of stored water.

7.6 Economic Viability

A typical domestic roof top rainwater harvesting system requires and investment of about Rs.12,000/- to Rs.16,000/-, depending on the capacity of the storage tank. This works out to Rs.2.34 to Rs.1.49 per litre of water stored. This is quite high when compared to the free water available through government-sponsored schemes, where community participation and labour are not required at the construction stage. Hence, investment to this extent is a costly option and may be unaffordable to many rural households. The cost of roof top rainwater harvesting systems could be brought down to a certain extent by using local materials such as bamboo for gutters, down pipe and first flush pipe. Contribution from users could be also be raised in terms of labour and materials to meet a part of the investment.

It is advisable to have the user household themselves meet a sizeable portion of the cost of RRHS to ensure its sustainability and replicability. This would also encourage ownership and appropriate maintenance of the system at the level of households. Extending soft loans repayable in easy instalments would be appropriate for this purpose. The existing Government schemes, which finance women self-help groups in rural areas such as the *Rashtriya Mahila Kosh* and NABARD self-help group schemes, could be utilized for extending such loan facilities to the rural households.

7.7 Social Acceptance

7.7.1 Acceptance of Roof Water as Drinking Water

Colour, odour and taste are the three important considerations for people in choosing sources of drinking water. Clean water without any odour and with a ‘good taste’ is usually preferred for dinking purpose. In case of water used for cooking, water having lesser amounts of dissolved salts is preferred, because it consumes less time to boil the food grains and vegetables. As the rainwater contains very little dissolved salts and is almost free from pollutants, it tastes ‘good’ and is suitable for drinking and cooking purposes.

However, in roof top rainwater harvesting systems, water is collected in the storage tank during rainy season and is drawn from the system only after other sources like ponds, wells and hand pumps dry up or become inaccessible to the household. This means that the water collected in RRHS remains stored for a period of 3 to 6 months before it is actually used. This makes water from RRHS not readily acceptable to many people. Therefore, awareness and education programmes need to be organized on the potability of water when the system is appropriately maintained. People's perceptions need to be given due importance during such programmes, to enable them to develop appropriate understanding of the system.

Domestic water use varies from place to place depending on the culture and habits of people as well as availability of water. Water made available at the doorstep is likely to increase the water usage. Increased water use would improve the cleanliness and hygiene within the household as well as improve the health of the people. Yet, people have to be educated to draw water in controlled quantities so that they could benefit from the system throughout the period of water scarcity to which the system is designed.

7.7.2 Willingness of Households to Participate

Domestic Roof top Rainwater Harvesting Systems are meant for meeting the water needs of individual households and are constructed right at their doorstep. Hence, proper care and maintenance of the system by the household is essential for a reliable supply of good quality water.

Willingness of household to participate in planning, construction and maintenance of such systems are very important for the success of the programme. The motivating factors such as availability of sufficient water, economy and ease in maintenance, ownership of the system etc., which encourages the people to participate should be identified and proper orientation should be given.

7.7.3 Traditional Practices of Roof Water Collection

Collection of roof water on small scale from house roofs to meet the immediate household needs is a traditional practice in some parts of India such as north-eastern states, Rajasthan and eastern coastal areas of Tamil Nadu. Small vessels or drums are used to collect and store water. Locally available bamboo is split and used as gutters. Existence of such practices makes RRHS to meet long-term needs acceptable. However, training and transfer of knowledge on systematic and economical construction of these systems is required.

7.8 Water Quality and Health

Rainwater is often used for drinking and cooking and hence it is vital that the highest possible quality standards are maintained. Rainwater, unfortunately, often does not meet the World Health Organization (WHO) water quality guidelines. This does not mean that the water is unsafe to drink. It has been found that a favourable user perception of rainwater quality (not necessarily perfect water quality) makes an enormous difference to the acceptance of RWH as a water supply option. Generally, the chemical quality of rainwater will fall within the WHO guidelines and rarely presents problems. There are two main issues when looking at the quality and health aspects of domestic rainwater harvesting systems.

7.8.1 Bacteriological Water Quality

Rainwater can become contaminated by faeces entering the tank from the catchment area. It is advised that the catchment surface always be kept clean. Rainwater tanks should be designed to protect the water from contamination by leaves, dust, insects, vermin and other industrial or agricultural pollutants. Tanks should be located away from trees, with good-fitting lids and kept in good condition. Incoming water should be filtered or screened, or allowed to settle to take out foreign matter. Water, which is relatively clean, on entry to the tank, will usually improve in quality if allowed to be inside the tank for some time. Bacteria entering the tank will die off rapidly if the water is relatively clean. Algae will grow inside a tank if sufficient sunlight is available for photosynthesis. Keeping a tank dark and in a shady spot will prevent algae growth and also keep the water cool. As already mentioned, there are a number of ways of diverting the dirty 'first flush' water away from the storage tank. The area surrounding a RWH structure should be kept in good sanitary condition, fenced off to prevent animals fouling the area or children playing around the tank. Any pools of water gathering around the tank should be drained and filled.

7.8.2 Insect Vectors

There is a need to prevent insect vectors from breeding inside the tank. In areas where malaria is prevalent, providing water tanks without any care for preventing insect breeding can cause more problems than it solves. All tanks should be sealed to prevent insects from entering. Mosquito-proof screens should be fitted to all openings. Some practitioners recommend the use of 1 to 2 teaspoons of household kerosene in a tank of water, which provides a film to prevent mosquitoes settling on the water.

7.8.3 Water Treatment

There are several simple methods of treatment for water to made suitable for drinking

- Boiling water will kill any harmful bacteria which may be present
- Adding chlorine in the right quantity (35ml of sodium hypochlorite per 1000 litres of water) will disinfect the water
- Slow sand filtration will remove any harmful organisms.
- A recently developed technique called SODIS (Solar Disinfections) utilises plastic bottles, which are filled with water and placed in the sun for one full day. The back of the bottle is painted black.

The reasons for variations in chemical constituents and bacteriological properties of water from RRHS could be many, the most important ones of which are listed below:

- Even though the water flows over the house roof for a short distance, it may dissolve some chemicals deposited on the roof or the residues of chemical reactions between the atmospheric gases and the roof material.
- In general, rainwater is pure and free from contamination. However, the air pollution from factories, industries, mining etc. does influence the chemical quality of water vapour in the atmosphere. When this water vapour condenses and comes in contact with the roof material, it may react and leave residue on the roof. This phenomenon usually occurs over areas surrounding industries. The impact of this pollution on the rainwater quality is not alarming, but needs attention.
- Rainwater, while passing on the roof may carry the dust and debris resulting in change in the quality of water.

- Organic matter from the bird droppings, rotten tree leaves, seeds and algae will be dissolved and carried by the rainwater while flowing on the roof top. This may also cause quality changes of water stored in the tank.
- Breeding of mosquitoes or entry of insects through the openings of the tank such as over flow pipe may affect the quality of water.

Chemical and bacteriological contamination of roof water during the collection and storage processes can be prevented effectively by proper and regular maintenance of the system. The users of the system need to be trained in various activities of maintenance.

7.8.4 Analysis of Water Samples

As bacteriological contamination cannot be detected by the naked eye, it is necessary to analyze the quality of water in laboratories by collecting few water samples from storage tank. These tests help in verifying the presence of pathogenic bacteria.

7.8.5 Disinfecting Water

Disinfecting is the process of killing the disease-causing micro organisms present in the water. This can be done either by boiling the water in a vessel before consuming it or by dissolving bleaching power in required quantity to the water stored in the tank.

For disinfecting using bleaching powder, the general dosage recommended is 10 milligrams of bleaching powder containing 25% of free chlorine per litre of water. This meets the required standard of 2.5 milligrams of chlorine per litre of water.

After adding the bleaching power, the water shall be stirred thoroughly for even distribution of the disinfectant. The water should be kept for about 30 minutes after adding bleaching powder before it is ready for use. The quantity of bleaching power to be added for different water depths in the storage tank is shown in **Table 7.3.**

Table- 7.3 Recommended Dosage of Bleaching Powder for disinfecting Water

Storage Capacity of tank (L)	Dosage of bleaching powder (in grams)			
	Full Tank	Tank three fourth (3/4) full	Tank half (1/2) full	Tank one fourth (1/4) full
5,000	50	37.5	25	12.5
6,000	60	45	30	15
7,000	70	52.5	35	17.5
8,000	80	60	40	20
9,000	90	67.5	45	22.5
10,000	100	75	50	25

7.9 Ready Reconers for Design of Roof Top Rainwater Harvesting Systems

Ready reconers for computing the availability of rainwater for roof top rainwater harvesting, for computing the peak flow from roofs and for determination of the size of storage tanks are given in **Table 7.4a, 7.4b and 7.4c** respectively.

Table 7.4 Ready Reconers for Design of Roof Top Rainwater Harvesting Systems

a. Availability of Rainwater for Roof Top Rainwater Harvesting

Rainfall (mm)	100	200	300	400	500	600	800	1000	1200	1400	1600	1800	2000
Roof top Area (Sq m)	Harvested water from Roof top (cu m)												
20	1.6	3.2	4.8	6.4	8	9.6	12.8	16	19.2	22.4	25.6	28.8	32
30	2.4	4.8	7.2	9.6	12	14.4	19.2	24	28.8	33.6	38.4	43.2	48
40	3.2	6.4	9.6	12.8	16	19.2	25.6	32	38.4	44.8	51.2	57.6	64
50	4	8	12	16	20	24	32	40	48	56	64	72	80
60	4.8	9.6	14.4	19.2	24	28.8	38.4	48	57.6	67.2	76.8	86.4	96
70	5.6	11.2	16.8	22.4	28	33.6	44.8	56	67.2	78.4	89.6	100.8	112
80	6.4	12.8	19.2	25.6	32	38.4	51.2	64	76.8	89.6	102.4	115.2	128
90	7.2	14.4	21.6	28.8	36	43.2	57.6	72	86.4	100.8	115.2	129.6	144
100	8	16	24	32	40	48	64	80	96	112	128	144	160
150	12	24	36	48	60	72	96	120	144	168	192	216	240
200	16	32	48	64	80	96	128	160	192	224	256	288	320
250	20	40	60	80	100	128	160	200	240	280	320	360	400
300	24	48	72	96	120	160	192	240	288	336	384	432	480
400	32	64	96	128	160	192	256	320	384	448	512	576	640
500	40	80	120	160	200	240	320	400	480	560	640	720	800
1000	80	160	240	320	400	480	640	800	960	1120	1280	1440	1600
2000	160	320	480	640	800	960	1280	1600	1920	2240	2560	2880	3200
3000	240	480	720	960	1200	1440	1920	2400	2880	3360	3840	4320	4800

b. Computation of Peak Flow from Roof

Rainfall Intensity mm/hr for 20 min	50 (min.)	100 (min.)	150 (min.)	200 (min.)
Roof top area sq m	Peak flow in litres/s (lps)			
20	0.28	0.56	0.83	1.11
30	0.42	0.83	1.25	1.67
40	0.56	1.11	1.67	2.22
50	0.69	1.39	2.08	2.78
60	0.83	1.67	2.50	3.33
70	0.97	1.94	2.92	3.89
80	1.11	2.22	3.33	4.44
100	1.39	2.78	4.17	5.55
200	2.78	5.56	8.33	11.11
500	6.95	13.89	20.83	27.78
1000	13.92	27.78	41.67	55.55

c. Size of Storage Tank

(Depth of live storage above the outlet pipe = 1.4m)

Tank Capacity (in cum)	Diameter of Tank (in m)
1.60	1.21
2.40	1.48
3.20	1.71
4.00	1.91
4.80	2.09
5.60	2.26
6.40	2.41
7.20	2.56
8.00	2.70
9.60	2.95
11.20	3.19
12.00	3.30
12.80	3.41
14.40	3.62
16.00	3.81
16.80	3.91
19.20	4.18
20.00	4.26

Note: For rural areas, the diameter of tank may be limited to 3 m. The tank would be adequate to meet the drinking water requirements of a family of 5 members for 6 months. For large storage, two or more tanks may be provided instead of a single large tank.

7.10 Computation of Flow through Half Section Gutters

Flow through channels of constant cross section is computed using the formula

$$Q = A \times V$$

Where Q is the maximum carrying capacity of the channel, A is its area of cross section and

$$V = c\sqrt{mi}$$

'c' is the Chezy's coefficient, which is dependent upon the nature of channel material. The value of 'c' for cemented or finished surfaces is 0.55.

$$m = \frac{\text{Area of cross section of flow}}{\text{Wetted Perimeter}} = \frac{A}{P}$$

i = Slope of channel bed

Using the above formula, flows through half section gutters of different diameter channels have been calculated by assuming 1: 1000 slope (**Table 7.5)**.

Table 7.5 Flow through Half-Section Gutters of Channels of Different Diameter

Diameter of half channel gutter (mm)	Max. carrying capacity (Q) (lps)
100	1.08
150	2.97
200	6.10
250	10.67
300	16.82

7.11 Data Requirements for Design of Roof Top Rainwater Harvesting Systems

The summary data sheet showing the data requirements for design of a successful roof top rainwater harvesting system is shown in **Table 7.6**

Table 7.6 Summary Data Sheet for Designing Rainwater Harvesting System

1. Type of buildings:	
a. Residential	
b. Commercial	
c. Industrial	
d. Park	
c. Open Area	
2. Layout plan of the building:	
a. Roof top area	
b. Paved area	
c. Open area	

3. Water Availability	
a. Rainfall (Data on daily basis for two years) (if available)	
b. Rain fall intensity	
c. Number of rainy days	
d. Height of roof	
4. Water withdrawal:	
a. Number of tube wells	
b. Discharge	
c. Number of hrs operated per day	
5.Quality of source water:	
6. Number and locations:	
a. Tube wells	
b. Bore wells	
c. Hand pumps	
7. Type of roof:	
a. Flat roof	
b. Sloping roof	
8. Rainwater disposal system: a. Drain pipes	
i) Up to ground	
ii) Above ground	
b. If Sloping roof	
i) Gutters	
ii) Size of gutter	
9. Type of drain pipes	
a. G1	
b. Cement	
c. PVC	
d. Others	
10. Hydrogeological settings	
a. Depth to water level	
b. Geological formation water bearing strata and water bearing formation	
c. Type of soil	
d. Depth of clay bands/clay	
e. Depth of tube wells	
f. Present discharge of tube wells	
g. Assembly chart of tube wells	
h. Hydraulic conductivity	
i. Specific yield of aquifer	
j. Storage capacity of aquifer	
k. Ground water flow pattern	
l. Thickness of soil cover	
m. Infiltration rate of:	
i) Soil	
ii) Aquifer	

11. Any other information such as:	
a. Problems due to submergence area and location	
b. Rainwater coming from adjoining area	
c. Lack of storm water drains	
d. Decline/failure of tube wells	
e. Tube wells started giving saline or bad quality of water.	

