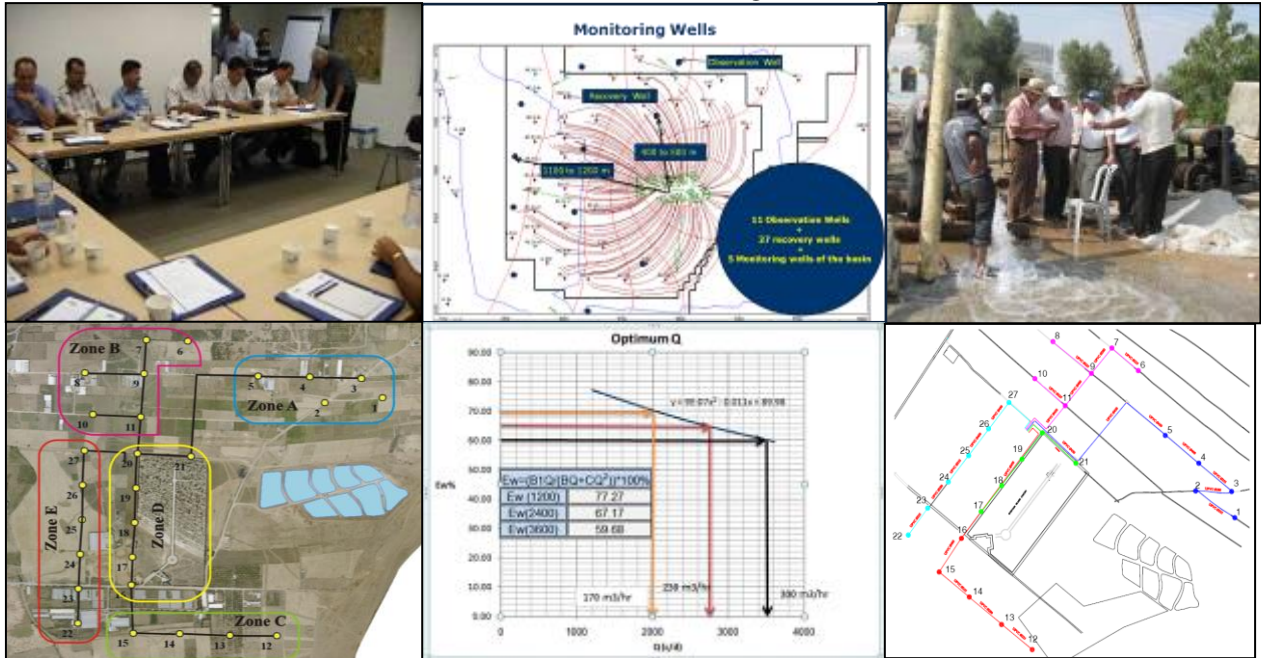


Consulting Services for Detailed Design and Tender Documents of Effluent Recovery and Irrigation Scheme of North Gaza Emergency Sewage Treatment (NGEST)

Contract Number: NGEST/AF-QCBS01-08/DD



Design Report

Consultant

**Joint Venture Association of the Center for Engineering and
Planning (CEP) and the FCG International Ltd.**

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LIST OF ABBREVIATION

Avg.	Average
BLWWTP	Beit Lahia Wastewater Treatment Plant
BH	Borehole
BOD	Biological Oxygen Demand
CCQC	<i>Consulting Center for Quality and Calibration</i>
Cl	Chloride
CM	Cubic Meter
cm	Centimeter
CMWU	Coastal Municipalities Water Utility
CAMP	Coastal aquifer Management Program
COD	Chemical Oxygen Demand
DTM	Digital Terrain Model
EC	Electrical Conductivity
EDR	Electro Dialysis Reversal
FCV	Flow Control Valve
EA	Environmental Assessment
GV	Gate Valve
GWL	Ground Water Level
hr	Hour
HGL	Hydraulic Grade Line
in	Inch
Kg	Kilograms
Km	Kilometers
l	liter
LL	Liquid Limit
L/c/d	Liter/Capita/Day
m	Meters
MBC	<i>MASHHARAWI BROS.</i>
mcm	Million cubic meters
MOA	Ministry of Agriculture
MOH	Ministry of Health
MOLG	Ministry of Local Governorates
MSL	Mean Sea Level
MT3D	Modeling Transport Three Dimensional
MW	Monitoring Well
MDB	Main Distribution Board
MODFLOW	Model Flow
NGEST	North Gaza Emergency Sewage Treatment
NGWWTP	Northern Gaza Wastewater Treatment Plant
PA	Palestinian Authority
PALNET	Palestinian Grid System
PH	Acidity or Alkalinity
PL	Plastic Limit
PMU	Project Management Unit
ppm	Parts per million

PRV	Pressure Reducing Valve
PS	Pumping Station
PWA	Palestinian Water Authority
RO	Reverse Osmosis
SAR	Sodium Adsorption Ratio
SIR	Surface Infiltration Rate
SP	Soil Profile
SPT	Standard Penetration Test
TDS	Total Dissolved Solids
Ton	Metric Ton
TP	Test Pit
TPS	Terminal Pumping Station
TOR	Terms Of Reference
TS	Total Solids
TSS	Total Suspended Solids
TKN	Total Keldan Nitrogen
WH	Water Holding
WHO	World Health Organisation
WTP	Water Treatment Plant

EXECUTIVE SUMMARY

Overview

The draft design report prepared by FCG and CEP joint venture consultant includes the design criteria, inputs and outputs for the recovery (recovery wells, collection pipes, observation wells and associated facilities) and reuse (water tanks, booster pumping station, irrigation water network and associated facilities) schemes. It also gives a general description of project background, objectives and future extensions for both the recovery and reuse schemes. Full field investigations and surveys and reports have been included as appendices to this report and a summary of these have been included in the main body of the report. Special emphasis has been placed on groundwater modeling to verify and update previous model and also to use the model in predicting future conditions of groundwater. The model has also been used for planning and design of the recovery wells. Taking into consideration the environmental sensitivity of the project, a comprehensive monitoring program has been developed to observe groundwater quality. The design criteria and system design have covered all physical components of the project. Supporting data, design calculations, and drawings have been included as appendices to this report. Cost estimates have covered the investment, operation and maintenance parts of the project. The investment cost was found to be around **28,304,478 USD** and the operation and maintenance cost per year is about 10% of the capital cost. Two implantation stages are proposed for carrying out the project. Each stage includes two tender packages. The first stage- first package (Supply and Stall) will include 15 recovery wells and concerned connection pipes, the civil works within the booster pumping station, five booster pumps, one 4000 m³ water tank and 5 monitoring wells. The first stage-second package (Small Works) includes irrigation network for 5000 donums. The cost for the first stage is around **11,969,344 USD**. The remaining works are to be implemented during the second stage. The second stage- first package (Supply and Stall) will include 12 recovery wells and concerned connection pipes, the remaining civil works within the booster pumping station, five booster pumps, one 4000 m³ water tank and 5 monitoring wells. The second stage-second package (Small Works) includes irrigation network for 10,000 donums. The cost for the second stage is around **16,335,133 USD**.

The following is a summary of main inputs and results from the design report.

Objectives and Scope of Work

The effluent recovery and irrigation scheme (current project) is a part of North Gaza Emergency Sewage Treatment (NGEST) project (overall project) which includes municipalities of Jabalia, Beit Lahya, Beit Hanoun and Um Al Nasser. The NGEST project consists of two parts; Part A and Part B. Part A which has been completed includes the Terminal Pumping Station (TPS) located at Beit Lahya Wastewater Treatment Plant (BLWWTP), pressure main from TPS to the location of the Northern Gaza Wastewater Treatment Plant (NGWWTP) and infiltration basins located at NGWWTP. Part B of the project which is the NGWWTP is under construction. The current project comes as an integral part of the NGEST project to provide a detail design and tender documents for implementation of risk management facilities to:

1. Avoid a potential long term irreversible impact to the groundwater in the surrounding areas.
2. Implement mitigation measures against environmental, social and public health impacts to nearby communities.

The risk management facilities for effluent recovery comprise of recovery wells, collection pipes, observation wells and associated facilities. The reuse facilities comprise of water storage tanks, booster pumping station, irrigation water network and associated facilities. The recovery and reuse scheme has been designed for the first phase of 35,600 m³/day capacity of the NGWWTP to be reached in the 2015 design year. This scheme will be extended to 69,000 m³/day effluent of the 2025 design year. The scope of current assignment has been to design and supervise the construction of the risk management facilities of the first phase capacity. In addition, the current assignment has taken into consideration the requirements of the future extension. Future extension requirements include additional infiltration basins, agricultural land, recovery wells, water tanks, booster pumping station, irrigation networks, etc.