7.12 Design Example

Problem:

A house has a sloping roof of G.I.sheet with an area of 50 sq m. The owner of the house has a family of 5 members. Design a roof water harvesting system. The 10 year rainfall for the areas is as follows:

Year 1	320 mm
Year 2	360 mm
Year 3	311 mm
Year 4	290 mm
Year 5	330 mm
Year 6	280 mm
Year 7	335 mm
Year 8	380 mm
Year 9	355 mm
Year 10	340 mm

The maximum rainfall intensity is 10 mm/hour. The lower edge of the roof is 3 m above the ground.

Solution:

Arranging the rainfall in descending order, we get: 380, 355,340, 335, 330, 320, 311, 290, 280

The highest rainfall of 380 mm is equalled or exceeded only once in 10 years. Therefore, it's expected that the return period of this much rainfall is 1 in 10 years, which is 'rare'. On the other hand, the lowest rainfall of 280 mm is equalled or exceeded in all the 10 years. Thus, this is the most reliable figure. So, the system may be designed for this rainfall.

From **Table 7.4a**, for the roof area of 50 sq m and rainfall of 280 mm, the available water works out as 11.2 cum or 11,200 litres

Allowing for a consumption of 10 lpcd, this water should be sufficient for 224 days or at least 7 months. As houses are of low height in rural areas, 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.

Size of Collector Channel (Gutter)

During heavy rains having intensity of 10 mm/hr or more, the runoff coefficient may be taken as 0.9 (assuming a net loss of 10% 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 sq m is worked out as

Roof Area (m^2) x Rainfall intensity (m/sec) x Runoff coefficient

$$= 50 \times \frac{10}{(1000 \times 60 \times 60)} \times 0.9 = 1.25 \times 10^{-4} = 0.125 \text{ lps.}$$

Assuming the slope of the collector channel as 5 cm for 1 m, i.e. 1 in 200

Trial -1

Providing a collector channel of 0.1 m diameter

Cross sectional area of the channel (A) = 0.003925 sq m

Perimeter (P) = 0.157m

Hydraulic Mean depth (R) $= \frac{0.003925}{0.157} = 0.25\text{m}$

For slope of 1 in 200 for the collector channel,

Velocity of flow (V) = 0.24 m/sec

Discharge (Q) = AX V
= 0.003925 x 0.24
= 0.000942 cum/sec

As the design discharge is only 0.000125 cum/sec, the channel is oversized and hence, is not acceptable.

Trial-II

Considering a channel of 0.05 m diameter

Area (A) = 0.00098 sq m

Perimeter (P) = 0.0785 m

Hydraulic Mean Depth, R $= \frac{0.00098}{0.0785} = 0.0125m$

Velocity (V) = 0.152 m/sec

Discharge (Q) = A x V
= 0.00098 x 0.152
= 0.000148 cum/sec.

As this corresponds well with the designed discharge, this channel diameter is acceptable.

The channel may be made of plain Galvanized Iron (G.I) sheet. Width of the G.I.sheet required for channel is the perimeter of the channel

P = 0.0785 m = 78.5mm

Providing 25 mm extra for fixing with rafters / purlins,

Total width required = 78.5 + 25 = 103.5 mm

Say 104 mm

8. IMPACT ASSESSMENT

Artificial recharge structures are constructed mostly with the objective of augmenting ground water resources and/or to improve its quality. Assessment of impacts of the artificial recharge schemes implemented is essential to assess the efficacy of structures constructed for artificial recharge and helps in identification of cost-effective recharge mechanisms for optimal recharge into the ground water system. It also helps to make necessary modifications in site selection, design and construction of structures in future.

Impact assessment may require monitoring of the recharge structure, ground water regime, changes in pattern of water supply, cropping pattern, crop productivity and/or water quality. In recent years, tracers such as Tritium, Rhodomine B, fluorescent dyes and environmental isotopes are also being used for demarcating the area benefited by artificial recharge structures.

The methodology of impact assessment is highly site-specific and can vary considerably depending upon various factors such as hydrogeological set-up and ground water utilization pattern. General guidelines for impact assessment of artificial recharge structures are discussed briefly in the following sections.

8.1 Monitoring of Recharge Structures

Surface structures such as percolation ponds, check dams and cement plugs need to be monitored at regular intervals to assess the actual storage created in the structures, period of impounding, capacity utilization of the structure, rate of percolation and siltation problems if any. Quantification of storage in the structures may require setting up of monitoring devices within the structures. Devices such as gauges for area-capacity analysis are commonly used in surface recharge structures. Daily monitoring records are preferred for realistic assessment of storage created by multiple fillings of the structures. Evaporation and seepage losses from the structures are also to be accounted properly to evaluate the recharge efficiency of the structures.

In case of subsurface structures, the intake water supplied to the structures is measured by suitable measuring devices. Appropriate measuring devices such as flow meters and ‘V’ notches can be used for measurement. Daily records of such measurements help quantify the amount of water utilized for recharge purpose.

8.2 Water Level Monitoring

The objective of water level monitoring is to study the effect of artificial recharge on the natural ground water system. The monitoring system should be designed judiciously to monitor impact of individual structures which can further be extended to monitor the impact of groups of such structures in the area where artificial recharge is being done. Monitoring of water levels during the planning stage of artificial recharge projects helps in assessment of the ground water conditions of the area and helps in identification of the most suitable method for ground water augmentation. A properly designed observation well network is used for understanding the ground water flow pattern and the spatial and temporal changes in water levels/potentiometric heads in the area.

During the planning and feasibility study stage, the observation well network is generally of low well density but spread over a large area with the primary aim of defining the boundaries of the aquifer to be recharged and to know the hydraulic characteristics of the natural ground water system. After identification of the feasible artificial recharge structures, the observation well network is redefined in a smaller area with greater well density.

For effective monitoring of the changes in the water levels due to artificial recharge, the network should have observation wells near the center of the recharge facility, at a sufficient distance from the recharge facility to observe composite effects and also near the limits of hydrological boundaries. If the aquifer being recharged is overlain by confining /semi-confining layers, piezometers should be installed to monitor the water levels of overlying and underlying aquifers separately to study the effects in both the aquifers. In cases where surface water bodies are hydraulically connected with the aquifers being recharged, it is advisable to monitor the water level profiles of both surface water and ground water.

Demarcation of the zone of influence of the artificial recharge structure is one of the main objectives monitoring in the context of artificial recharge projects. The following observations are generally associated with the area benefited by an artificial recharge structure:

1. Well hydrographs in the area benefited will have a flat apex during the period when there is water in the recharge structure (tank, pit etc.).
2. Wells located outside the zone of influence normally show an angular apex during the period of recharge, whereas those situated within the zone of influence have a flatter apex.
3. The recession limbs of well hydrographs close to a recharge structure normally have gentle gradients as compared to those located far off.
4. Crops in the zone of influence are normally healthier when compared to those outside the benefited area. Furthermore, crops with high water requirements are more likely to be grown in the zone of influence.
5. Well yields in the zone of influence will normally be higher when compared to those outside it. Wells in benefited zone may have more sustainability in lean period than those located outside.

The behaviour of water table / piezometric head profile prepared from the data collected from the observation well network over a period of time can clearly establish the efficacy of the artificial recharge scheme. Answers to questions related to the extent of the area benefited and the quantification of ground water augmentation could also be worked out from such data. The study of fluctuation over time for both surface and ground water levels in the same area may also indicate whether the ground water augmentation is taking place as envisaged or not. In case any deviation is observed, the reasons for the same could be identified and necessary remedial measures taken up.

8.3 Water Quality Monitoring

A proper evaluation of potential water quality and aquifer quality problems associated with artificial recharge is a key component of a ground water recharge scheme. The development of reliable pre-, operational and post-operational monitoring programs is an integral part of the development of a successful ground water recharge scheme.

A reliable water quality monitoring system for an artificial recharge scheme will involve i) Evaluation of existing water quality data, ii) pre-operational monitoring, iii) operational monitoring and iv) post-operational monitoring.

8.3.1 Evaluation of Existing Water Quality Data

The first step that should be followed in evaluating the potential water quality problems associated with a proposed ground water recharge project is to obtain detailed information on the chemical characteristics of the proposed recharge waters. A critical examination of the existing data on the waters that would be recharged to the aquifer should be made to first determine their reliability and representativeness. In case the available data is not considered to be reliable, collection and analysis of source water samples may be done afresh.

8.3.2 Pre-operational Monitoring

The augmentation of recharge by surface waters and their associated contaminants can greatly increase the potential for ground water quality problems due to the increased hydraulic and contaminant loading. The characterization of ground water quality is often not adequately done to properly evaluate potential ground water and aquifer quality problems associated with a ground water recharge project. it is important to properly assess how the variable parameters in sampling such as bore hole volume purged and rate of purging before sampling influences the composition of the samples. Chemical parameters of particular importance in reliably assessing ground water quality samples are the redox conditions within the aquifer and the presence of suspended solids in the samples. Because of the chemistry of ferrous and ferric iron, small changes in the redox (oxidation reduction) characteristics of the sample as a result of the introduction of oxygen into the sample during sampling can drastically change the chemical characteristics of the samples. Hence, it is important to maintain the oxygen concentrations in a sample collected from an aquifer the same as that of the aquifer. Failure to do so could readily change the distribution between dissolved and particulate forms of many trace contaminants of water quality concern.

The presence of suspended solids in a water sample from an aquifer is a clear indication that the sampling well has been improperly constructed and developed and /or the sampling procedure used, especially the purging, has been improperly done. Aquifers typically do not contain large amounts of suspended material. Aquifer samples that contain suspended material are unreliable to properly characterize chemical characteristics of the ground waters within the aquifer at the point and time of sampling.

It is also important that the sampling program for the ground water is properly developed to reflect the site specific hydrogeology of various principal components of the aquifer. Failure to do so could readily lead to erroneous conclusions concerning the chemical characteristics of the aquifer waters and the chemical reactions that can take place within the aquifer upon introduction of recharge waters to it. Depending on the situation, at least one year and often several years of data may be needed to reliably characterize the aquatic system of interest. The best way to determine the length of time necessary in pre-operational monitoring as well as the frequency of monitoring a particular system is to examine the ability to predict the chemical characteristics of the system prior to collecting the next set of samples. Once it becomes clear that the characteristics of a particular recharge water source and aquifer are predictable with a high degree of certainty based on past monitoring results, it should then be possible to reduce the frequency and duration of pre-operational monitoring. If, however, it is not possible to make these predictions reliably because of the high variability in the systems, proceeding with the operation of the proposed recharge project could be met with significant problems in detecting incipient water quality problems before they adversely impact large parts of the aquifer.

8.3.3 Operational Monitoring

With the initiation of the recharge activities, a significant increase in the frequency of sampling, especially near the point of recharge, should occur. Actually the operational sampling program should be initiated several months before actual recharge starts in order to evaluate the ability to conduct the monitoring program with the facilities and personnel available. If the pre-operational monitoring program has been passive**,** then it should, at the time of initiation of recharge, become an active program, where the data is examined in detail as soon as it is available for the purpose of determining its reliability and any potential problems that are developing with the recharge project. In addition to chemical and microbiological measurements in the recharge waters as well as within the aquifer, detailed monitoring of the hydraulic characteristics of the injection / infiltration system should be conducted to determine the changes in the hydraulic characteristics of the recharge system and the aquifer in the vicinity of the recharge. In addition to monitoring the chemical contaminants in the recharge waters as well as aquifer, consideration should be given to the contaminant transformation products that might be formed in the recharge water. An area of particular concern in the recharge waters is whether there is sufficient BOD in these waters to exhaust the dissolved oxygen in the aquifer waters for those aquifer systems that are oxic prior to initiation of recharge. Bore hole dissolved oxygen measurements should be made at frequent intervals at various distances from the point of recharge in order to detect incipient dissolved oxygen depletion that could lead to its exhaustion from the recharge waters. Since, in general, except for nitrate-related issues, anoxic conditions in aquifers tend to lead to poor water quality, care should be taken to prevent the recharge waters from becoming anoxic within the aquifer. Failure to do so could readily result in iron, manganese and hydrogen sulphide problems. If problems of this type start to develop, it may be necessary to add dissolved oxygen either directly or through the introduction of hydrogen peroxide, in the recharge waters in order to prevent problems of this type from occurring.

Once the operational monitoring program data have become stabilized, i.e. are predictable based on past monitoring results, then the frequency of operational and post-operational monitoring can be decreased. This will likely take several years of operation, however, for fairly constant composition recharge waters and fairly homogeneous aquifer system with respect to its hydrogeologic and chemical characteristics.

The type of water quality monitoring programme depends on the specific problem being studied, such as changes in ground water quality, effect of soil salination, prevention of any contamination etc. The samples to be collected will also depend on the purpose and are generally categorized into a) Indicative, b) Basic and c) Comprehensive. Indicative samples are collected at 1 to 4 months intervals and are used to ascertain the presence of recharged water in the aquifer. Basic samples are taken at monthly intervals for wells already influenced by recharge to determine the effect of recharge on ground water quality and the purification provided by flow through the soil and aquifer system. Comprehensive samples are taken at intervals of 6 months to 1 year for observation wells and production wells to determine water quality with respect to specific standards for intended water use.

8.3.4 Post-operational Monitoring

When groundwater recharge is terminated, it is important that the monitoring of the aquifer be continued until the waters in the aquifer stabilize in composition. This will normally take several years of monthly monitoring. This monitoring should continue for quarterly intervals for several years.

8.4 Examples of Impact Assessment

Central Ground Water Board has taken up and completed a number of artificial recharge studies throughout the country during 8^{th} and 9^{th} plan periods. The methodology of impact assessment of artificial recharge schemes is explained with the help of one of these schemes in which a percolation tank was constructed for artificial recharge at Ichkheda in Maharashtra.

The site is located about 1.5 km Northwest of village Ichkheda and 3.5 km Northeast of Adgaon. It has maximum storage capacity of 45 TCM and maximum submergence area of 22500 m^2 at F S L. Salient features of the tank are given in **Table 8.1**. Unconfined aquifer consisting of talus and scree deposit (bazada) lies below the submergence of tank. The soil cover is negligible in the tank bed and infiltration rate of 20-30 cm/hour was observed. The soil moisture was less than 5 percent during pre monsoon, 1995.

8.4.1 Catchment Characteristics

The tank has a catchment area of 0.96 km^2. It was constructed across the local nala which is a second order stream. Total length of the stream above the site is 3.6 km. The drainage course is curvilinear with dendritic drainage pattern. The average gradient of catchment is 4^0. The drainage density in the catchment is very low, indicating highly porous and permeable nature of the substrata. The catchment characteristics are presented in Table 8.1.