Investigations, Studies, Criteria and Design Parameters

The following comprehensive field investigations, surveys and studies have been carried out to enable the design of the physical components of the project:

- 1. Geotechnical investigations:** Physical and chemical tests for agricultural use to estimate the water demand and types of crops. The test results showed that the soil is *loamy soil* which is suitable for agricultural purposes for a wide range of crops. In addition soil tests were carried out to determine the mechanical properties for the design and construction of the piping systems and structures.
- 2. Hydrogeological and water quality investigations:** New investigations were used to update the groundwater model and the assessment of the groundwater quality status. All information collected from SWECO investigation and the current investigations have been used in groundwater modeling and the design of the recovery wells. Five 85m pumping tests were made in addition to hydraulic permeability tests up to 10 m depth and soil classification above the water table were also made. Laboratory chemical tests on water samples collected during the investigation at the end of pumping were conducted.
- 3. Topographical survey:** Topographical survey covered the piping network routes, booster pumping station, water tanks, service buildings, wells and other associated facilities. In general, the topography of the project area is a flat sloping where the level varied from 87m to 40m at the northern-east and northern west sides of the agricultural land, respectively. The site layout topography for the booster pumping station and associated facilities is almost flat with less than 2 m difference. The maximum difference in levels between the booster pumping station and the irrigation net works is about 50 m. While the maximum difference in the levels between the recovery wells and water tanks is about 18 m.
- 4. Hydrogeological Assessment and Modeling:** Groundwater modeling was used to verify and update previous model, predicting future conditions of groundwater and for the planning and design of the recovery wells. A comprehensive monitoring program has been developed to observe groundwater quality. The observation program has extended to monitor both the recovery and reuse schemes up to the end user.
- 5. Agricultural study:** A comprehensive study was carried out for the determination of the irrigation plan in the project area. The study has taken into consideration main influencing factors and requirements such as crop patterns, water quality, agricultural zones, irrigation scheduling and demands, soil characteristics, environmental factors, weather, climate change, leaching requirements, losses, etc. According to the study the total agricultural land in the project area is about 15,000 dunoms. The agricultural land was subdivided into six zones (zones A, B, C, D, E and F) of almost equal size averaging 2500 dunoms each. Each zone is

to be irrigated once each 6 days. The peak demand was found to be 50,885 m³/day is in the month of June and the lowest demand of 30,187 m³/day is in the month of October. The following table shows the monthly variation in the demand for the 2015 design year.

Month	Water demand (m ³ /d)
Jan.	33081
Feb.	35816
Mar.	34995
Apr.	34204
May	46622
June	50885
July	50136
Aug.	49073
Sept.	40290
Oct.	30187
Nov.	31484
Dec.	33146
Average	39160

6. Water demand for irrigation: A comprehensive study and field survey were carried out to determine the variations in irrigation demands across the year and during the day. The obtained results have influenced the design of the physical components of the reuse scheme that includes the water tanks, booster pumping station, and irrigation networks. The following table includes the maximum and minimum water demand and storage requirements for the six irrigation zones.

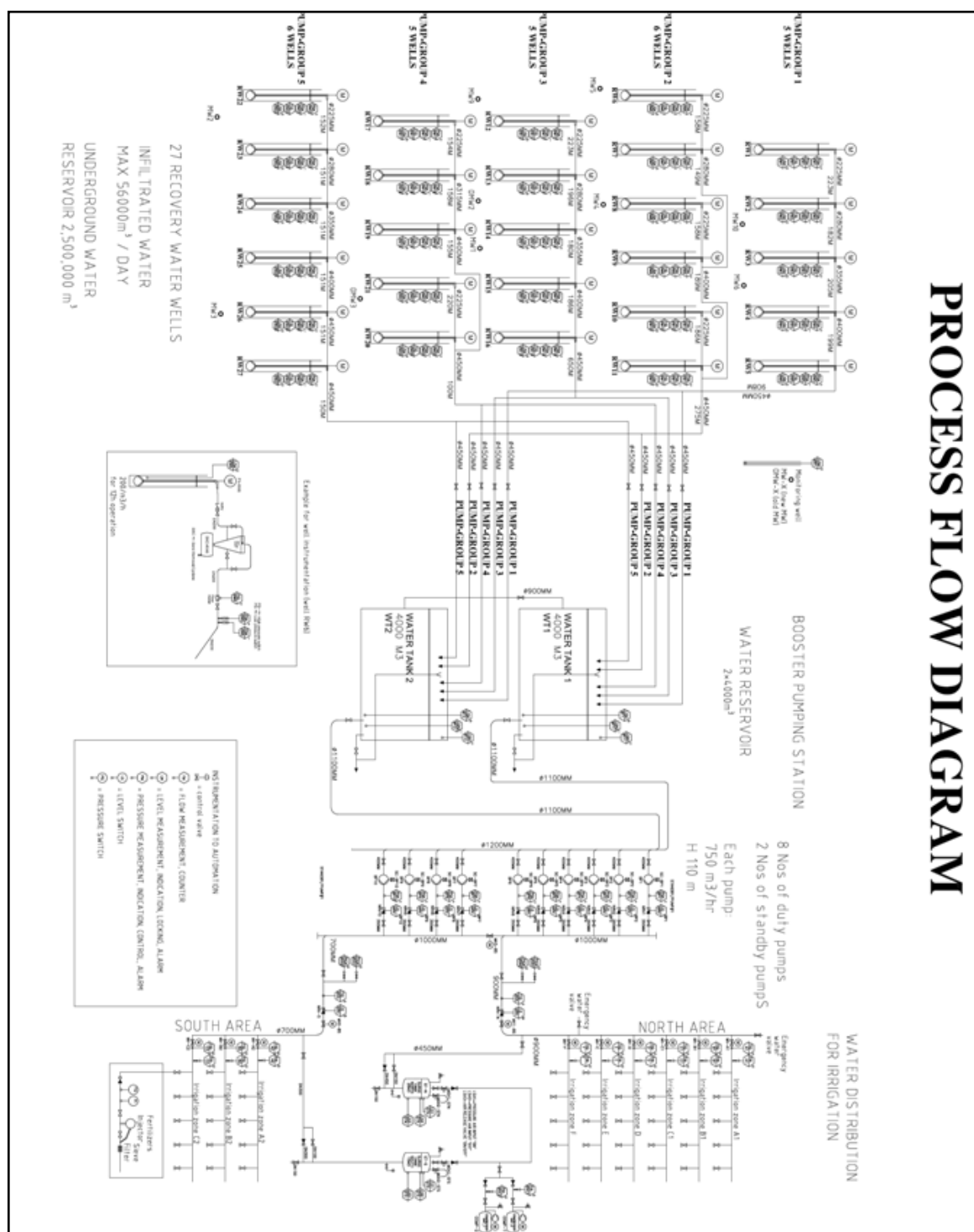
Irrigation Zones	Peak June Month					Lowest October Month				
	Working hours	Constant Supply (m ³ /hr.)	Max. Demand (m ³ /hr.)	Min. Demand (m ³ /hr.)	Storage (m ³)	Working hours	Constant Supply (m ³ /hr.)	Max. Demand (m ³ /hr.)	Min. Demand (m ³ /hr.)	Storage (m ³)
Zone A	12	4240.4	4544	3580.2	1789.8	8	3773.4	3920.6	3391.9	672.9
Zone B			4922.3	2846.8	4142.9			4115.3	2764.6	1660.8
Zone C			4731.6	3389.2	2848.7			4026.7	3086.5	1202.6
Zone D			4921.9	2870	3987.8			4107	2842.3	1550.9
Zone E			5149	2256.5	5688.7			4215.4	2355.1	2189.1
Zone F			5574	2110.6	7527.6			4461	2162.5	3223

Irrigation Zone F was found to have critical design requirements for the considered peak summer day. Maximum and minimum hourly pumping rates are 6000 m³/hr. 2100 m³/hr., respectively. Two water storage tanks of 4000 m³ each are required.

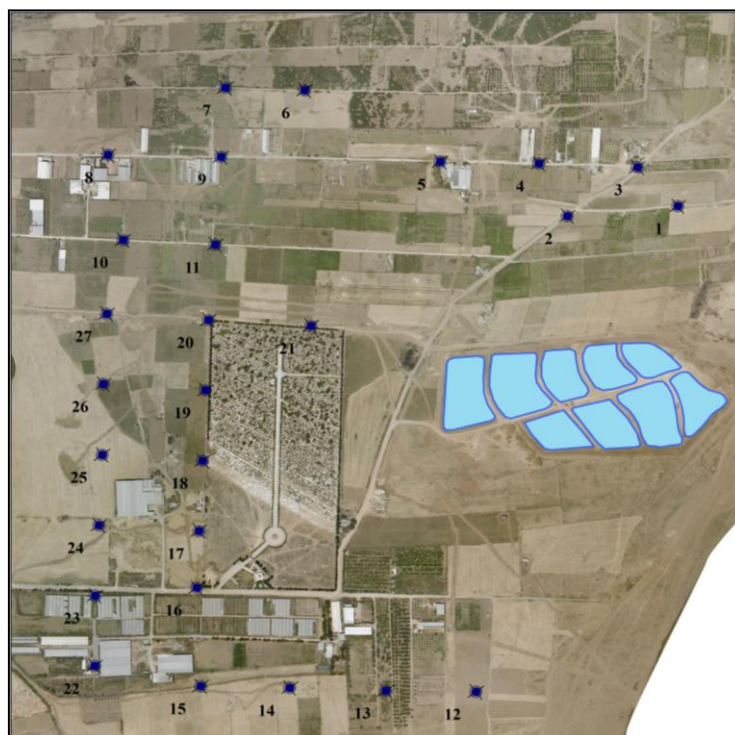
Design Outputs

The input and results concerning hydraulic, mechanical, structural, electrical designs for wells, storage tanks, booster pumps, piping network and associated facilities have been included as appendices to this report. The following is a summary of main design results.

1. **Flow process diagram:** The following figure shows all project components and their interconnections.



2. **Recovery wells:** The total number of wells is 27 of a pumping capacity of 150 m³/hr to 200 m³/hr. The number of operation wells is 25 wells with a capacity of 170 m³/hr. Two wells are allocated to provide flexibility in operation and to compensate any shortage in water supply in case of emergency if for example some wells are failed. The wells were carefully allocated around the infiltration basin with a distance of 550 m to 750 m from the basin. The minimum distance allows of a retention time equal to 1000 days which ensures the operation of the sand aquifer treatment process. The wells are concentrated in the water flow direction which allows capture the plume and prevent exceeding the 750 m distance from the basin. The following figure shows the locations of the recovery wells.

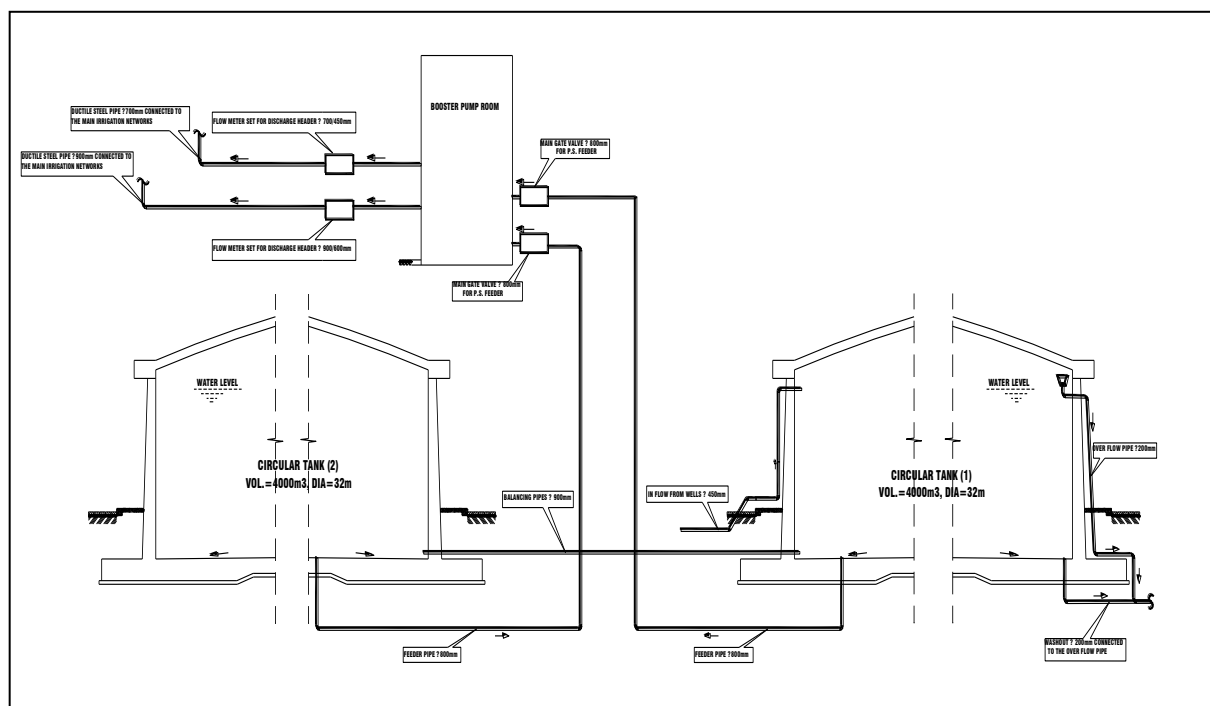


The recovery wells have 20 inch external diameter of borehole, the diameter of screen is 12 inch with an opening size ranges between 0.6 mm to 0.8 mm and the opening slot percentage is 30%. The length of screen is 13 m located in sand or coarse sand layer below the water table. Stainless steel screens are used. The gravel pack size is ranging from 2 mm to 4 mm. The distance between the recovery wells is estimated based on the water table drawdown records from observation wells during the pumping tests. The distance between the wells is not less than 140 m. The pump is a vertical turbine pump installed in the bottom of the well.

3. **Monitoring wells:** Adequate number of observation wells is proposed to give accurate data about groundwater status. Ten new observation wells are used for monitoring groundwater quality. In addition, 27 recovery wells and 5 existing monitoring wells will be also used for monitoring purposes. The total number of monitoring wells will be 42. The water pumped to the irrigation network is monitored through samples of water from random farms taken to check the quality at the end user. Trunk lines, water tanks, and irrigation networks are also monitored by taking random samples from each component.
4. **Piping networks:** The design of collection and irrigation networks was based on the adopted hydraulic model. Several diameters of ductile iron and UPVC pipes are used in both networks depending on the size of the pipe. The irrigation network diameters ranged from 900 mm

(1200 mm inside the booster station) to 50 mm. The velocities ranged from 2.85 m/sec. to 0.65 m/sec.

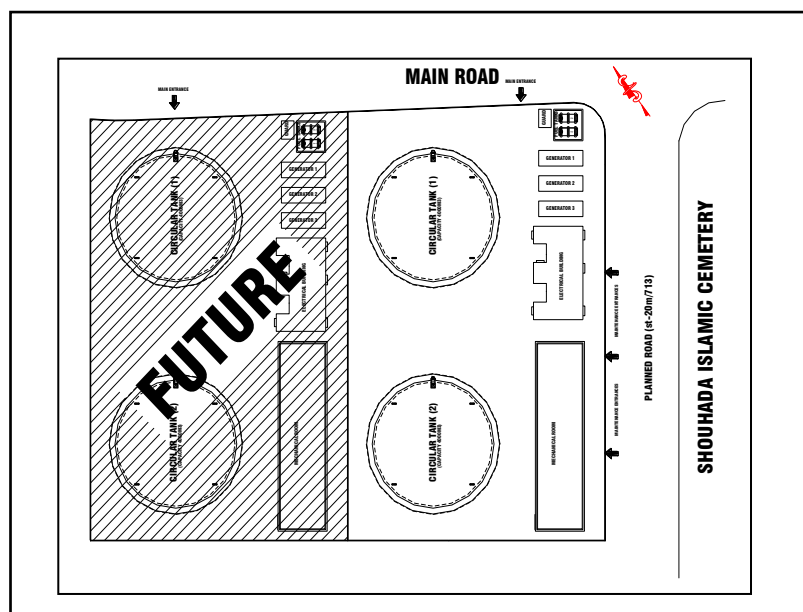
5. **Water tanks:** Two 4000 m³ water tanks of 32 m diameter and 5 m height are used. The thicknesses of the water tank walls and foundation ranged from 400 mm to 600 mm. The structural design results indicated satisfaction for both ultimate and serviceability limit states. The collection pipes from the recovery wells are connected to the tanks. There are two inlet pipelines from two well groups with a diameter of 450 mm to one tank and three inlet pipes with diameter equal to 450 mm from three well groups to the other tank 2, as shown in the following figure. The two tanks are connected to each other to provide flexibility and are provided with washout and overflow pipes. The feeder from each tank to the booster pump stations is 1100 mm diameter with a main gate valve.



6. **Booster Pumping Station:** The booster pumps are located in a pumping hall together with the suction and pressure manifolds and with all necessary pipe works. The pumping station will serve both irrigation networks; the south area with three irrigation zones and north area with six irrigation zones. There are all together 8 of duty pumps and 2 of stand-by units, all similar pumps, installed parallel and pumping from a common suction manifold into a common pressure manifold.

The pump size is selected based on the maximum system flow rate 6000 m³/hr with the total dynamic head (TDH) 101 m wc. The number of duty pumps for each pumping mode is selected based on the consultant analyses with pumping model software, and showing the pump discharge pressure for irrigation zones with different flows.

The following figure shows the booster pump station including future extension.



Costs

The investment cost was found to be about **28,304,478 USD** and the operation and maintenance cost per year is about 10% of the capital cost. Two implantation stages are proposed for carrying out the project. The cost for the first stage is around **11,969,344 USD**. The following table shows the capital cost of the main items.

Item No.	Description	Total Rate (USD)
1	General Items	262,400
2	Circular Tank 4000 M3 (2 Tanks)	1,012,010
3	Booster Site (Civil)	281,022
4	Mechanical Building (Mech)	2,285,150
5	Electrical Building	225,690
6	Guard Room	10,622
7	Recovery Wells (27 Well)	2,833,917
9	Monitoring Wells (5 Wells)	222,600
10	Well Networks (around 6.7 Km)	674,190
11	Instrumentation & Automation Scada System	1,961,250
12	Electrical Works	2,885,897
13	Irrigation Network (around 128 Km)	15,649,730
Grand Total		28,304,478

1 BACKGROUND

1.1 Project Area

The work under current assignment is a Consultancy Services for Detail Design, Tender Documents and Construction Supervision of the Risk Management Facilities Components "Effluent Recovery and Irrigation Scheme" of North Gaza Emergency Sewage Treatment (NGEST). The services are part of North Gaza Emergency Sewage Treatment Plant Project-Additional Financing for Implementing Risk Management Facilities.

The North Gaza Emergency Sewage Treatment (NGEST) project being implemented by the Palestinian Water Authority (PWA) covers about 55 km² and includes municipalities of Jabalia, Beit Lahya, Beit Hanoun and Um Al Nasser. The population of the project area is around 415,000 by 2015. The project consists of two parts; Part A and Part B. Part A which has been completed includes the Terminal Pumping Station (TPS) located at Beit Lahya Wastewater Treatment Plant (BLWWTP), pressure main from TPS to the location of the Northern Gaza Wastewater Treatment Plant (NGWWTP) and infiltration basins located at NGWWTP. Part B of the project which is the NGWWTP is under construction. After the completion of the NGWWTP the two parts will be the main integral parts of the whole system of wastewater treatment, infiltration and reuse. The main components of the NGEST project are shown in Fig. 1.1.

1.2 Objectives of the NGEST Project

The main objectives of the NGEST project are:

1. To mitigate the immediate and gathering health and environmental safety threats to the communities surrounding the sewage lake at the existing BLWWTP.
2. To provide a satisfactory long-term solution to the treatment of wastewater for the Northern Governorate in Gaza.

The infrastructural facilities in Part A of the project were urgently implemented to addresses the immediate health and environmental threats posed by the sewage lake at Beit Lahia. Despite that the construction of the NGWWTP has not been completed yet, nine infiltration basins have been constructed and operated. Draining of the lake has already alleviated the threats of potential failure of its embankments and the flooding of adjacent communities. Until the NGWWTP is put into operation, low-quality effluent from the BLWWTP is being pumped directly into the nine infiltration basins.