The hilly catchment covering 0.3 km^2 area is occupied by basaltic flows and is covered by teak and bamboo forest. Foothill catchment (0.66 km^2) occupied by bazada formations is partly cultivated by rain fed crops. There is no human settlement or other storage structure in the catchment. Great Boundary Fault, passing across the catchment, demarcates the contact of basalt and bazada formations.

8.4.2 Hydrology

No river gauging station exists on the *nalah*. The nearest rain gauge station is situated at Yawal. The rainfall data of 1951 to 1990 was used. The dependable monsoon rainfall at 50% worked out as 676 mm (26.64”). The yield per square mile as per Strange’s table is 7.0624 for 676 mm rainfall considering the catchment as bad. The net yield at site is estimated as 73.89 Th M^3. Based on the site conditions it was proposed to fully utilise the catchment yield.

Table 8.1 Catchment Features of Ichkheda Percolation Tank

S.No.	Features	Particulars
1.	Hilly catchment	0.30 Km^2
	Foothill catchment	0.66 Km^2
2.	No. of I^{st} order streams	2
	No. of II^{nd} order stream	1
3.	Length of I^{st} order streams	0.7 km
	Length of II^{nd} order stream	2.9 km
4.	Bifurcation ratio	1:2
5.	Shape of the catchment	Elongated (N-S)
6.	Max. length of the catchment(L)	2.7 km
7.	Width of the catchment (W)	0.6 km
8.	Form factor (A/L^2)	0.13
9.	Circulatory ratio	0.17
10.	Elongation ratio	0.20
11.	Drainage density (km / km^2)	3.8
12.	Relief ratio	0.068
13.	Slope of the streams in hill	20^0
	Slope of the streams in foot hill	1.5^0
14.	Nature of catchment	Average

Clear over fall (COF) type of waste weir of 16 metres length and 1 m flood lift, designed to discharge 27 m^3/second was provided on the left bank. Rehabilitation was not required as no house or structure was submerged. Around 2.6 ha private agricultural land was acquired for the construction work. The construction of tank was completed in June, 1995 and since then it stored water every year.

8.4.3 Analysis of Efficiency

Monitoring of the tank was commenced on 24-6-95 and continued up to 1998. The tank balance analysis with regard to the gross storage and percolation fraction etc. were done for the entire period of impounding. The inflow into the tank as observed during various periods was appropriately accounted. Evaporation losses were calculated on daily basis and visible seepage was measured. Thus net percolation amount contributing to the ground water recharge was calculated as per the procedure discussed earlier.

A gauge of 6 metre height was installed and daily monitoring of tank level was done. The contour plan of the tank submergence was prepared, immediately after the completion of construction, as shown in **Fig. 8.1**. The area-capacity curves were then drawn showing the area of submergence in thousand sq m (ThM^2) and storage capacity in thousand cubic metre (TCM) with respect to gauge reading as shown in **Fig. 8.2.**

The efficiency of the percolation tank in monsoon (June-Oct) and non-monsoon (November onwards) of three years has been calculated in the range of 95-98% and 92 -95% respectively. The visible seepage were nil and evaporation losses were ranging 2 - 5% in monsoon and 5 -8% in non-monsoon. Though the tank has maximum storage capacity of 45 Th M^3 at FSL, due to regular and repetitive filling occurring during the monsoon and then its simultaneously percolation, the structure could store much more quantity of water. The capacity utilisation of the tank, considering multiple fillings, ranged between 140% to 344% as shown in **Table 8.2**. The average percolation varied from 0.5 TCM/day in 1997-98 to 0.8 TCM/day in 1996-97.

Table 8.2 Efficiency and Capacity Utilisation of Ichkheda Percolation Tank

S. N.	Period (Months)	Impounding of water (Days)	Gross Storage (Th M^3)	Evapo. Losses (Th M^3)	Net perco--lation (Th M^3)	Percolation efficiency (%)	Capacity Utilisation (%)
1	2	3	4	5	6	7	8
1.	June to October, 95	85	63.18	1.08	62.10	98.3	140
	Dried on October 26, 1995						
2.	June to October,96	135	143.30	4.06	139.24	97.2	318
	Nov. 96 to January 97	64	11.5	0.92	10.58	92.0	26
	TOTAL (2)	199	154.80	4.98	150.82	97.4	344
3.	June to October, 97	139	82.9	3.94	78.96	95.2	184
	Nov.97 February 98	97	35.8	1.67	34.13	95.3	80
TOTAL (3)		236	118.7	5.61	113.09	95.3	264

- Net percolation is difference of column(4) and (5) i.e. (4)-(5)
- Percolation efficiency is ratio of column (6) to (4) i.e. (6)/(4)as %
- Capacity utilisation is ratio of column (4) to the maximum storage capacity of tank at FSL i.e. 45 Th M^3.

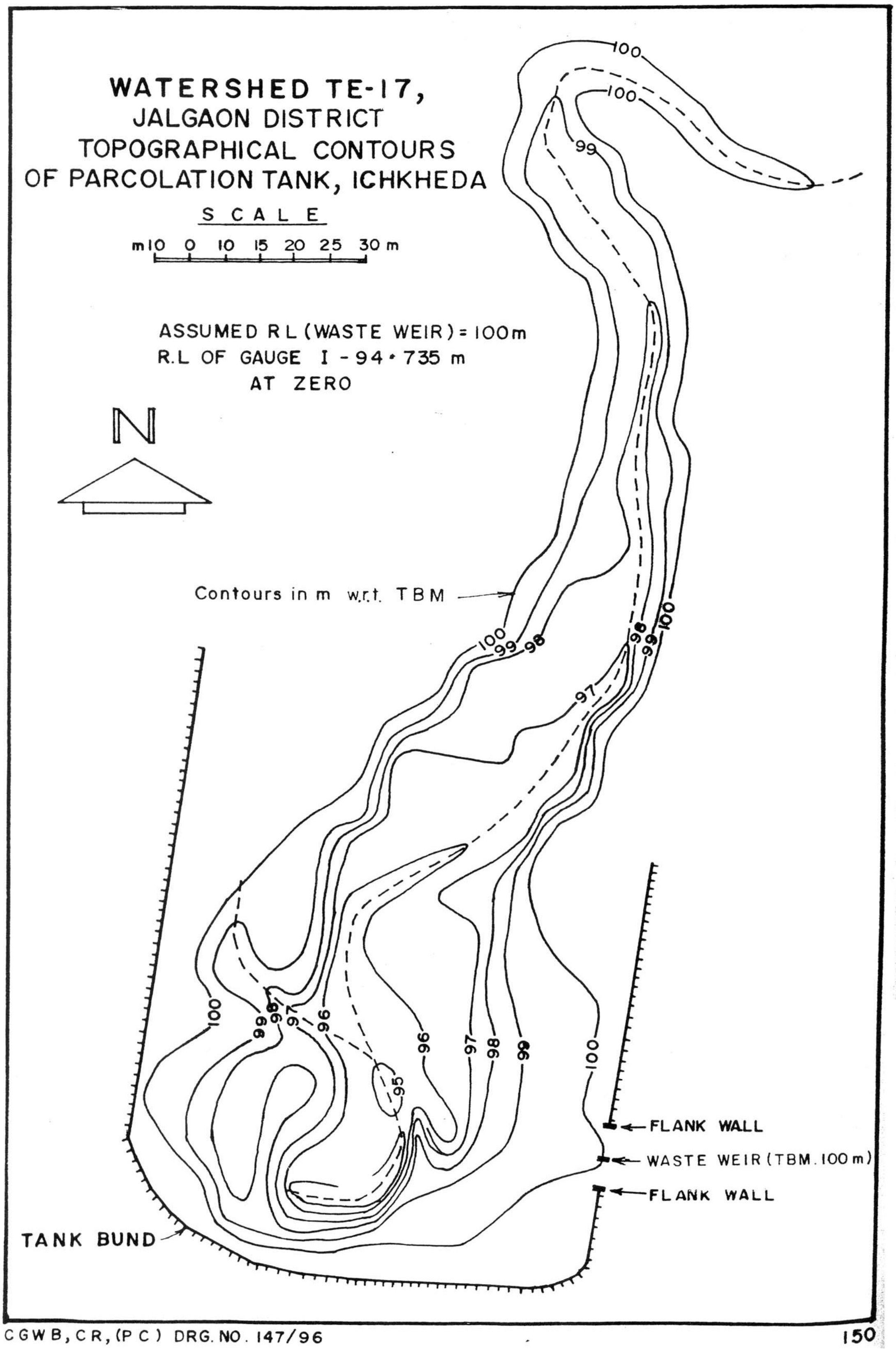


Fig. 8.1 Topographic Contours of Percolation Tank at Ichkheda.

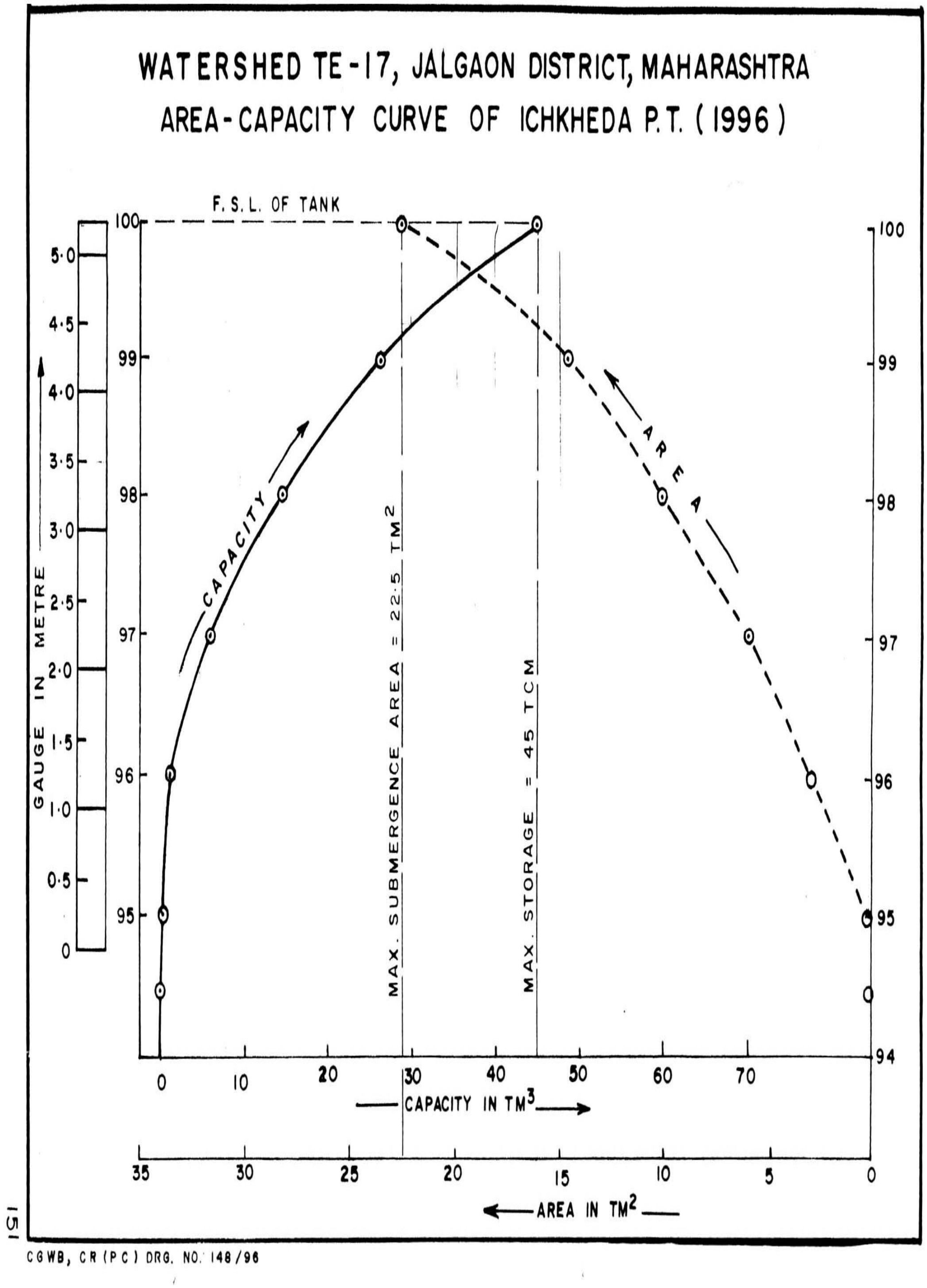


Fig. 8.2 Area-capacity Curve of Ichkheda Percolation Tank.

8.4.4 Monitoring of Impact of Recharge

To demarcate the area of influence, water level data measured in a network of open wells has been analyzed. The water level in the tank and hydrographs of 5 wells were drawn as shown in **Fig.8.3**. It is observed that the hydrographs of wells located up to 1.5 km away from the percolation tank are showing nearly flat apex between August to March. The rise in depth to water level of the wells commenced in the month of July and attained the shallowest depth in September, whereas areas not influenced by the percolation tank show shallowest water levels in November. The specific observations and enquiries with the local farmers have also revealed that the impact of this tank is visible up to 1.5 km downstream. The area of influence is estimated to be about 80 ha. up to a distance of 1.5 km downstream of the structure (**Fig. 8.4)**).

The benefited area is cultivated with cash crops. Sugarcane is also grown in the area. About 25 dug wells in this zone have been benefited due to artificial recharge from this percolation tank. The farmers in the area have switched over to more water intensive crops and new dug wells are being constructed which will bring more area under ground water irrigation. The recharge from percolation tank has resulted in the sustained yield of ground water during the summer. The rise in pre monsoon water level up to 2 m was observed during 1996 and up to 6 m in 1997 with respect to 1995. The increase in pumpage hours of dug wells by 2-3 hours per day during Rabi and 1-2 hours in summer was also observed. It is estimated that an additional recharge of about 150 TCM can bring up to 30 ha of more area under assured irrigation during Khariff and Rabi seasons, considering an average crop water requirement of 0.50 m/year.

8.4.5 Impact of Recharge on Chemical Quality of Ground Water

The impact of ground water recharge on the quality of ground water was also studied. The chemical analysis of percolation tank water and water samples collected from 4 observation wells located within the area of tank's influence were used for the study. The comparison of concentration of various elements present in the percolation tank water, native and augmented ground water is shown in **Table 8.3**

A perusal of Table 8.3 indicates that

1. The pH of tank water is almost neutral (7.1). The overall quality of tank water is superior to the ground water quality.
2. Concentration of all the elements in percolation tank is significantly less than ground water except concentration of K^+ and $SO_4^=$ which is more in tank water.
3. The modification in the native ground water were positive as ground water in the benefited command showed lesser concentration of all the elements except Ca^{++} which is not significantly different in either cases.
4. The concentration of NO_3^- is more in the tank water than the ground water samples collected from the command of the tank.

The overall study indicates that the recharge from percolation tank has improved the chemical quality of ground water in the benefit zone.

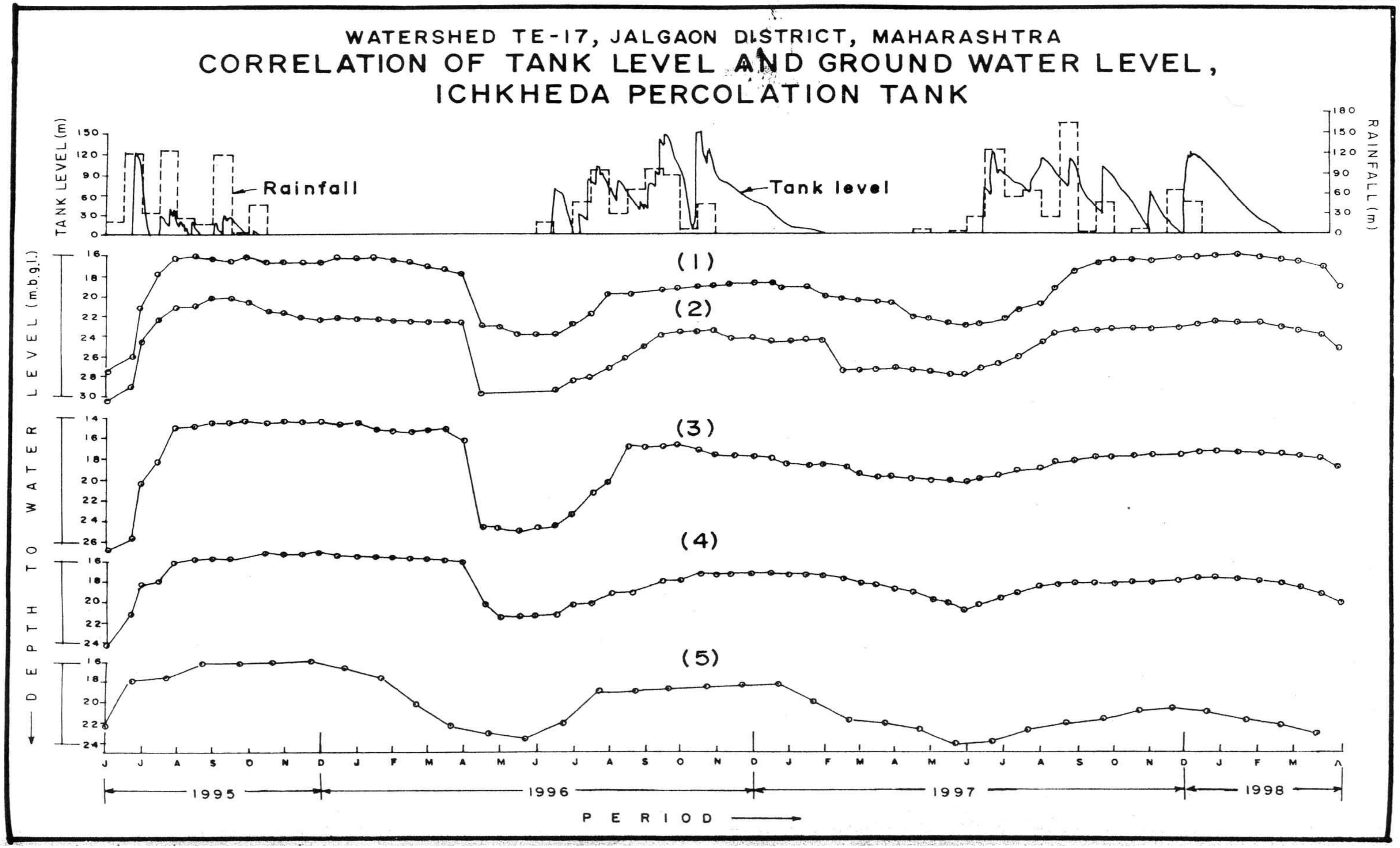


Fig. 8.3 Correlation of Tank Level and Ground Water Levels in Observation Wells, Ichkheda Percolation Tank.

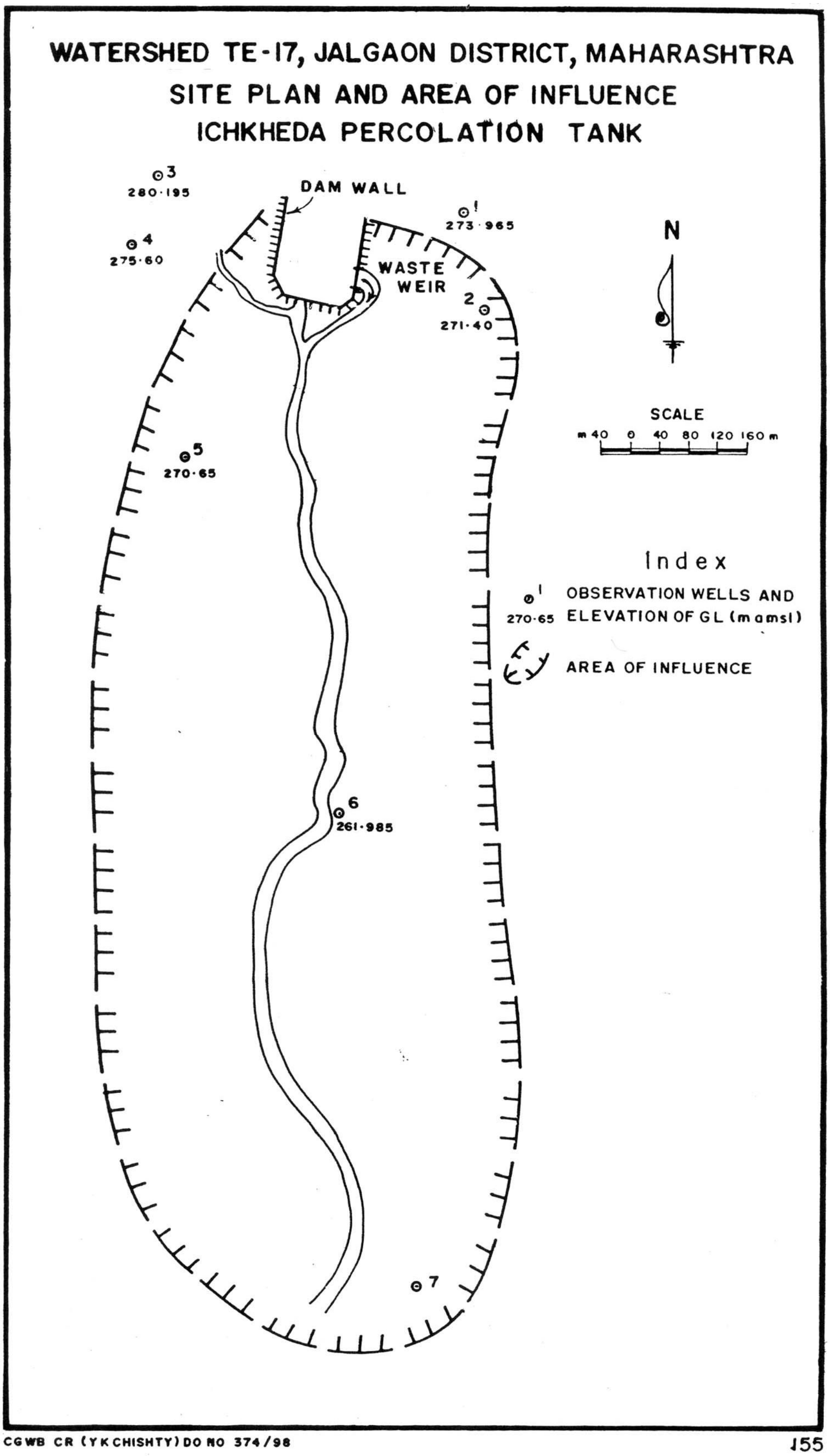


Fig. 8.4 Area of Influence of Percolation Tank at Ichkheda.

Table 8.3 Tank and Ground Water Quality, Ichkheda Percolation Tank

S.No.	Chemical Constituents	Tank Water	Native Ground Water	Augmented Ground Water
1	PH	7.1	8.6-8.7	8.6
2	Ec at 25 ^{0}C (μ S/cm)	250	610-670	500-520
3	TH as C_aCO_3 (mg/l)	110	245-280	215-235
4	Ca^{++} (mg/l)	36	44-66	48-56
5	Mg^{++} (mg/l)	5	23-33	18-25
6	Na^{+} (mg/l)	1.1	21-41	15-18
7	K^{+} (mg/l)	9	<1to 2	Nil-<1
8	$CO_3^{=}$ (mg/l)	Nil	24-36	18-24
9	HCO_3^{-} (mg/l)	122	256-311	238-268
10	Cl^{-} (mg/l)	3	14-21	11-18
11	$SO_4^{=}$ (mg/l)	5	Tr-2	Nil-Tr
12	NO_3^{-} (mg/l)	17	7-10	1-2
13	Fluoride (mg/l)	0.1	NA	NA

8.5 Impact Assessment of Schemes Completed by CGWB

Central Ground Water Board has implemented a number of pilot schemes for popularisation of cost-effective technologies for artificial recharge of ground water during the 8th and 9th Plan Periods. Various structures like check dams, percolation ponds, recharge shafts and subsurface dykes were constructed in different hydrogeological settings during these periods. Impact assessment of the schemes has been carried out using direct/indirect methods. Results of the impact assessment of the schemes are shown in **Table. 8.4.**

Table 8.4 Results of Impact Assessment of Artificial Recharge Schemes Implemented by Central Ground Water Board.

Sl. No.	State / Union Territory	No. of schemes for which impact assessment done	Artificial Recharge Structures	Impact assessment
1.	Andhra Pradesh	6	Percolation Tanks	4500-5900 Cubic meter runoff water recharged in one year
		3	Check dams	1000-1250 Cubic meter runoff water recharged in one year
		1	Combination of recharge pits and lateral shafts	370 Cubic meter runoff recharged in one year
2	Arunachal Pradesh	1	Roof Top Rainwater Harvesting	**7000 cubic meter runoff water harvested in one year**
3.	Assam	1	Roof Top Rainwater Harvesting	5500 Cubic meter runoff water harvested in one year
4.	Bihar	1	Roof Top Rainwater Harvesting	4700 cubic meter runoff water recharged in one year
5.	Chandigarh	6	Roof Top Rainwater Harvesting	1440-13,000 Cubic meter runoff water recharged in one year
		1	Rainwater Harvesting through Roof Top & Pavement catchments	34.50 lakh cubic meter runoff water recharged in one year
		1	Recharge Trenches	9.50 lakh cubic meter rainwater runoff recharged in one year
6.	Gujarat	3	Rainwater Harvesting through Roof Top & Pavement catchments	11000-45000 runoff water recharged in one year

Sl. No.	State / Union Territory	No. of schemes for which impact assessment done	Artificial Recharge Structures	Impact assessment
7.	Haryana	1	Roof Top Rainwater Harvesting	2350 Cubic meter runoff water recharged in one year
		1	Combination of Recharge shafts and injection wells	3.50 lakh cubic meter runoff water recharged in one year. Declining rate reduced from 1.175 m/yr to 0.25 m/yr.
8.	Himachal Pradesh	3	Check dams	1.20-21.00 lakhs cubic meter runoff water recharged in one year.
9.	Jammu and Kashmir	2	Roof Top Rainwater Harvesting	300-1200 Cubic meter runoff water harvested in one year
10.	Jharkhand	1	Roof Top Rainwater Harvesting	4500 cubic meter runoff water recharged in one year.
11.	Karnataka	1	Combination of Percolation Tanks, Watershed Structures, Recharge wells, Roof Top Rainwater Harvesting	2-3.5 m. rise in water levels and 9-16 ha area benefited from percolation tanks 8.60 lakh cubic meter water recharged through recharge well. 3-5 m rise in ground water levels through watershed structures. 530 cubic meter recharged from Roof Top Rainwater Harvesting.
12.	Kerala	1	Sub-surface Dyke	Augmented 5000 Cubic meter of ground water in upstream side with 2 m rise in groundwater levels.
		1	Recharge wells	2800 Cubic meter runoff water recharged in one year
		3	Percolation tanks	2000-15000 Cubic meter runoff water recharged in one year
		1	Tidal regulator	4000 Cubic meter runoff water conserved and a difference of 1.5 m was observed in upstream and downstream water level.
		2	Check Dam	5,100 - 30,000 Cubic meter runoff water recharged in one year

Sl. No.	State / Union Territory	No. of schemes for which impact assessment done	Artificial Recharge Structures	Impact assessment
13.	Laksha-dweep	1	Roof Top Rainwater Harvesting	300 Cubic meter rainwater harvested in one year
14.	Madhya Pradesh	4	Sub-surface Dykes	Rise in water level in dug wells in the range of 0.80-3.80 m and 6-12 m in hand pumps have been observed.
		1	Percolation Tank	Rise in ground water levels by 1-4 m. in command area downstream of tank has been observed.
		1	Roof Top Rainwater Harvesting (1000 houses)	More than 2 lakh cubic meter runoff water recharged in one year.
		1	Combination of sub-surface dykes and check dam	Rise in water levels in existing tube wells in upstream area by 0.30 m to 2.00 m has been observed.
15.	Maharashtra	2	Roof Top Rainwater Harvesting System	196-280 cubic meter runoff water recharged in one year
		1	Combination of Percolation Tanks and Check Dams.	Benefited area – About 60 to 120 ha. per Percolation Tank, 3 to 15 hectare per Check Dam Water level rise – Upton 1.5 m.
		1	Percolation tanks, Recharge Shaft, Dug well Recharge.	Benefited area – 400-500 hectare around the scheme.
16.	Meghalaya	1	Roof Top Rainwater Harvesting	6800 cubic meter runoff water harvested in one year
17.	Mizoram	1	Roof Top Rainwater Harvesting	50,000 cubic meter runoff water harvested in one year
18.	Nagaland	3	Roof Top Rainwater Harvesting	2,480 – 14,065 cubic meter runoff water harvested in one year

Sl. No.	State / Union Territory	No. of schemes for which impact assessment done	Artificial Recharge Structures	Impact assessment
19.	NCT Delhi	2	Check dams	Water levels have risen unto 2.55 m in the vicinity of Check Dams and area benefited is unto 30 hectare from each check dam in JNU & IIT. 1.30-lakh cubic meter of rainwater was recharged in one year in Kushak Nala.
		7	Roof Top Rainwater Harvesting	800 – 5000 Cubic meter runoff water recharged in one year
		8	Rainwater harvesting through Roof Top & Pavement catchments	8500 – 20,000 cubic meter runoff water recharged in one year
20.	Orissa	1	Rainwater harvesting through Roof Top & Pavement catchments	1,200 cubic meter runoff water recharged in one year
		1	Renovation of creeks & sub - creeks, Construction of Control Sluices and recharge bore wells	Quantity of fresh water impounded in 798119 cubic metres and irrigation potential is 11000 has in a year.