Part B of the project addresses the medium to long term needs of northern Gaza Strip for adequate wastewater treatment. The construction of the NGWWTP will ultimately solve existing problems associated with BLWWTP. The treated effluent from the NGWWTP will be infiltrated into groundwater and then recovered to be used for irrigation of surrounding agricultural land of 8 km average length (north south) and 2 km average width (east west).

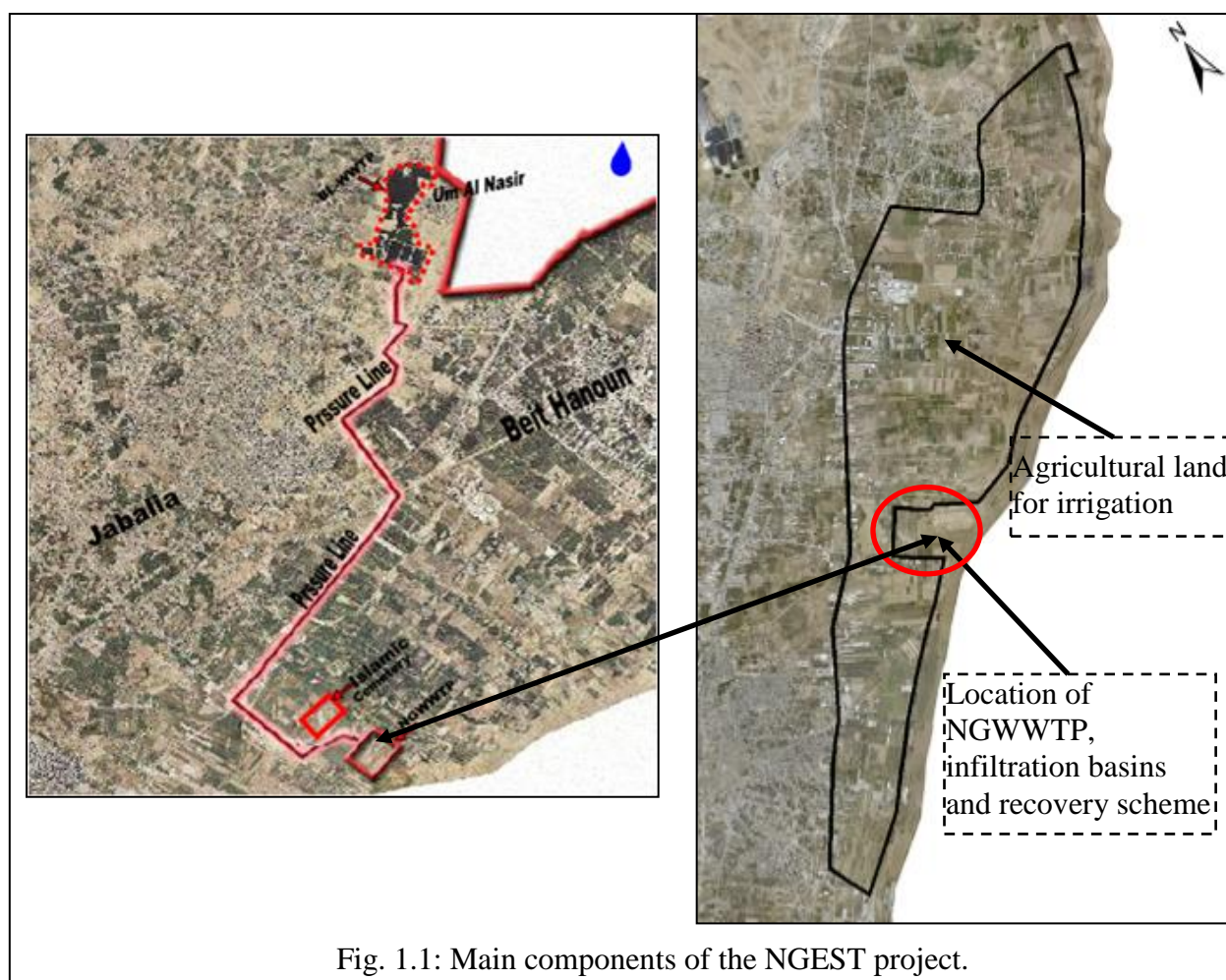


Fig. 1.1: Main components of the NGEST project.

1.3 Objectives of Current Assignment (Recovery and Reuse Scheme)

The NGEST project necessitates the implementation of risk management facilities which is the aim of the current assignment *“Consultancy Services for Detail Design, Tender Documents and Construction Supervision of Effluent Recovery & Irrigation Scheme”*. This is to:

3. Avoid a potential long term irreversible impact to the groundwater in the surrounding areas.
4. Implement mitigation measures against environmental, social and public health impacts to nearby communities.

It should be mentioned that the reuse of the recovered water in irrigation will assist in reducing the water scarcity problem in Gaza Strip.

The current assignment consists of two stages, i.e. the design and the construction supervision stages. Thus the objectives of the assignment are:

1. To prepare the detailed design for the Risk Management facilities.
2. To prepare complete set of bidding documents for the construction of contractual packages.

3. To provide construction supervision services for the Risk Management components.

1.4 Future Considerations

The under consideration recovery and reuse scheme has been designed for the first phase of 35,600 m³/day capacity of the NGWWTP to be reached in the 2015 design year. This scheme will be an integral part of the regional irrigation scheme to accommodate the 69,000 m³/day effluent of the 2025 design year. The scope of current assignment has been to design and supervise the construction of the risk management facilities of the first phase capacity. In addition, the current assignment has taken into consideration the requirements of the future extension for the 2025 design year. Future extension requirements include additional infiltration basins, agricultural land, recovery wells, water tanks, booster pumping station, irrigation networks, etc.

1.5 Physical Components of the Recovery and Reuse Scheme

1.5.1 Physical Components of the Recovery Scheme

The physical components of the recovery part of the scheme for the 35,600 m³/day capacity include:

1. **Recovery wells:** One of the most challenging tasks in this project is to determine the number and the locations of the groundwater recovery wells that will be able to capture the infiltrated water in the appropriate time and quantity.
2. **Collection pipes:** Collection pipes are used to collect and transmit the recovered water from the recovery wells to water tanks.
3. **Monitoring wells:** Monitoring wells are used to observe the groundwater table and the groundwater quality status.

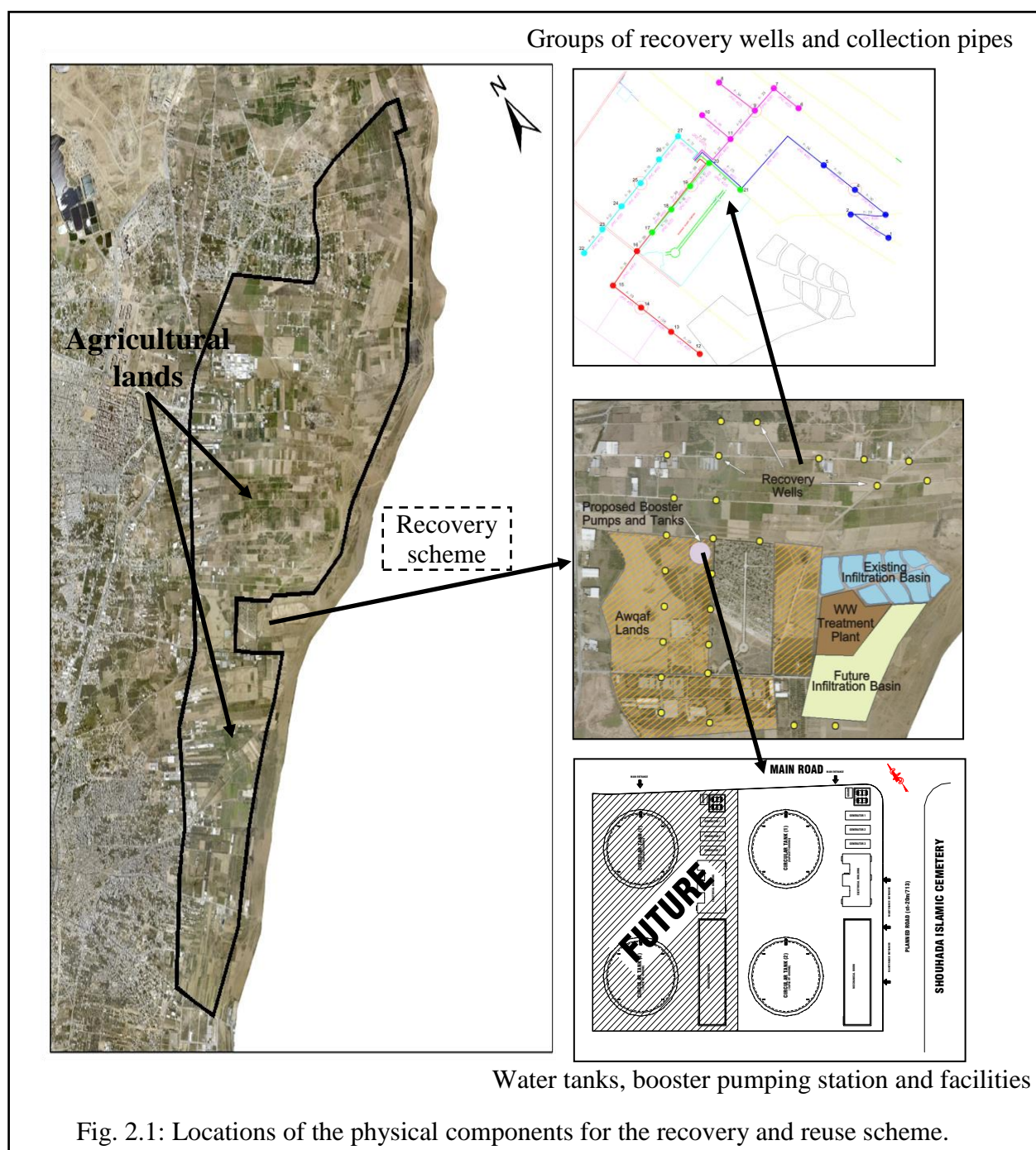
1.5.2 Physical Components of the Reuse (Irrigation) Scheme

The physical components of the reuse part of the scheme for the 35,600 m³/day capacity include:

1. **Water tanks:** The recovered water from the wells is collected into two water tanks of about 4000 m³ each that are in turn connected to a booster pumping station.
2. **Booster pumping station and associated facilities:** A booster pumping station is used to transmit the water from the tanks to the farms. The booster pumps will maintain a minimum pressure of 2.5 bars in the irrigation network at farm gates.
3. **Irrigation distribution network:** Water supply pipelines (trunk lines) are used for transmitting the water from the booster pumping station to the agricultural land. Water networks are used for irrigation the agricultural lands.