Sl. No.	State / Union Territory	No. of schemes for which impact assessment done	Artificial Recharge Structures	Impact assessment
21.	Punjab	1	Roof Top Rainwater Harvesting	500 cubic meter runoff water recharged in one year
		3	Recharge wells	9 – 15.50 lakhs cubic meter runoff water recharged in one year.
		1	Trenches	Average rise in water level unto 0.32-0.70 m has been observed.
			Combination of vertical shafts, injection wells & recharge trenches	Recharge of 1.70 lakh cubic meter runoff water caused average rise of 0.25 m. in ground water levels around the scheme area.
		1	Combination of recharge shafts and injection wells	14,400 Cubic meter runoff water recharged in one year.
22.	Rajasthan	1	Check dams	88,000 Cubic meter runoff water recharged in one year. Water level rise - 0.65 m.
		12	Roof Top Rainwater Harvesting	350-2800 Cubic meter runoff water recharged in one year.
		3	Sub-surface Barriers	2000-11500 Cubic meter runoff water recharged in one year. Water level rise from 0.25 to 0.60 m.
23.	Tamil Nadu	1	Sub-surface Dyke	39.25 ha. area benefited.
		7	Percolation Tanks	10,000-2,25,000 cu. m runoff water recharged in one year.
		1	Roof Top Rainwater Harvesting	3700 cubic meter runoff water recharged in one year
24.	Uttar Pradesh	7	Roof Top Rainwater Harvesting	350-23033 cubic meter runoff water recharged in one year
25.	West Bengal	1	Combination of Farm Ponds, Nala Bunds, Sub-surface Dykes	Water level rise of 0.15 m. observed.
		1	Sub-surface Dykes	Rise in water levels by 0.45 m. observed

9. ECONOMIC EVALUATION OF RECHARGE SCHEMES

Economic viability is a critical parameter to be ascertained before taking a decision to implement any artificial recharge scheme. The appraisal of economic viability has to be carried out after taking into account all possible expenses including those for investigation, source water (conveyance, treatment), construction of recharge structures, operation and maintenance etc. All benefits should be appropriately accounted for and assessed in order to decide the acceptability of the scheme as per its priority in the overall scheme of development. Economic analysis of artificial recharge projects aims at ascertaining their economic and financial viability.

9.1 Benefit Cost Analysis

It is important to carry out the Benefit Cost Analysis for all major public works before a decision is taken on the allocation of funds. The Benefit Cost Analysis presents the quantifiable efforts, environmental and social aspects of any public projects in money terms.

The analysis of the financial benefits and costs requires expressing the cash flow elements under the non-financial operations in comparable terms. Costs are related to investments occurring during the lifetime of the project. However, benefits originate from the productive use of the projects. Therefore, both costs and benefits are expressed in quantitative terms and translated into monetary terms by using market values of the inputs and outputs concerned. Actually, the costs and benefits occur at different points of time. In order to make them comparable, it is customary to express both in terms of their present value by applying the appropriate discounting factors. After accounting both the costs and benefits against their market values, appropriate criteria are applied to determine the profitability of the project.

The Benefit Cost analysis of the projects is sometimes also called Project Appraisal. This Project Appraisal is done before the decision is taken to invest, whereas evaluation sometimes is done to analyze the performance and effects of the project after it has been executed.

The most important factors of project appraisal are Financial, Economic and Social Benefit Cost Analysis next to Institutional, Environmental, and Social impact assessments.

9.1.1 Financial Benefit Cost Analysis

The financial Benefit Cost analysis views any project from the point of view of the INVESTOR. The analysis would suggest/indicate the investor whether it is worthwhile to invest in the project in comparison of other investment opportunities. In practice, the investor may be a private person, a non-government Organization, or a government institution.

In evaluating the advantages of an investment opportunity, it is essential to give proper weightages to two major aspects, i.e., Liquidity Analysis and Profitability Analysis.

The liquidity analysis would demonstrate whether, for the entire lifetime of the project, receipts from equity capital and borrowing plus the annual income (Cash inflows) will be sufficient to meet the obligations for payments to be made (Cash outflows.)

Cash inflows, in this context, comprise the following elements.

i) Investment funds, which may consist of equity capital or loans,
ii) Loans and credit, during operation, and
iii) Revenues from sales and subsidies.

On the other hand, Cash Outflows to be considered include

i) Investments in fixed assets, working capital, pre-investment costs, preparatory surveys etc.,
ii) Interest, dividends and repayments and
iii) Direct payments.

For the project to be economically viable, the profitability analysis should show that various sources of finance involved would yield an acceptable financial return.

9.1.1.1 Measure for Profitability

This analysis becomes very crucial for identifying better opportunities for the investor's money. A number of methods have been developed to measure the profitability of investments. The commonly used methods are, i) Benefit Cost Ratio (B/C) Ratio), ii) Net Present Value (NPV) and iii) Internal Rate of Return (IRR). These methods are described below in brief.

a) **Benefit Cost Ratio**: The Benefit Cost (B-C) ratio, also known as ‘Profitability Index’ (PI) or ‘Desirability Factor’ is being widely used in the initial stages of project appraisal. It is defined as:

$$\text{B-C Ratio} = \frac{\text{Present value of total benefits}}{\text{Present value of total costs}}$$

If B-C ratio > 1, the project is considered to be attractive and profitable.
If B-C ratio <1, the project would not earn the inputs back and are thus not recommended for execution.

Limitation:

Without more information such as net benefits of running costs, cost escalation considered for gross benefits etc., the B C ratio is not well defined.

Hence, NPV and IRR should be considered in addition to B C ratio for proper evaluation of projects.

b) **Net Present Value (NPV) :** The Net Present Value is uniquely defined and widely used in the selection of ground water development projects and artificial recharge projects. It is defined as the difference between the present value of total benefits and the present value of total costs. In this method, the inflows and outflows expected in future are discounted for a change in the value of money. Accordingly, cash flows expected in future years are discounted and their values at the beginning of the project are arrived at. Discounting presumes that money, like the other factors of production, has a cost. Discounting is done at the interest rate, which is the cost of capital, known as the discount rate. Interest rate and discount rate are practically the same, the only difference being the point of view. Interest assumes looking from the present to the future, whereas discounting look backwards from the future to the present.

The net present value of investment proposal is computed as

$$\sum_{t=0}^{n} \frac{cft}{(1+k)^t}$$

Where

cft = Cash flow occurring at the end of year't',
n = Life of the project and
k = Cost of capital used as the discount rate.

If NPV> 0 (Positive), the project is considered to be profitable and will yield more benefits than the investments.

The following example describes the procedure for calculation of NPV.

Assuming 'n' as 5 years, 'k' as 10%

Years	0	1	2	3	4	5
Net Cash flow	-1000	200	200	250	350	400

$$\textbf{NPV} = \frac{-1000}{(1+0.1)^0} + \frac{200}{(1+0.1)^1} + \frac{200}{(1+0.1)^2} + \frac{250}{(1+0.1)^3} + \frac{350}{(1+0.1)^4} + \frac{400}{(1+0.1)^5}$$

= 22.1

Hence, the project can be accepted as the NPV is positive.

c) **Internal Rate of Return (IRR):** Though the NPV, which gives the net present value in absolute terms and Benefit Cost ratio, which gives the ratio of profit to cost consider the time value of money, neither of these methods indicate the rate of return. The Internal Rate of Return is a measure of the return on the investment that the project yields. It is the discount rate that equates the present value of cash inflows with the present value of outflows of the project. In other words, it is the discount rate that causes a project's net present value to equal zero and profitability index to equal unity.

It is represented by the rate 'r' such that,

$$\sum_{t=1}^{n} \frac{cft}{(1+r)^t} = 0 = NPV$$

Where,

cft = Cash flow for period 't' whether it be a net cash out flow or inflow.
N = Life of the project.

If the initial cash outlay or cost occurs at time '0', the above calculation cab be expressed as,

$$Cf\ 0 = \frac{Cf_1}{(1+r)^1} + \frac{Cf_2}{(1+r)^2} + \frac{Cf_3}{(1+r)^3} + ... + \frac{Cf\ n}{(1+r)^n}$$

The rate 'r' discounts the stream of future cash flow through $cf_{1......}cf_n$ to equal the initial outlay at time '0'

The accepted criteria generally employed for the IRR method are to compare the IRR with the required rate of return known as "cut off rate. If IRR exceeds the required rate, the project is acceptable.

The following example describes the procedure for calculation of IRR.

n = 5 years

Years	0	1	2	3	4	5
Net Cash flow	-1000	200	200	250	350	400

IRR is the value of 'r' which satisfies the equation NPV = 0.

$$1000 = \frac{200}{(1+r)^1} + \frac{200}{(1+r)^2} + \frac{250}{(1+r)^3} + \frac{350}{(1+r)^4} + \frac{400}{(1+r)^5}$$

The value of 'r' is calculated by trial and error method

For r = 10%, NPV equals to 22
For r = 10.5%, NPV equals to 7.5

Therefore, IRR will be nearer to 10.5%

9.1.1.2 Interest and Inflation

In financial analysis the rate of interest to be used is the actual interest to be paid for financing of the project. Generally, it is the market rate of interest or interest foregone if the necessary funds are withdrawn from a bank account or other investment opportunities.

Sometimes, if the funds allotted for the project are offered at an interest rate that deviates from the market rate in positive or negative way, the actual (annual) costs of interest and repayments have to be taken into account.

For all practical purposes, the base of the profitability analysis of any project should be an assumption of constant prices. But it is nearly impossible to control the inflation as such, and the assumption of constant prices is only justified as long as the relative value of inputs and outputs does not change over time.

9.1.1.3 Uncertainties and Sensitivities

The uncertainties involved in the calculation of benefits and costs should be minimal so that the outcome of any Benefit Cost analysis is realistic and not over-optimistic. However, these variables could be related to the level of costs and incurrence of cost. In order to avoid too low a cost, it is customary to include a contingency allowance of 20-25%. An extended construction period will decrease the present value of costs.

The quantitative assessments of benefits in some cases are fairly accurate but are not that easy in most of them. They vary widely depending on the efficiency of the future operator or the actual value of the benefits remaining far below the expectations. As an example, some of the irrigation projects or artificial ground water recharge schemes in which the peoples'/ farmers participation is not adequate may yield benefits below expectations. An extended construction period will decrease the present value of benefits.

To verify the effect of the changed conditions, it is recommended to carry out a sensitivity analysis so that information regarding the level and time of occurrence of critical data of benefits and costs vary.

9.1.2. Economic Benefit Cost Analysis

The main component of economic benefit cost analysis is to evaluate the resources in a national context. The analysis views it from the national point of view such that the benefits add to the national economy.

There are certain aspects, which are to be fully understood for such analyses to be correct and realistic. The effects of the projects such as external effects, valuation of benefits and costs, labor wages and perfect completion are to be studied in detail. Though these effects constitute a cost or benefit to the society, they are not reflected in the project's financial receipts or in expenditures. In addition to this, some aspects of ecological damage and pollution of soil, water or air may have to be taken into account in the analysis. It follows that the valuation inputs and outputs of the project differ in financial and economic analysis. Hence, it is customary to introduce certain conversion factors to transform financial prices into economic prices.

9.1.2.1 Conversion Factors

The financial analysis simply uses the market prices of inputs and outputs whereas in economic analysis, the prices that express the real scarcity of the inputs and outputs have to be used. Some conversion factors are required to transform financial prices into economic prices. These factors are derived according to the commodity and the region of operation. In many cases, however, it is neither possible nor practical to use specific standard conversion factors.

9.1.2.2 Capital and Interest

The valuation of capital in terms of the rate of interest or the rate of discount constitutes the economist's interest for evaluating any project proposals. Sometimes, it becomes beyond the limits of the planner to determine the economic rate of interest (accounting rate of interest). In fact, a responsible government agency should determine the rate of interest to be used in the economic analysis in such a way that it helps in adopting a uniform rate for all projects and the results of the analysis comparable.

9.1.2.3 Economic Appraisal

The economical appraisal of any ground water development/recharge project is critical as the benefits are not only indirect but also time-consuming. Certain guidelines for economic appraisal are summarized below:

a. The inputs and outputs should be distinguished as 'tradables' and 'non-tradables'.
b. The assumption always is that the project under consideration will not change the price of the output.
c. Compared to the calculation in financial prices, some adjustments have to be made by applying appropriate conversion factors converting financial prices into economic prices.
d. The economic analysis should not only consider the effects for the producer, but also for the user.
e. Labor and wages under skilled and unskilled categories have a special significance in the valuation for economic analysis. The real contribution to the economy probably varies by region, type of labor and season. Hence, an extensive labor market survey is required for proper analysis.
f. Although the computational part of the appraisal is rather straightforward, the essential purpose of the exercise is to ensure that the project has a positive effect on the efficient application of national resources.
g. The outcome of the economic appraisal of a development project is decisive for the acceptance of the project.
h. If the project is acceptable from the economic point of view but not from the financial, it implies that the project will contribute to an efficient application for national resources with additional requirement of financial support.
i. If the project yields attractive returns to the Government but does not make a contribution to the efficient use of national resources, it requires additional policy measures to rectify the situation.

9.1.3 Social Benefit Cost Analysis

The financial and economic benefit cost analyses are concerned mainly with the profitability of investments and on the efficient use of national resources. However, it is to be borne in mind that not only the growth of the wealth and welfare, but who gets the benefit out of it is also equally important.

Sometimes, a ground water development/recharge project may have excellent financial and economic returns but the benefits are distributed to only a small group of people who are already relatively well-off. In such cases, social justice has not occurred. In other words, not only the growth but the distribution of benefits is important. This is the essence of Social Benefit Cost analysis.

The task becomes easy if the impact of the development project on the distribution of welfare and the proper beneficiaries are identified. Social Benefit Cost analysis is, thus, a part of the project appraisal and is always pursued by major International Financing Agencies.

It should be realized that any project, validated on the basis of social welfare considerations would have a definite recognized price. Hence, as a marker, the project preferred on the basis of social point of view will not be identical to that which is preferred from an economic point of view.

9.2 Socio-economic and Financial Appraisal of Artificial Recharge Schemes

The economic and institutional aspects of artificial recharge ground water are important but somewhat elusive. Experiences with full-scale artificial recharge operations of ground water in India are still limited and, as a consequence, the cost information on such operations is incomplete. The available data in certain hydrogeological environs, where recharge experiments are initiated and are in progress, suggests that the costs of ground water recharge vary substantially. These costs are a function of availability of source water, conveyance facilities, civil constructions, land acquisition, and ground water pumping and monitoring.

Apart from the purely technical aspects of ground water utilization, the economical and institutional problems may ultimately prove more critical in determining the efficiency of the artificial recharge projects. Although literature on the economics of ground water deals with specific problems of ground water management, there are certain common principles, which have to be taken care of for assessment of costs and benefits.

9.2.1 User Cost

The change in storage of ground water at any time is simply the difference between the recharge rate and extraction rate. It is also to be noted that the effective use of ground water is attained when the difference between benefits and costs is maximized in positive direction over time. In fact, the water pumped in the current period results in lowered water table in future periods. Therefore, the incremental cost of pumping from thus lowered level has to be accounted for, and is called the user cost. Under deteriorated conditions of lowered water levels, the overall pumping should be carried over to the point where the benefits from the last unit of water exceed the extraction cost plus the user cost. Similarly, if the ground water levels rise by any other mechanism developed through certain artificial recharge methods, the extraction cost becomes minimized over a time with no user cost. Hence, the change in the user cost over a time is dependent upon the discount rate and the effects of ground water storage on pumping costs.

9.2.2 Steady State Pumping Condition

In certain circumstances, over-exploitation of ground water may be optimal and efficient from economic point of view. This becomes true in cases where the total benefits generated through excessive ground water development are much more or quite high relative to total costs. At this juncture, the over-exploitation may be fully justified from an economic perspective. Of course, it is evident that the over draft conditions could not be continued indefinitely.

In some situations, the ground water level will be lowered until a point is reached at which the costs of extraction are greater than the benefits generated from various uses to which the water has been put. At this point, it is not economical to abstract ground water at rates greater than the recharge rate. Thus, the relative magnitude of costs of pumping and benefits ensure that only the annual recharge is being extracted. This means that the long-term pumping rates should not exceed long term recharge rates for a given aquifer. This ultimate hydrologic condition is referred to as the Steady State (i.e., the difference between the recharge rate and the extraction rate equals zero)

9.2.3 Artificial Recharge Component

For obtaining the economics of artificial recharge of ground water, an artificial recharge component is to be included in the analysis. Additionally, the source water cost and the benefits of use must be also be accounted for. It is essential to establish whether supplemental sources of water should be used for ground water recharge or for direct use. It is economically not viable if the price charged for ground water is less than the marginal cost of recharge. Further, the selection of appropriate artificial recharge structure suitable for the existing hydrogeological environment of the proposed area is a must. Otherwise, the benefit cost analysis and other economic measures for ground water use and management are likely to yield negative results.

The principles mentioned above guide the socio-economic and financial analysis of projects for artificial recharge to ground water. A step by step description of inputs and outputs are shown below for developing benefit cost analysis of recharge projects. Under this, it is necessary to highlight some of the inputs, specially the recharge inputs for some proposed artificial recharge structures, for better appraisal of the benefits.

9.2.4 Recharge Potential of Some Artificial Recharge Structures

Various artificial recharge experiments carried out by different organizations in India have established the feasibility of the methods in unconfined, semi-confined and confined aquifer systems. However, economic considerations make some particular methods viable in a particular area or for a particular aquifer. Consequently, it is possible to estimate upper limits of quantities of recharge through each artificial recharge structure based on studies carried out in different hydrogeological set-ups. Some of the typical recharge estimates are given below in general form with field examples from alluvial unconfined/semi confined aquifer systems.

9.2.4.1 Check Dam & Percolation Tank

i) Average water spread area = 'a' Hectares
ii) Seepage Rate = 'b' Cu m/sec/million sq m of wetted perimeter

iii) Inflow and storage period = 'c' Days

iv) Quantity of induced recharge in MCM

$$= \frac{a \text{ X } 10 \text{ X } b \text{ X } 3600 \text{ X } 24 \text{ X } c}{10^6 \text{ x } 10^6} = \text{'P' MCM}$$

As a case example, assuming the water spread area of each check dam/percolation tank as 10 hectares, inflow and storage period of 100 days, monsoon seepage rate as 2.6 cu m/sec/million sq m of wetted perimeter and considering 4 to 5 floods during the rainy season, surface water recharge of nearly 2.25 MCM through each structure could be considered as realistic. Further, by construction of recharge tube well in the storage area, increase in the quantum of recharge could be ensured.

9.2.4.2 Spreading Channel

i) Total wetted perimeter for full length of spreading channel = 'a' million Sq m
  a. Seepage Rate = 'b' Cu m/sec/million sq m of wetted perimeter
ii) Availability of water in Channel = 'c' days.

iii) Quantity of induced recharge in M Cu m

$$= \frac{a \text{ x } 10 \text{x } b \text{ x } 3600 \text{x } 24 \text{ x } c}{10^6 \text{ x } 10^6} = \text{'q' M Cu m}$$

As a case example, considering the length of spreading channel as 10 km with bottom width of 3 m and top width of 5 m with 1:1 slope, the total wetted perimeter for full 10 km length of spreading channel works out to 0.085 million Sq m. Further, assuming the seepage rate of 10 cu m/sec/million Sq m of wetted perimeter and availability of recharge water for nearly 100 days in a year, it is estimated that a recharge of nearly 5 MCM/yr could be made into the aquifer system.

9.2.4.3 Recharge Tube well

i. Injection recharge rate = 'a' lps
ii. Number of days of recharge = 'b' days
iii. Quantity of recharge in MCM

$$= \frac{a \text{ x } 86.4 \text{ x } b}{10^6} = \text{'R' MCM}$$

Considering the injection rate of 5 lps, nearly 0.15 MCM of water could be artificially recharged through each injection well in a year.

9.2.4.4 Underground Dams /Subsurface Dykes

Considering a cut-off at 9m and area of effective sub-surface storage of 100 hectares in up-stream of each under ground check dam, the specific yield of river bed as 20% and 4 to 5 wet spells during the rainy period of 100 days, it is expected that through each underground dam with 2 recharge tube wells, about 1.0 MCM ('S' MCM) of surface water could be artificially recharged into the aquifer system.

For any programme of artificial recharge to ground water, the above unit recharges estimated for each method should be multiplied by the proposed number of units under each category such that the total cumulative input to the ground water would be quantified. This information is vital for the financial, economic and social benefit analysis.

9.2.5 Financial Outlay

For arriving at the cost outlay for the artificial recharge projects, it is essential to identify the mechanism through which the whole process occurs. It is not only the costs of the structures of the proposed measures but also certain relevant costs involved in pre and post project studies which are essential to be included in the total costs. Costs involved for the investigative techniques such as Hydrogeological, Hydrometeorological, Hydrological, Geophysical and Geochemical studies for identifying suitable locations/areas for implementing artificial recharge schemes should be included. If possible, financial assistance should be provided to organize small pilot study projects before undertaking any major projects.

It is also necessary to take environmental aspects into consideration and, hence, financial support should be provided to the afforestation works in the vicinity of the project area. This attempt will enhance the ground water recharge, reduce soil erosion and improve the health of the watershed.

Watershed management through soil management and water conservation methods provides an enhanced ground water recharge into the flow system. Therefore, it is recommended that appropriate fund allocation for watershed management should be provided.

Monitoring of the ground water regime for assessing the sustainability of the project objectives and benefits requires certain committed funds. The funds may be utilized for procuring instruments, setting up of laboratories, research and development etc.

The summarized details of Cost Outlay of artificial recharge projects are given below:

a.	Cost of pre-investigative Studies	Rs. CR1
b.	Cost of afforestation works	Rs. CR2
c.	Cost of watershed management works	Rs. CR3
d.	Cost of construction of the suggested measures	Rs. CR4
e.	Cost of monitoring ground water regime	Rs. CR5

	Total cost of project	Rs. CR

9.2.6 Benefits of Suggested Measures

As mentioned earlier, the present day trend of over exploitation of ground water has resulted in faster depletion of water levels. As a result, the users have to periodically replace/repair the pumps and in many cases re-drill tube wells. This phenomenon, which has now become common in many parts of the country results in very high operational costs toward ground water development over a period of time.

Augmentation of ground water resources in such areas will not only help in bringing up or in stabilizing of water levels but will also reduce the user's financial commitments toward the replacement of pumps or re-drilling of tube wells.

Since artificial recharge of ground water is a time-consuming process, the benefits would be felt only over a period of time and will mostly be of indirect nature, as the measures adopted are mainly oriented towards protecting and improving the natural ground water environment.

It is fair to assume that once the aquifer system is augmented with additional recharge component, the institutional finance for ground water development will be available to the users. The indirect benefits, which are economical as well as social, could be summarized as below:

a. Control over further depletion of ground water levels, obviating the need for replacement of high head pumping machinery.
b. Sustained abstraction of ground water ensures long term irrigation, manifold increase of agricultural area and economic cropping patterns.
c. Minimization of frequency of re-drilling of tube wells over time
d. Changes in the energy consumption scenario due to rise/ stabilization of water levels.
e. Restoration of well irrigation in areas where wells have gone dry.
f. Provision of drinking water facilities in habitations hitherto having no such sources
g. Increase in employment potential by using local labor either skilled or semi-skilled.
h. Increase in per-capita income of the local people resulting in better living standards. People's participation in the development work enhances the benefits.
i. Restoration of institutional finance not available earlier for construction of wells / tube wells in overexploited areas.
j. Environmental improvements helping in reduction of pollution hazards.

9.2.7 Financial Appraisal of the Benefits

It is slightly elusive to view the indirect benefits in real financial terms. In order to have a near-realistic assessment, the financial amounts are shown in general form below.

i) Considering 'Z' MCM of additional recharge to ground water which otherwise goes as surface Runoff, its total value even at a rate of Rs. 0.1 per 1 m^3 works out to rupees 'R1.'

ii) The replacement of pumps, say every 5 years for about 100 tube wells per year result in saving of Rupees 'R2' per year (Cost of the pump around Rs. 7,500).

iii) On an average, 100 tube wells are re-drilled every year. The annual savings on this account are expected to be nearly Rupees 'R3' (Average cost would be around Rs.1,75,000 per well in alluvial area).

iv) Considering the electrical energy saving of 500 KW per tube well per year, the total savings for 'X' number will be of the order of 500 x 'X' KW. Even if valued at the rate of Rs. 1.0 per KW, the total annual saving could be of the order of nearly Rupees 'R4'.

v) Considering a surplus ground water potential of 'Z1' MCM through the above measures after meeting the existing abstraction, an additional irrigation potential of nearly Z1 MCM x h hectare = H hectares (1 MCM irrigates 100 to 150 hectares under normal cropping pattern) is created.

vi) Considering an average return at the rate of 'r' Rupees per hectare under the existing cropping pattern, the additional income from agricultural return is likely to be H x r = 'R5' Rupees (one Hectare yields an average annual return of Rs. 15,000 under existing cropping pattern).

vii) The financial benefits are summarized below for assessing the benefit cost ratio of measures for artificial recharge to ground water (**Table 9.1)**

Table 9.1 Summarized Financial Benefits of Artificial Recharge Schemes

a.	Cost of surface water which goes as runoff	Rs. R1
b.	Savings against pump replacements	Rs. R2
c.	Savings against re-drilling of tube wells	Rs. R3
d.	Savings in electrical energy consumption	Rs. R4
e.	Income form additional agricultural production	Rs. R5
f.	**Total financial benefits**	**Rs. BR**

9.2.8 Profitability Analysis

The benefit cost ratio is the ratio of present value of total benefits to the present value of total costs. This ratio expression can be slightly enlarged to suit the ground water recharge projects, which are mainly financed by State or Central Government Departments. In most such projects, the returns to the Government are minimal when compared to investment of capital on other projects.

Therefore, it is assumed that the annual benefits of 'BR' Rupees will offset the capital cost investment of Rupees 'CR' in a tolerable period of say 'Y' years. Application of this criterion is essential to determine the profitability of the project. For working out a B/C ratio, it is customary to take the ratio of total benefits to the Annual Cost of expenditure (in Rupees 'AR') for the ground water development projects.

This annual cost of expenditure includes i) Interest loss at the market rate of the total cost, ii) Maintenance & Repair charges at the feasible percentage rate of the total cost iii) Depreciation of civil works at the rate permissible, and iv) Some miscellaneous expenditure at a reasonable percentage of the total cost.

The above approach is detailed below for the estimation of B/C Ratio (**Table 9.2)**

Table 9.2 Computation of Annual Cost of Expenditure

A. Annual Total Benefits	Rs. BR
B. Cost of the Total Project	Rs. CR
C. Annual Expenditure a) Interest loss 'n' say 10% b) M & R charges say 2.5% c) Depreciation of civil works n say 5% d) Miscellaneous expenditure n say 1% **Total**:	 Rs. AR1 Rs. AR2 Rs. AR3 Rs. AR4 ---------------------- **Rs. AR**

Therefore, The Overall Benefit Cost Ratio = $\dfrac{\mathbf{BR}}{\mathbf{AR}}$

If the B/C ratio is greater than 1, the project is considered to be attractive. As most of the ground water recharge projects belong to 'social obligatory' type of expenditure on the part of Government, weightage towards the B/C Ratio should be viewed with less priority

9.3 Case Study

Conservation of water through artificial recharge is often the only alternative in drought- prone areas. Construction of percolation tanks is practiced in Maharashtra to conserve and recharge the ground water in drought prone areas of the State.

A detailed study of 7 percolation tanks in parts of Baramati taluka of Pune District covering an area of 66 Sq.km with an average storage capacity of 0.13 MCM was taken up for financial analysis to see whether they are cost effective or not.

Based on the study, Dr. S.S.Rao of NABARD has concluded that the financing of percolation tanks is not economically viable without any subsidy from Government. The tanks not only serve for recharging the ground water but also serve as community tanks, are environment friendly and help control soil erosion. Therefore, it was recommended based on the study that a minimum 75% of subsidy should be allowed for construction of tanks. Similarly, 30% of subsidy should be allowed for the construction of wells and for the costs of pump sets. The summarized results of the case study given below indicate the required percentage of subsidy for keeping the project cost effective and viable (**Table 9.3).**

Table 9.3 Summarized Results of Case Study

Sl. No.	Indicator	Subsidy Nil Well (30%) P.Set (30%)	Subsidy for Tank (50%) Well (30%) P.Set (30%)	Subsidy for Tank (75%)
1.	B/C ratio at 15% discounted rate	0.50	0.82	1.16
2.	RR at 15% discount	0.38	9.11	19.49
3.	Water rate in Rs/Cu m	3.66	1.71	1.60 1.08 Estt. & OM Tank (100% subsidised)
4.	Repayment Schedule % of repayment to net incremental income 1st year 2nd to 9th year 10th to 15th year	 182.50 237.30 221.70	 96.40 126.10 115.20	 57.30 76.10 65.20

As seen from above, the scheme is not financially viable unless the Government provides a subsidy of at least 75% for tank and 30% for well and pump sets and also provide the charges for maintenance and establishment of tank during its construction and subsequent maintenance.