2 SELECTED SITE AND LOCATIONS

The recovery and irrigation scheme are located in the Eastern part of the Northern Governorate in Gaza Strip. The irrigation network will also serve agricultural land located in the north eastern part of Gaza Governorate. Fig. 2.1 shows the locations of the physical components of the recovery and irrigation scheme. Description and discussion of the identified locations and sites are as follows:



2.1 Locations and Sites for the Recovery Scheme

2.1.1 Locations of Recovery Wells

The recovery wells have been distributed around the infiltration basins in the north, west, and south directions as shown in Fig. 2.1 and Fig. 2.2. The wells are distributed in two rows in accordance with groundwater modeling outputs and existing hydrogeological conditions. The distribution of wells also suits future extension of the recovery scheme for the 2025 design year. The wells have been located at road sides to facilitate easy access and land acquisition. The exact locations of wells are discussed in the design chapter and shown in the relevant design drawings.

2.1.2 Location of Collection Piping System

The recovery wells have been connected to the water tanks using five collection pipe networks shown in Fig. 2.1. The majority of the collection pipe networks are located in existing roads and the remaining networks are located in new proposed roads. The exact locations of collection pipe networks are discussed in the design chapter and shown in the relevant design drawings.

2.1.3 Location of Monitoring Wells

Two rows of monitoring wells are located before and after the recovery well rows. Two additional monitoring wells are also located to the eastern of the infiltration basins as shown in Fig. 2.2. The monitoring wells have also been located at road sides to facilitate easy access and land acquisition, if necessary. The exact locations of the monitoring wells are discussed in the design chapter and shown in the relevant design drawings.

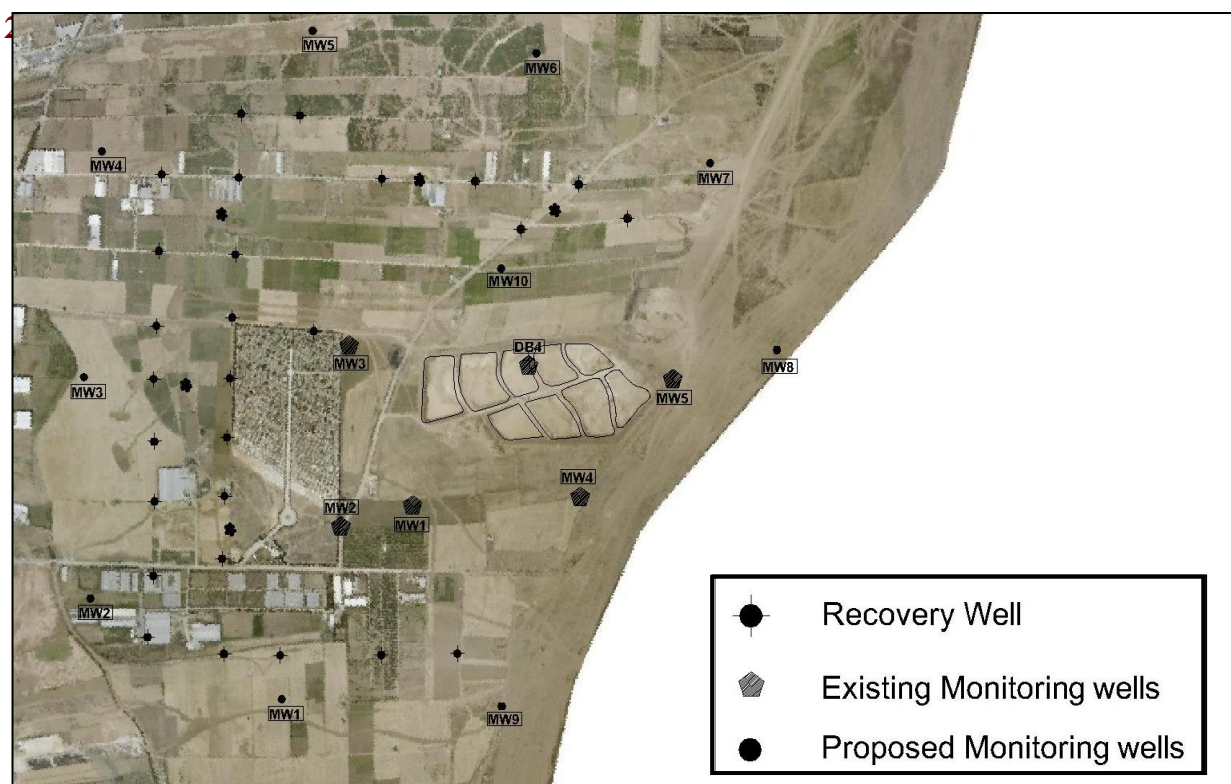


Fig. 2.2: Locations of monitoring wells.

Locations and Sites for the Reuse Scheme

The identified locations of the reuse scheme components, i.e. the water tanks, booster pumping station, and irrigation network are discussed as follows:

2.2.1 Location and Site Plan for the Water Tanks and Booster Pumping Station

The water tanks and booster pumping station lie in the same site. The two 4000 m³ water tanks, the booster pumping station, and associated facilities have been located to the north western side of the cemetery bounded by one road from the north. A new road to the east of the site is proposed by the consultant to be adjacent to the cemetery to provide access to the site as shown in Fig. 2.3. The same location is also proposed to accommodate the water tanks, booster pumping station, and associated facilities needed for the 69,000 m³/day effluent in the future. The total area which is a Waqif land is about 15 donums to accommodate both the current and future extension of almost equal areas. The site layout shown in Fig. 2.3 has been determined such as to allow easy construction of the future components and enable the client to reserve the whole land for the project current and future use. For these purposes the area for current phase is located at inner side of the site while the area for the future use is located at the outer side of the site adjacent to the road.

As for the project overall planning, the site lies almost in the middle of the agricultural land and close to the recovery wells which are distributed around it. These arrangements would result in efficient designs for the piping system connecting recovery wells with the water tanks, the recovery wells, the booster pumping station, and irrigation network. In this case the distances between the site and project physical components would be shorter compared to the case if the site was located at one end of the project area. It should be mentioned that the topography of the site including the water tank elevation is not critical since transmitting of water is carried out using pressure pipes which will be insignificantly influenced by small variation in the elevation head of the tanks.

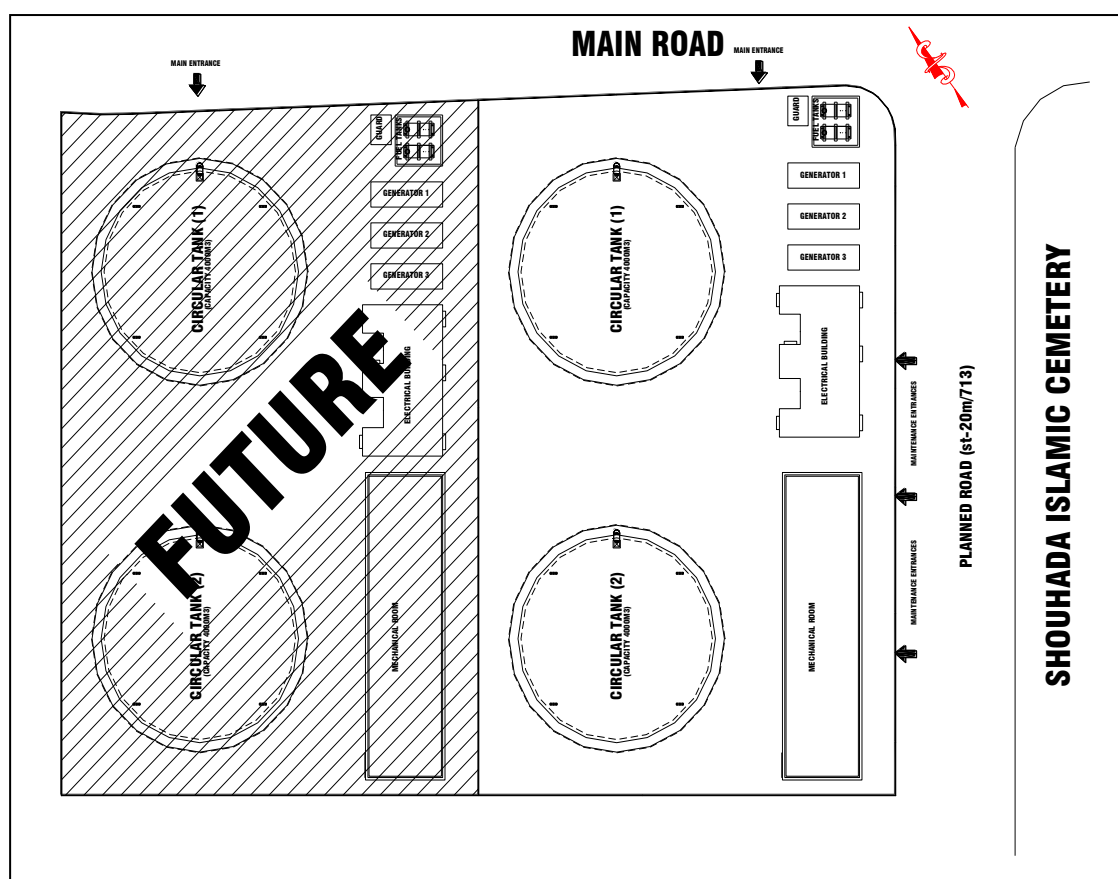


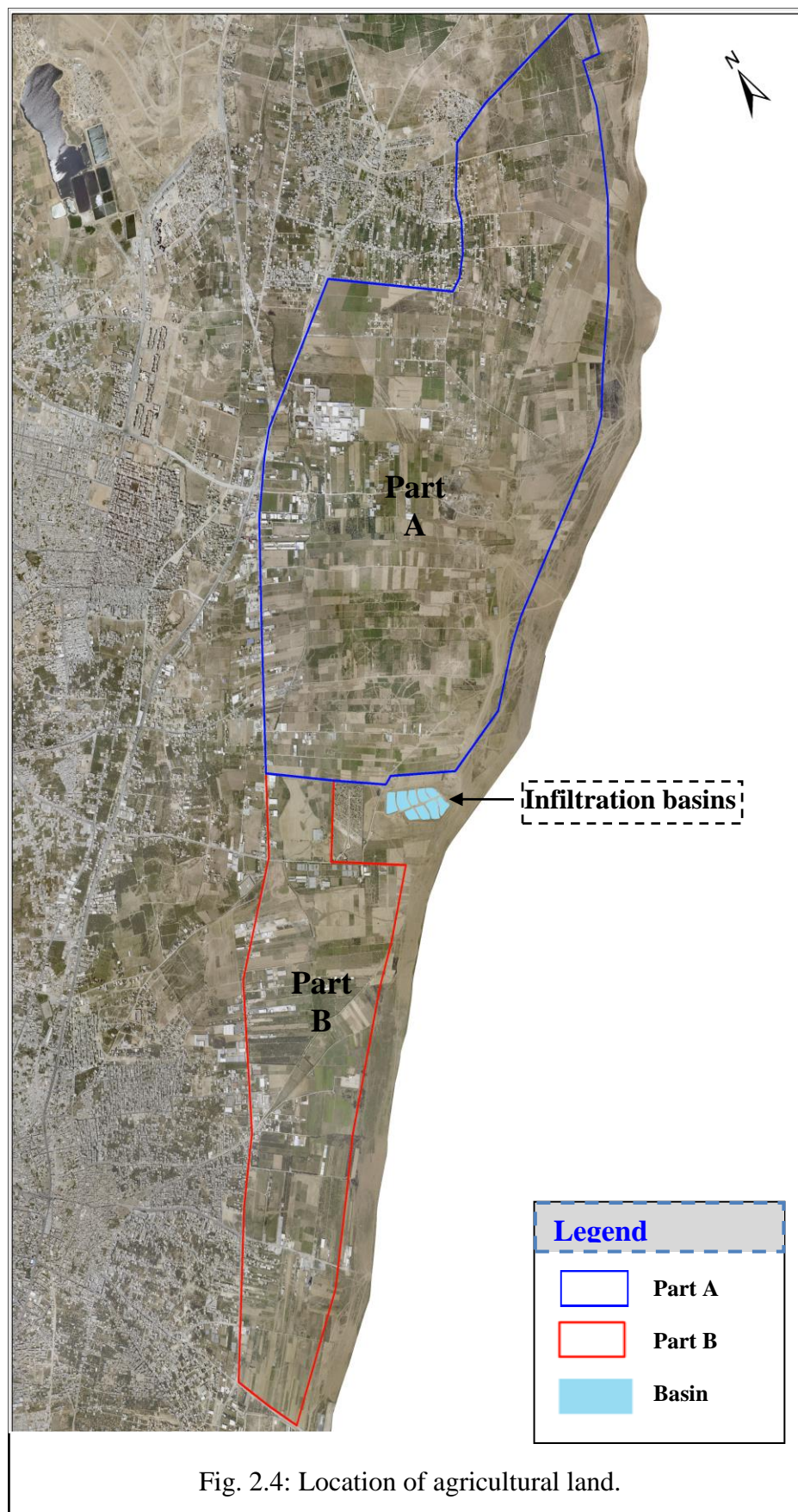
Fig. 2.3: Site location and layout for the water tanks and booster pumping station.

2.2.2 Agricultural Land and Irrigation Network

The total project agricultural area is about 15,000 donums located at the north east of Gaza Strip adjacent to the eastern border as shown in Fig. 2.4. The agricultural area within the project that can be cultivated is about 12,000 donums whereas the remaining land is for other uses such as industrial and residential areas. Generally, the agricultural area can be subdivided into two main parts (A and B) according to their locations from infiltration basins. Part A of about 10,000 donums and Part B of about 5,000 donums are located to the north and south of infiltration basins, respectively as shown in Fig. 2.4.

In accordance with irrigation requirements, irrigation is to be carried out every 6 days. For this purpose, the agricultural land has been subdivided into 6 zones of almost equal sizes, i.e. A (A1 and A2), B (B1 and B2), C (C1 and C2), D, E and F as shown in Fig. 2.5. Each day only one of the zones will be irrigated.

It should be mentioned that the agricultural land was determined in the agricultural report in *Appendix 1* based on cropping patterns, daily and monthly crop water requirements, irrigation methods, and amount of recovered water.



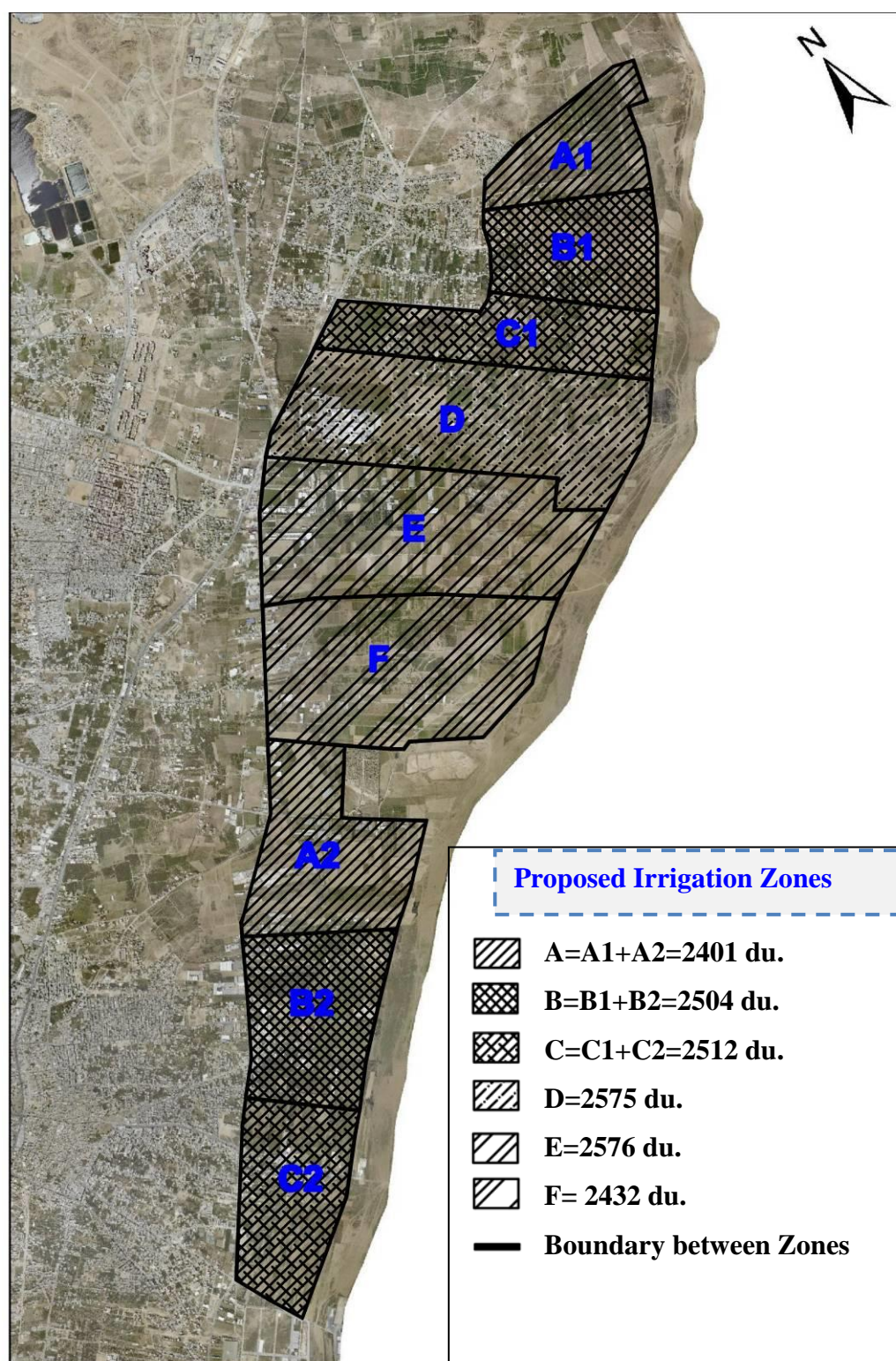


Fig. 2.5: Irrigation Zones.

2.3 Sites for Future Extensions

The future extensions for the 69,000 m³/day effluent for the design year 2025 include additional infiltration basins, water tanks, booster pumping station and associated works, and agricultural land. Tentatively, the requirements for future extensions are of the same order as for current phase effluent, since the increase in the effluent is almost equal to current design value of 35,600 m³/day. The following are the proposed extensions.