A formatted example on financial analysis of an artificial recharge Scheme is given below for a better understanding of the computational procedures (**Table 9.4**).

Table 9.4 Format with an Example of Financial Analysis of Artificial Recharge scheme.

	A) Scheme Information	
1.	Type of scheme	Percolation tank.
2.	Location	Baramati Taluka, Pune District.
3.	Capacity of percolation tank	0.13 MCM
4.	Total irrigated area prior to scheme	12 ha.
5.	Additional irrigated area after scheme	10 ha.
6.	Additional ground water structures after scheme	6 wells with pump sets
7.	Life of the scheme	15 years
	B) Investment Information	
1.	Construction cost of AR scheme @ Rs. 9,000/1000 cu m	Rs.12,48,000
2.	Cost of 6 Nos. additional wells @ Rs.22,500/-well	Rs.1,35,000
3.	Cost of 6 nos. of pump sets (5 HP) @ Rs. 12,100 per set	Rs.72,600
4.	Total cost of investment	Rs. 14,55,600
5.	Government subsidy on construction cost of percolation tank (75%)	Rs. 9,36,000
6.	Government subsidy on wells and pump sets (30%)	Rs. 62,280
7.	Cost of investment after subsidy	Rs. 4,57,320
8.	Year wise cost of investment and income in percentage Cost: Income: Recurring Cost:	0 yr / 1st yr / 2 to 15th yr 100 / 0 / 0 0 / 50 / 100 0 / 50 / 100
	C) Financial Information	
1.	Interest rate on loan	11.5%
2.	Repayment period for Tank and wells Pump sets	 15 years 9 years
3.	Recovery of instalments – First year Second year	 Interest Capital-interest
4.	Discharge from Pump sets, 6 nos, @ 5 lps	30 lps
5.	Running cost of pump sets (Electricity)	Rs. 11.19/hr.
6.	Replacement of pump sets	10 years
7.	Residual value of pump sets at the end of 9th year	30%
8.	Establishment charges @ 1% of cost of percolation tank	Rs. 12,480
9.	O&M of percolation Tank @ 2% of cost of Percolation tank	Rs. 24,960
10.	Land revenue (Rs/ha) a) Pre A.R.Scheme b) Post A.R. Scheme	 Rs. 80 Rs. 133

D. Comparison of Cropping Pattern, Yield, Cost of Cultivation and Rate of Crops for Pre and Post Period of Construction of Percolation Pond.

Cropping Pattern	Irrigation in ha.		Depth of Irr. (m)	No. of Irr.	Yield Qtls /ha		Total Yield		Cost of cult./ha		Income Rs/qtl.	Gross Income		Gross cost of cultivation.		Net income (Rs)		Water Req.(cu m)	No. of pumping hours
	Pre	Post			Pre	Post	Pre	Post	Pre	Post		Pre	Post	Pre	Post	Pre	Post		
Hyb. cotton	2	2	0.90	10	8	19	16	38	4000	5002	750	12000	28500	8000	10004	4000	18496	18000	167
Hyb. Maize	1	2	0.45	6	20	30	20	60	1500	2500	300	6000	18000	1500	5000	4500	13000	9000	83
Jawar	5	4	0.22	5	10	27	50	108	1000	2330	205	10250	22140	5000	9320	5250	12820	8800	81
Groundnut	4	4	0.15	6	5	17	20	68	2500	4854	800	16000	54400	10000	19416	6000	34984	6000	56
Wheat	0	4	0.45	6	0	25	0	100	0	3644	350	0	35000	0	14576	0	20424	18000	167
Gram	0	6	0.20	4	0	15	0	90	0	3414	500	0	45000	0	20484	0	24516	1200	111
Total	12	22										44250	203040	24500	78800	19750	124240	718000	665

E. Calculation of Incremental Income and Recurring Costs.

Sl.No	Particulars	Pre		Post	
1.	Cost of cultivation	Rs.	24,500.00	Rs.	78,800.00
2.	Interest on 75% of cultivation cost @ 11.5%	Rs.	2,113.13	Rs.	6,796.00
3.	Land Revenue @ Rs 80/ha(pre) and @ Rs. 133 (post)	Rs.	960.00	Rs.	1,596.00
4.	Running Costs of Pump Sets @ Rs. 11.19/hr		Nil	Rs.	7,441.00
5.	Total Cost	Rs.	27,573.13	Rs.	94,633.00
6.	Gross Income	Rs.	44,250.00	Rs.	2,03,040.00
7.	Net Income	Rs.	16,676.87 (A)	Rs.	1,08,406.50 (B)
8.	Net Incremental Income (B-A)		-	Rs.	91,729.63
9.	Recurring Cost (Pre Cost-Post Cost) x 75% (25% of expenditure is expected to be incurred by the farmer)		-	Rs.	50,295.28

F) Cash Flow Statement

Particular	YEARS															
	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Initial Investment	457320	0	0	0	0	0	0	0	0	72600	0	0	0	0	0	
Recurring Cost	0	25148	50295	50295	50295	50295	50295	50295	50295	50295	50295	50295	50295	50295	50295	50295
Total Cost	457320	25148	50295	50295	50295	50295	50295	50295	50295	122895	50295	50295	50295	50295	50295	50295
Benefits	0	79395	158790	158790	158790	158790	158790	158790	158790	158790	158790	158790	158790	158790	158790	158790
Residual Value										21780						21780
Total Income	0	79395	158790	158790	158790	158790	158790	158790	180570	158790	158790	158790	158790	158790	158790	180570
Net Income	457320	54247	108495	108495	108495	108495	108495	108495	130275	35895	108495	108495	108495	108495	108495	130275

G) Summary of Economic Analysis

1. Discount Rate	:	15%
2. Net Present Cost (NPC)	:	649994
3. Net Present Income (NPI)	:	755072
4. Net Present Value (NPV)	:	105078
5. BCR (NPV/NPC)	:	1.16
6. IRR % when NPV=0	:	19.49

H) Repayment Schedule

Investments Details	Bank Loan (Rs)	Ist Year (Int. only) (Rs)	2nd to 9th year (Cap +Int)	10th to 15th year (Cap + Int)
Pump set (Elec.)	50,820	5,844	10,052	
Cost of A.R. Pro.	3,12,000	35,880	45,873	45,873
Cost of Wells	94,500	10,868	13,894	13,894
Total	**4,57,320**	**52,592**	**69,819**	**59,767**
% of Repayment to Net Incremental Income		57.30	76.10	65.20

I) Estimation of Economic Water Rate

1. Equated Instalment	:	Rs.	69,819
2. Annual Energy Cost for 665 hrs @ Rs. 11.19/hr	:	Rs.	7,441
3. Establishment Charges 1% of Tank Cost	:	Rs.	12,480
4. O&M of Tank 2% of Tank Cost	:	Rs.	24,960
5. Total Annual Cost	:	Rs.	1,14,700
6. Discharge (cu m/hr)	:		71,800
7. Water Rate/cu m	:	Rs.	1.60
8. Total Annual cost with 100% Subsidy on Estt. And O&M of Tank	:	Rs.	77,260
9. Water Rate/cu m	:	Rs.	1.08

10. OPERATION AND MAINTENANCE

Periodic maintenance of artificial recharge structures is essential because infiltration capacity reduces rapidly as a result of silting, chemical precipitation and accumulation of organic matter. In case of surface spreading structures, annual maintenance consists of scraping the infiltration surfaces to remove accumulated silt and organic matter. In the case of injection wells, periodic maintenance of the system consists of pumping and /or flushing with a mildly acidic solution to remove encrusting chemical precipitates and bacterial growths on the well screens. The intervals between periodic cleanings can be extended by converting injection wells into dual purpose wells. However, in the case of spreading structures constructed with an overflow or outlet mechanism, annual desilting is a must. Structural maintenance is normally carried out either by government agencies or through initiatives of stakeholders.

Success of artificial recharge schemes and related developmental activities primarily depend on the cooperation of the community and hence, should be managed at the local level. From a basin management perspective, the division of a basin into many micro-catchments is, hence, an essential recognition of the community role. The success of implementation and optimal utilisation of the schemes depend on participation and active contribution of the public.

Several issues are to be considered in the operation and maintenance of artificial recharge structures. These have been categorised as issues of high concern and moderate concern (ASCE, 2001). Safety, optimisation techniques and programs, value of wet-dry cycles, frequency of pond cleaning and condition of filters attached to the structures fall under issues of high concern, whereas security issues and rising ground water levels are among those of moderate concern in this regard.

10.1 Operational Data Requirements

Realistic estimates of the quantum of water entering and leaving the recharge area/basin/sub-basin are essential for assessing the volume of water that is recharged. Stream gauging stations in streams are needed if natural flows or a combination of natural flow and imported water are being recharged. In case the entire water being recharged is imported, suitable devices should be used to measure the inflow into the structure. The accounting of a system that has both surface and sub-surface recharge structures should also include devices to measure precipitation and evapotranspiration, which should be added to the inflow and outflow respectively. Initial measurements should be of sufficient frequency to determine how each of the parameters being measured varies with time. Once the variation is determined, a schedule that provides accuracy and economy can be set, which should integrate all the data being measured for optimizing data collection costs.

The data that should be measured for a recharge system include but are not limited to the following:

- Flow rate, duration and quality of source water.
- Inflow and outflow rates, duration and quality of inflow and outflow into and out of each unit of the recharge system.
- Recharge rates versus time for each unit and for the system as a whole.

- Depth to water and quality of ground water in the area being recharged and adjacent areas.
- Power usage by individual units and for the system as a whole.
- Depth to water in the recharge structures versus time (in case of surface structures)
- Thickness and composition of surface clogging layer when the structure is dry (in case of surface structures).
- Pressure versus time (in case of pressure injection)
- Depth to water in recharge well versus time in case of gravity head recharge wells.
- Precipitation and evaporation from surface ponds.
- Temperature of water at inflow and outflow locations.
- Time, rate and volume of pumping for each structure and for the system as a whole.

The data mentioned above helps fine-tune the recharge facility and provides the basis for corrections in case of problems. Periodic tests of pump efficiency, sampling of water quality and ground water level measurements should also be made and recorded on a defined schedule.

Measurement of any flows that pass downstream of the last recharge structure is needed if the total recharge from the operation is to be assessed. The volume of water passing the downstream gauging station, adjusted for precipitation and evaporation can be subtracted from the measured inflow volume to determine the quantum of water recharged.

10.1.1 Water Level Measurement

Measurement of ground water level in the aquifer, also known as 'static water level' or 'potentiometric head' is very important in artificial recharge schemes. Water levels have to be measured after a sufficient time has elapsed since stoppage of pumping or recharge to allow the water level to become stabilized and the drawdown/mounding effects to be minimized. Measurement of water levels in wells adjacent to a surface or subsurface recharge structure are also important as they help determine the shape and rate of growth of the recharge mound.

10.1.2 Water Quality Measurement

Complete water quality sampling and testing of a recharge scheme including source and aquifer should be done initially to determine the suitability of water for the intended use. The testing will provide a basis for the design of any other water quality treatment facilities that may be needed. After implementation of the scheme, periodic water quality assessment should be made. Proper training should be imparted to the personnel involved to ensure that the samples are not contaminated during collection and transportation.

10.2 Preventive Maintenance

Preventive maintenance of artificial recharge structures implies a periodic action taken to forestall major repair or replacement of its components. It may be drying up and scarifying of recharge ponds, periodic pumping of recharge wells, or regular application of lubricants / protective substances to the mechanical parts or replacement of minor parts that are subject to deterioration or repeated failure. It also involves regular observation and recording of the behaviour of both static and dynamic components of the system to detect changes in their inherent condition that indicates the need for unscheduled maintenance. These include reduction in the recharge rates, temperature of mechanical parts or rate of settlement.

10.2.1 Maintenance of Surface Recharge Structures

Artificial recharge structures such as percolation ponds and check dams are examples of 'wet/dry cycle' operation (ASTE, 2001) in which the structures get filled up one or more times during monsoon and remain dry during the summer season. These structures can be maintained by removing the silt deposited at the bottom of the structure periodically. The optimal amount of cleaning would remove the accumulation of surface material that has reduced the recharge capacity of the structure.

10.3 Potential Problems

The Problems normally encountered in recharge projects are mainly related to the source water available for recharge, which generally require some sort of treatment before use in recharge installations. They are also related to the changes in the soil structure and the biological phenomena, which take place when infiltration begins, to the changes of land ownership and legal aspects.

10.3.1 Suspended Material

A major requirement for waters that are to be used in recharge projects is that they should be silt-free. Silt may be defined as the content of un-dissolved solid matter, usually measured in mg/l, which settles in stagnant water having velocities not exceeding 0.1 m/hr. This definition comprises a large variety of materials such as clay particles, organic matter and fine particles of calcite. The silt content of river water depends upon the type of soils in the area of run-off, the vegetative cover of this area, its topographic slopes, meteorological characteristics prevailing in its catchment and intensity of rainfall.

Suspended matter may clog the soil in two different ways. Near the surface, the interstices of the soil may be filled up and a layer of mud may be deposited on the surface. On the other hand, they may penetrate deeper into the soil and accumulate there. A layer of mud is formed on the surface by particles, the settling velocity of which exceed infiltration velocities. Smaller suspended particles are filtered out in the uppermost layer of the soil. The filtration process is governed not only my mechanical factors, but it seems to be strongly influenced by electro-chemical surface forces. Still finer particles, especially very fine grains of montmorillonite clay, are carried further into the soil. Observations in spreading grounds composed of medium-grained dune sands, showed that these particles become lodged at depths ranging from 10 to 20 m below the surface, and some of these particles are carried even deeper. Semi-pervious layers situated deep below the sand filter out even those particles and become progressively clogged.

Methods to prevent or minimize the clogging effect by suspended matter can be classified into the following broad groups:

(a) Periodical removing of the mud cake and scraping of the surface layer
(b) Installation of a filter on the surface, the permeability of which is lower than that of the natural strata (the filter must be removed and replaced periodically)
(c) Addition of organic matter or chemicals to the uppermost layer
(d) Cultivation of certain plant-covers, notably certain kinds of grass

Scraping of the surface layer is effective only in coarse-grained soils. In soils composed mainly of sand, repeated compaction by heavy machinery may easily nullify any benefit gained from scraping. Various chemicals and organic matter have been used to restore infiltration capacities. These include gypsum, various organic compounds, cotton-gin trash and alfalfa (grown while the pond is still wet and then spaded under). The growth of a permanent grass-cover has proved to be an effective method for maintaining infiltration capacities, but it is difficult to select a grass which grows under a given climatic and soil condition and is able to withstand alternate periods of flooding and drying.

Clogging by biological activity depends upon the mineralogical and organic composition of the water and basin floor and upon the grain-size and permeability of the soil. The only feasible method of treatment developed so far consists in thoroughly drying the ground under the basin. Experiences seem to indicate that short periods of operation (about one month), followed by drying, are more effective than prolonged periods of operation, even if they are followed by a prolonged and most thorough period of drying during the hot summer.

Clogging and consequent destruction of bore holes may occur as a result of erosion of the aquifer. If velocities of flow are too high, fine sand and particles from local clay layers may be dragged outward into the aquifer and clog it or even cause collapse of the well. The common-sense precautions against these mishaps in semi-consolidated aquifers are to keep injection rates somewhat below the rate of proved safe continuous pumping and to avoid frequent sudden changes of the injection rate, which may cause vibrations. Experience has shown that no deterioration of the aquifer occurs if these reasonable precautions are taken.

Air bubbles, which are sucked into the well through the injection pipe, cause violent vibrations when they finally escape upwards. The possibility of air seepage must therefore be completely eliminated. The only certain way to achieve this is to design and operate the installation so that positive pressures (exceeding atmospheric pressure) are maintained everywhere in the injection pipe, even if this entails a reduction of injection rates.

Bore holes are much more prone to silting than spreading grounds. No acceptable standard of turbidity can be given. Clarity of the Water should conform to the standards of good drinking water. Clogging of the bore hole wall by bacterial growth may occur, even if water of potable standard is injected. Even when chlorination at the well-head carried out, the wells may still require periodic re-development by mechanical means and pumping.

10.3.2 Environmental Problems

A number of environmental problems may stem from artificial recharge schemes. Such projects usually have to be carried out in the vicinity of densely populated and industrial areas, where large quantities of water are needed. The close vicinity of spreading-grounds to population centers often creates various kinds of problems. In particular, it is well known that stagnant water serves as a breeding ground for mosquitoes, flies and a variety of other biological nuisances. The best remedy is to operate parts of spreading-grounds in sequence, so that water remains in each part for a shorter period than the larvae stage of the insect's life-cycle. This remedy may, however, lead to unrealistically large land requirements.

Other types of damages can also be generated by such projects. Artificial recharge is a procedure designed to raise ground water levels, which, under certain circumstances, can cause substantial damage such as inundation of basement of buildings. Damages may also be claimed if the recharged water is of interior quality to that previously enjoyed by nearby well-owners. Such might be the case where saline water, originating from treated sewerage effluent, is recharged into a freshwater aquifer. In such instances, more water is made available, say, for irrigation, but the practice may simultaneously create a deterioration of water used for drinking purposes.

There is no general solution for such problems. Each case has to be studied independently, taking into account the physical, economic, human and legal aspects. There is no doubt, however, that in most cases the overall benefits of such projects in water-short areas largely transcend the drawbacks.

10.3.3 Water Quality Problems

Various chemical processes such as adsorption, ion-exchange, oxidation and dissolution are expected to occur during the process of artificial recharge. Adsorption processes can occur at several levels from precipitated flocs to individual ion adsorption. Iron oxyhydroxide and organic flocs are particularly sticky substances and usually get adsorbed on aquifer particles and reduce the amount of water that can move through the aquifer. Acidification can be attempted to recover some permeability.

Ion-Exchange is significant if the aquifer water is brackish to saline. The clay particles typically have Sodium in exchange positions and these sodium ions are stable in high TDS conditions. Artificial recharge results in a reduction in TDS due to injection of lower TDS water and this increase reaction between dissolved ions in ground water. Calcium will replace the sodium in the exchange position, converting the clay to calcium rich clay. This exchange destabilizes the attached clay particles and allows them to move into the pore spaces of the aquifer, causing plugging.

Most aquifers are under either moderately oxidizing or reducing environments. Artificial recharge process with recharge water of more oxidizing nature than the ground water will destabilize the equilibrium of native ground water oxidation reduction conditions and cause chemical reactions to occur. Siderite (Iron Carbonate), Pyrite and Marcasite are the most susceptible to oxidation by recharge water. Depending upon how much of these minerals are present in the aquifer, the plugging of pores by iron oxyhydroxide flocs takes place. This plays a major role in areas where the pollutants get release and mobilized in an aquifer.

Solubility is a complex process that can involve several phases. Formation of calcite due to chemical reaction can be considered as an example. Carbon-dioxide participates in macrobiotic reactions and depending on PH, carbonic acid, bicarbonate and carbonate form in sequence with increasing pH. Finally, with sufficiently high calcium and carbonate concentrations, Calcite precipitates.

10.4 Physical, Biological and Chemical Compatibility of Water

Artificial recharge through injection wells can be effectively achieved if the recharge water is chemically and physically compatible with the native ground water. If the recharge water has similar chemical and physical characteristics, it will mix with the ground water without producing any undesirable effect in the aquifer media. It is impractical to determine whether the two waters are compatible by using only the few known factors and also it will be difficult to monitor the physicochemical environment in the aquifer system which is subjected to stress and strain through artificial recharge.

10.4.1 Physical Compatibility

i) Temperature

Two physical properties that relate to the compatibility of the recharge water and native ground water are viscosity and density. Viscosity and density are inversely proportional to temperature. If the injected water is colder than the native ground water, the injected water tends to settle towards the bottom owing to its greater density and viscosity. Cold water has greater viscosity and therefore moves through the interstices of an aquifer less freely than warm water. Thus, if the temperature of water in an aquifer is reduced, the effective permeability of the aquifer also gets reduced.

ii) Suspended Material

The presence of small amounts of suspended material in recharge water will seriously affect the performance of an injection well and the aquifer materials adjacent to the well. The degree of clogging depends not only upon the amount of sediment but also upon the composition of the sediment, the size of the particles, the composition of the aquifer materials and interstitial space in the aquifer. The effect of clogging in an injection well can be recognized by observing the injection rate and water level in the injection well. Clogging due to silt entry into the injection well causes a steady increase in the head.

In order to minimize the entry of silt in the recharge well, effective filtration of surface water through slow-sand filter is generally followed. Depending upon the turbidity level of surface water, two-stage filtration can be adopted if required with a rapid filtration stage followed with slow filtration. It is necessary to monitor the turbidity of recharge water periodically. During the recharge process, if the records of water level and injection rate indicate clogging, it is necessary to surge (back flush) the injection well with heavy discharge pumps for a short duration. Surging helps in the removal of silt settled in the well section and in the aquifer zones adjacent to the well. If the recharge process is continued without surging, the suspended particles may enter into the interior part of the aquifer and it would be difficult to remove the particles settled in the interstices. It is advisable to do the surging periodically, irrespective of any indications of clogging.

iii) Air

Air introduced into a well during recharge may affect the permeability of the aquifer through both physical and chemical processes. The air bubbles occupy the space in the interstices, thereby reducing the effective porosity and block the movement of water. The bubbles are normally tightly held on to the aquifer materials by molecular attraction and get diffused very slowly. In fresh water used for injection, the ratio of dissolved oxygen, nitrogen and other atmospheric gases is virtually constant at normal water temperature, unless the oxygen content is reduced or increased by biological activity. Clogging by air bubbles is easily recognized by a sharp increase in injection well water level immediately after recharge operations start. The air clogging is easily seen by formation of air foaming when the injection operation is stopped. Physical entrapment of air in the recharge well can be minimized by carrying the injection pipe some distance below the static water level in the well. Surging operations may help in removing some air clogging near the well screen.

10.4.2 Biological Compatibility

Biological suitability of the recharge water is also an important factor controlling effective artificial recharge. Pathogenic bacteria can render a ground water unfit as a source of drinking water. Other harmless bacteria species may lead to coloring of ground water and cause unpleasant taste and odour. Bacteria in a suspended matter multiply rapidly when their food supply is abundant. When they enter the recharge well along with recharge water and multiply in the well screen openings or in the gravel pack, resulting in clogging or reduction in intake rate. In the case of bacteriological clogging, the rise in injection head reaches its maximum value after some days only. When biodegradable matter is present in the injection water, a complete sealing of the well may occur within one or two weeks. Pre-treatment through slow sand filtration of injected water and chlorination may prevent the growth of bacteria in an injection well. Maintaining a residual chlorine level of 1-2 mg/l in the injection well is recommended for minimising the biological clogging.

10.4.3 Chemical Compatibility

Bore hole injection operations encounter difficulties when the recharge water reacts with the native ground water or with the aquifer material. The reaction may lead to formation of insoluble deposits in the pore spaces, hindering the ground water movement. Three reaction stages over time and space may be distinguished:

i) At the very beginning of the recharge process, the native ground water is displaced by the recharge water. During the initial phase of recharge experiment, a mixed zone (a zone containing recharge and native ground water) is expected to form and in this zone, the reactions may take place. This is particularly disadvantageous with well injection, where in the immediate vicinity of the well a small reduction in pore space appreciably increases flow resistance. Formation of mixed zone cannot be prevented, but it is possible to prevent the formation of a mixed zone in the near vicinity of the well by injecting an amount of non-reactive water, which effects deposition at a distance sufficiently away from the well so that the intake capacity of the well is not affected.

ii) With passage of time, all the native ground water in the aquifer is replaced by recharge water. Reactions are now only possible between recharged water and aquifer matrix. Mostly, the reactions may result in an increase of the mineral content of recharged water.

iii) During the recovery of recharged water from the wells, the abstracted water from the recharge well will, in the initial stages, be very close to injected water in quality. With passage of time, extraction will be a mixture of recharge water and native ground water. As a result of incompatibility of recharge water and native ground water, blocking of the formation will again take place around the wells used for abstraction.

10.5 Maintenance of Roof Top Rainwater Harvesting System

Maintenance of roof top rainwater harvesting system (RRHS) is simple and costs little. As the entire system is household-based, it becomes one of the assets of the household and hence could be maintained best by the users themselves. It requires continuous care and maintenance just as any other asset in the household. In fact, maintenance of RRHS should get priority over other household assets, as it ensures the good health of all people in the household. Cleanliness of surroundings as well as the system including its various components such as roof, gutters, filtration unit and the storage tank, will ensure supply of water of potable quality throughout the water scarcity period for the drinking and cooking purposes of the household.

10.5.1 Tips for Maintenance of the RRHS

- Always keep the surroundings of the tank clean and hygienic
- Remove algae from the roof tiles and asbestos sheets before the monsoon
- Drain the tank completely and clean the inside of the tank thoroughly before the monsoon
- Clean the water channels (gutters) often during rainy season and definitely before the first monsoon rain
- Avoid first 15 or 20 minutes of rainfall depending on the intensity of rain. Use the first flush arrangement to drain off this first rainwater.
- Change the filter media every rainy season
- Cover all inlet and outlet pipes with closely knit nylon net or fine cloth or cap during non-rainy season to avoid entry of insects, worms and mosquitoes
- Withdraw water from the system at the rate of 5 litres/head/day. This will ensure availability of water throughout the water scarcity period.

- Leakage or cracks in the storage tank should be immediately attended to. This will obviate the need for major repairs caused by propagation of cracks.
- Heavy loads should not be applied on the lid.
- Water should not be allowed to stagnate in the collection pit
- The tap should have lock system to prevent pilferage or wastage of water
- The filter material should be washed thoroughly before replacing in the filter bucket
- In coastal areas, the outer side of the tank may be painted with corrosion-resistant paint at least once in 3 years and in other areas lime (Calcium Carbonate) based whitewash may be applied regularly.

People may be educated by providing the above tips for maintenance of the system through pictures, handouts and wall posters. The implementing agency should visit the structures as follow-up to monitor and motivate the users in proper maintenance of the systems. There could be informal group discussions among the users on the maintenance aspects of the Roof Top Rainwater Harvesting Systems.

As a precautionary and preventive measure, the water from the storage tank may also be tested for the presence of disease causing micro organisms. This task may be taken up by the implementing agency as an immediate follow up of the construction of the systems. This helps the agency to find out the users attention to the maintenance of the system as well as necessary awareness to be given on various maintenance aspects.

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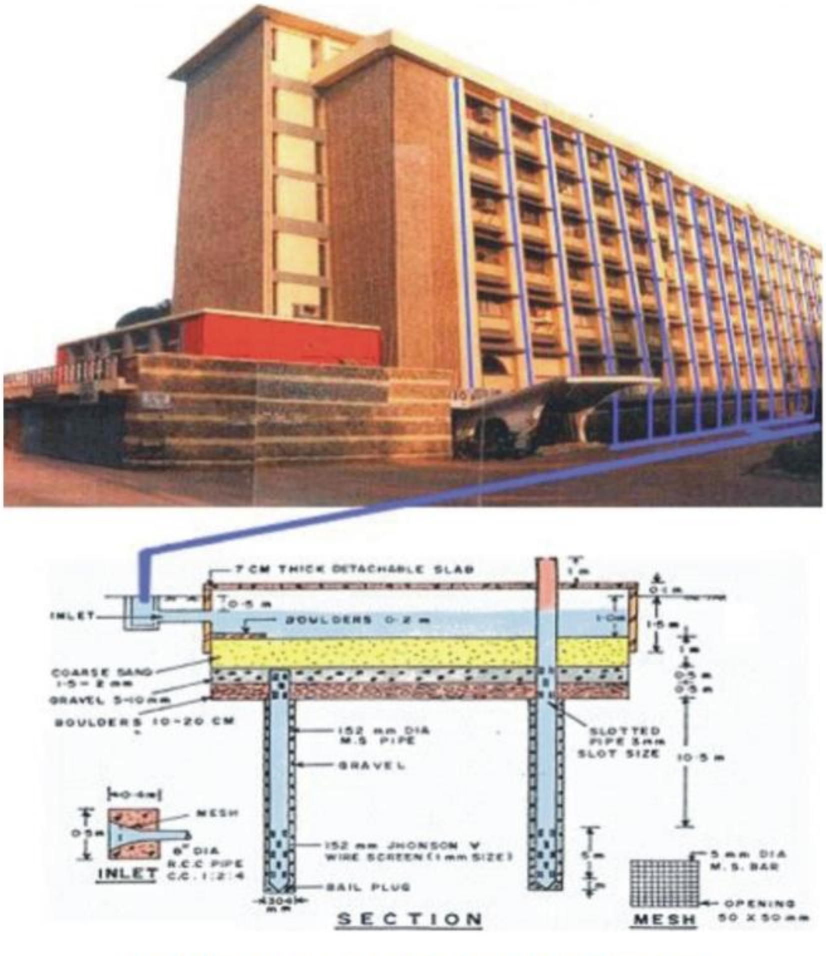

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