2.3.1 Infiltration Basins

The consultant has studied future extension for infiltration basins and recovery requirements that will allow 69,000 m³/day overall infiltration of fully treated wastewater effluent in the design year 2025. The location of land for the new infiltration basins has been identified considering prevailing soil conditions, relation with current facilities, e.g. infiltration basins and treatment plan, project components related to recovery scheme for the first phase, etc.

The location for the extension of the infiltration basins for the 69,000 m³/d effluent is proposed to be adjacent to the treatment plant in the south eastern direction as shown in Fig. 2.6. The soil profile for this location would be most suitable since the top clay layer prevailing in the project area is thin or not existing as indicated in the soil profiles in *Appendix 4*. The identified location is also suitable for the recovery scheme where recovery wells will serve both existing and future infiltration basins. Also, the location is suitable from operational point of view since the operation team will be able to monitor and operate the whole facilities located in the area, i.e. infiltration basins, sewage treatment plant. The recovery wells, monitoring wells and booster pumping station are also located near this location. According to initial calculations there will be a need for about 120 donoms for future infiltration basins.

2.3.2 Water Tanks and Booster Pumping Station

The water tanks, booster pumping station, and associated facilities are proposed to be in the same location for current phase as shown in Fig. 2.6. This location has many advantages as was discussed earlier.

2.3.3 Agricultural Land

The determination of the additional agricultural land of about 15,000 m² is a difficult task considering the scarcity of land in the project area and in Gaza Strip in general. The only available land that can be used in the northern of Gaza Strip is located to north west side. Most of this land is being already used for agricultural purposes. This area also acts as a main source for recharging the groundwater aquifer. Other possible agricultural lands may be located in the southern part of Gaza Governorate and Middle Area Governorate as proposed for example in CAMP study. However, a part of their far locations, these lands are reserved for other local reuse projects serving the concerned governorates.

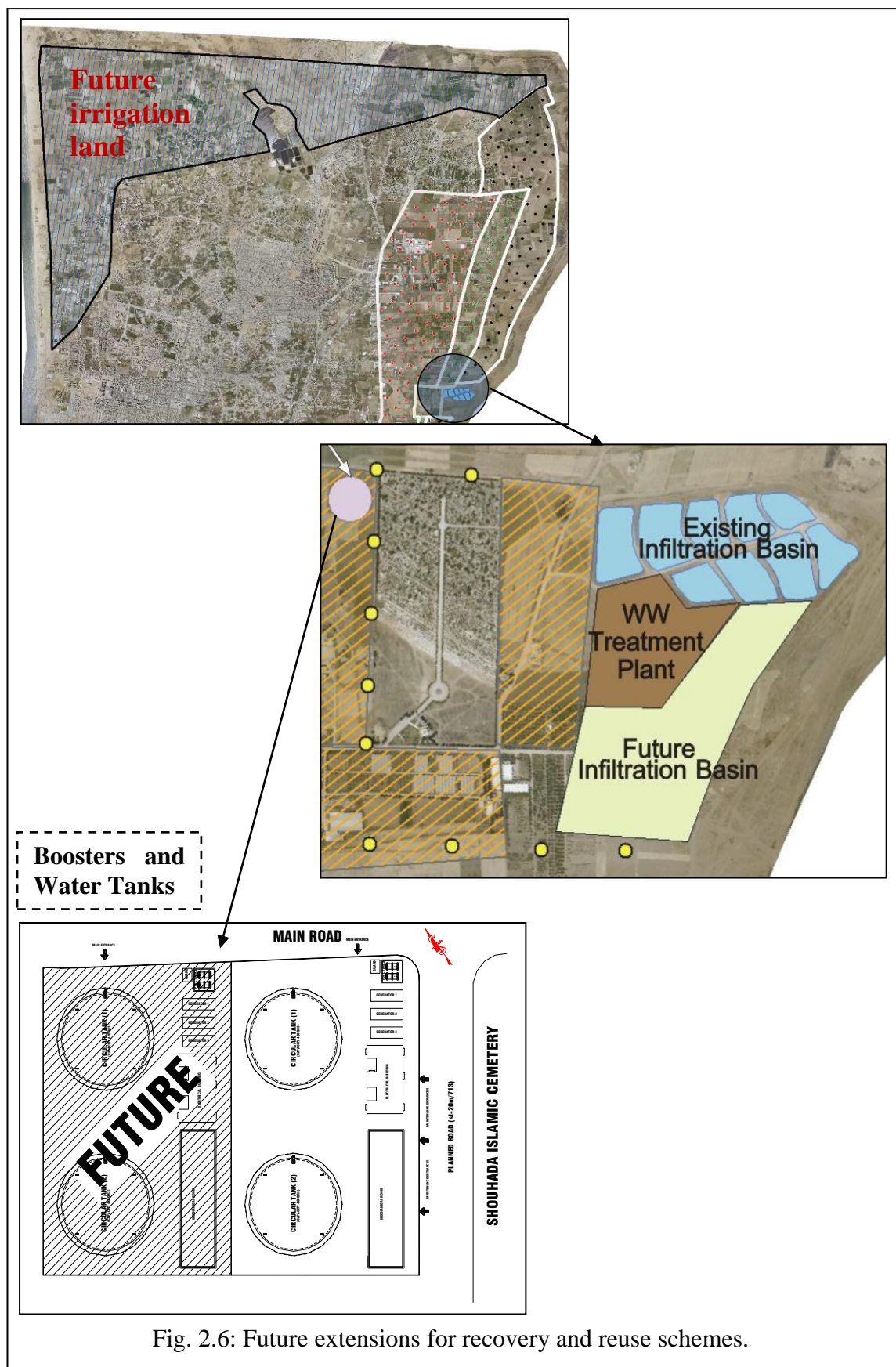


Fig. 2.6: Future extensions for recovery and reuse schemes.

3 SITE INVESTIGATIONS AND SURVEYS FOR RECOVERY AND REUSE SCHEME

3.1 Soil Investigations

3.1.1 Background

The soil testing program which was carried out during the period April – August 2010 aimed to determine the physical, chemical and mechanical characteristics of the soil in the project area. The purpose was to design the recovery and reuse project components based on actual data obtained from the field. In addition, the soil test results assisted the development of the irrigation scheme of the agricultural land including crop patterns, irrigation needs, operation, etc. The mechanical properties of the soil were mainly used for the geotechnical design of the piping network, booster pumping station, water tanks, service buildings, and other associated facilities.

3.1.2 Soil Testing Program

A summary of the soil investigation tests in the project area is given in Table 3.1. Fig. 3.1 shows the locations of these tests. The scope of the investigation included the following testing:

1. Testing of Physical Properties:
 - a. Determination of soil classification, texture, bulk density for about one meter depth at selected 24 locations within the project agricultural land.
 - b. Testing of water holding capacity of 20 locations in the field.
 - c. Testing the surface infiltration rate of selected 5 locations.
2. Testing of chemical properties:
 - a. Conduct the following chemical tests for the first 30 cm of the selected 24 locations:
 - i. EC (Electrical Conductivity) and Salinity;
 - ii. PH (Soil Acidity or Alkalinity);
 - iii. SAR (Sodium Adsorption Ratio);
 - iv. CaCo₃ (Calcium Carbonate).
 - v. Organic Matter.
 - b. Conduct EC (Electrical Conductivity) and Salinity test for soil samples from 30-60 cm of the 24 selected locations.

Table 3.1: Soil testing program.

Item	Description	Unit	Sampling				Tests	
			Type	No. of locations	No. of samples per location	Total no. of samples	No. of tests per sample	Total no. of tests
1	Soil Tests for Agriculture Use							
1.1	Physical Properties							
1.1.1	Soil classification and texture (soil texture, structure, series, Depth and bulk density)	No.	1 m depth pits	8	3	72	1	72
		No.	Test pits*	16				
1.1.2	Water holding capacity test (saturation capacity, field capacity and PWP)	No.	Surface tests	20	1	20	1	20
1.1.3	Surface infiltration rate	No.		5	1	5	1	5
Sub-total for 1.1- Physical Properties				49		97		97
1.2	Chemical Properties							
2.2.1	Chemical Test for Top Layer (0 - 30cm)		1 m depth pits and Test pits*	24				
2.2.1.1	EC (Electrical Conductivity) & Salinity	No.			1	24	2	48
2.2.1.2	PH (Soil Acidity or Alkalinity)	No.			1	24	1	24
2.2.1.3	SAR (Sodium Adsorption Ratio)	No.			1	24	1	24
2.2.1.4	Caco3 (Calcium Carbonate)	No.			1	24	1	24
2.2.1.5	Organic Matter	No.			1	24	1	24
2.2.2	Chemical Test for 2nd. Layer (30 - 60cm)							
2.2.2.1	EC (Electrical Conductivity) & Salinity	No.			1	24	2	48
Sub-total for 1.2- Chemical Properties				24		144		192
Total for 1. Soil Tests for Agriculture Use				49		241		289
* Same location used for sampling of both the agriculture use and irrigation network different tests.								
2	Soil Tests for Irrigation Network and Recovery Piping System							
2.1	Seive analysis	No.	Test pits*	16	1	16	1	16
2.2	Natural water content.	No.			1	16	1	16
2.3	Liquid limit + Plastic limit	No.			1	16	2	32
Total for 2. Soil Tests for Irrigation Network and Recovery Piping System				16		48		64
* Same location used for sampling of both the agriculture use and irrigation network different tests.								

Item	Description	Unit	Sampling				Tests	
			Type	No. of locations	No. of samples per location	Total no. of samples	No. of tests per sample	Total no. of tests
3	Soil Tests for Bosster Pumping Station, Water Tanks and other Facilities							
3.1	Drilling (2) boreholes of 25 m depth	Lump Sum	Boreholes	2				
3.2	Drilling (2) boreholes of 15 m depth	Lump Sum		2				
3.3	Sieve analysis	No.		4	2	8	1	8
3.4	Natural water content.	No.			2	8	1	8
3.5	Liquid limit + Plastic limit	No.			2	8	2	16
3.6	Standard Penetration Test (SPT)	Lump Sum			53	53	53	53
3.7	Consolidation Test #	No.			1	1	1	1
Total for 3. Soil Tests for Bosster Pumping Station, Water Tanks and other Facilities				4		78		86
# Test will be executed if the soil is clay.								

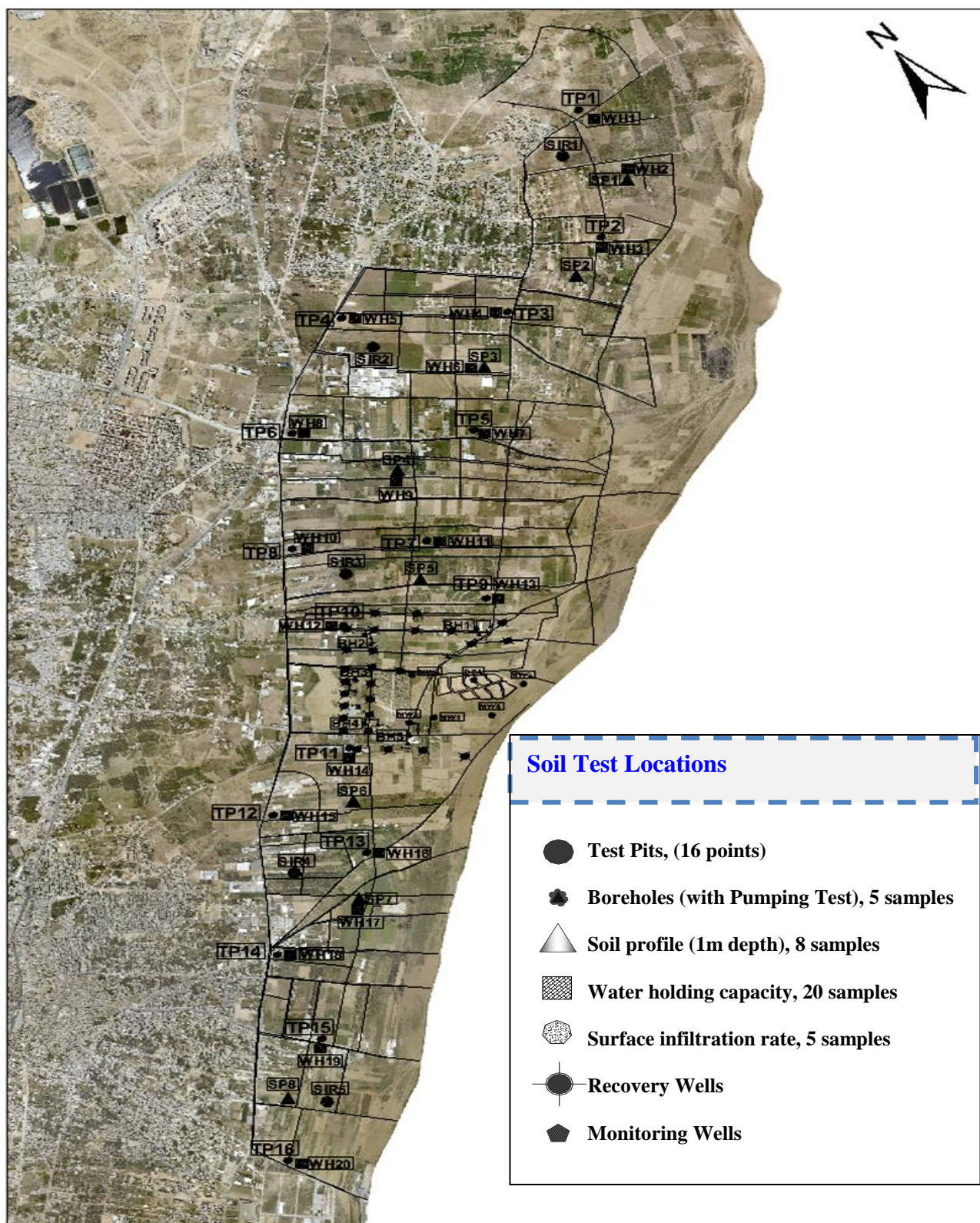


Fig. 3.1: Location of soil tests.

3.1.3 Results of Soil Tests for Agriculture Use

The following is a summary of soil test results for agricultural use. *Appendix 4* includes the detailed soil test results for each location.

3.1.3.1 Soil Classification and Texture

Soil texture classification was made in accordance with Brady classification chart for loams. Most of the soil in the agricultural area was found to be **Loamy soil (Sandy Loam, Silt Loam, Loamy Sand)**. **Sand** was found in few locations only. The loamy soil is suitable for agricultural purposes where a wide range of crops can be cultivated in this soil as detailed in the agricultural report in *Appendix 1*. Table 3.2 includes a summary of the soil classification in the various locations.

Table 3.2: Texture soil classification.

Location	Soil Type
SP1, SP2, SP5, , SP7, TP1, TP3, TP4, TP5, TP6, TP8, TP10, TP12,	Sandy Loam – Loamy Sand
SP8, TP7, TP11	Silt Loam
TP2	Loamy Sand - Sand
SP3,TP9, TP13, TP14, TP15, TP16, SP4, SP6	Sandy Loam- Silt Loam

3.1.3.2 Test Results for Surface Infiltration

Five locations were selected for surface infiltration tests as shown in Fig. 3.1. Testing was carried out using Single Ring Infiltrometer – Falling Head Method. Table 3.3 shows the results of ultimate infiltration capacity at each location. The infiltration capacities are high “around 20 cm/hr.” for sand soil and low “around 9 cm/hr.” for sandy silt soil. The obtained ultimate infiltration capacities were used in determining the irrigation requirements for each crop type as detailed in the agricultural report in *Appendix 1*.

Table 3.3: Ultimate infiltration capacity for irrigation use.

Location	Ultimate infiltration Capacity cm/hr	Soil Type (UNIFIED)
SIR 1	20.4	Sand
SIR 2	8.4	Sandy Silt
SIR 3	18.0	Silty Sand
SIR 4	14.4	Silty Sand
SIR 5	9.6	Sandy Silt

3.1.3.3 Test Results for Field Water Holding Capacity

Water holding capacity tests were conducted in the field for 20 test locations shown in Fig. 3.1. Soil samples were taken after 48 hours and tested for moisture content in the lab. Table 3.4 shows the obtained test results. The water holding capacities ranged from 25.9 to 6.4 (%age by weight) for sandy silt soil and silty sand soil, respectively. The obtained water holding capacities were used in determining the irrigation requirements for each crop type as detailed in the agricultural report in *Appendix 1*.

Table 3.4: Field water holding capacity.

Location	Water Holding Capacity %age by Weight	Soil Type
WH1	11.7	Silty Sand
WH2	12.9	Silty Sand
WH3	17.3	Silty Sand
WH4	23.1	Sandy Silt
WH5	19.6	Sandy Silt
WH6	25.9	Sandy Silt
WH7	22.5	Sandy Silt
WH8	17.6	Silty Sand
WH9	25.1	Sandy Silt
WH10	18.4	Sandy Silt
WH11	23.8	Sandy Silt
WH12	25.1	Sandy Silt
WH13	22.8	Sandy Silt
WH14	10.7	Silty Sand
WH15	6.4	Silty Sand
WH16	31.5	Sandy Silt
WH17	17.5	Silty Sand
WH18	23.8	Sandy Silt
WH19	22.4	Sandy Silt
WH20	23.2	Sandy Silt

3.1.3.4 Chemical Test Results

Samples for chemical analysis were taken for each of the twenty locations shown in Fig. 3.1. Tables 3.5 and 3.6 include the test results representing the depths of (0 to 30 cm) and (30 to 60 cm), respectively. The results of the soil chemical tests assist the determination of fertilizing requirements, crop types and irrigation requirements as detailed in the agricultural report in *Appendix 1*.

Table 3.5: Results of chemical tests for (0 to 30 cm) depth.

Location	EC μ S/cm	TDS mg/l	pH	Organic Matter	SAR	CaCo3
TP1	520	322	8.32	1.2	4	12
TP2	628	389	8.83	2.4	1.7	11
TP3	630	391	7.77	3.2	2	17
TP4	260	161	7.84	5.4	1.25	16
TP5	445	276	7.72	3.2	1	14
TP6	240	149	7.72	4.4	1	11
TP7	420	260	7.76	5	1.9	14
TP8	1330	824	7.64	1.6	2.2	12
TP9	270	167	8.15	3.8	1.85	16
TP10	870	539	8	1.6	3	14
TP11	500	310	7.89	5	1.8	12

TP12	311	193	8.06	1	0.9	10
TP13	285	177	7.94	1.4	1.7	16
TP14	645	400	7.76	6.2	1.3	17
TP15	300	186	8.07	2.8	1.8	13
TP16	804	499	8.61	4.2	2.89	20
SP1	485	300	8.18	4.6	2.1	16
SP2	560	347	8.16	4.4	1.5	12
SP3	338	209	7.89	4.8	1.5	13
SP4	390	242	7.76	4.4	1	15
SP5	521	323	8.13	2.6	1.57	15
SP6	743	461	7.97	4.6	2.6	11
SP7	256	159	7.91	3	1.4	11
SP8	385	239	8.06	1.2	2.3	19

Table 3.6: Results of chemical tests for (30 to 60 cm) depth

Location	EC μS/cm	TDS mg/l
TP1	426	264
TP2	410	254
TP3	730	453
TP4	353	219
TP5	404	251
TP6	670	415
TP7	605	375
TP8	1,288	799
TP9	533	330
TP10	570	353
TP11	555	344
TP12	411	255
TP13	512	317
TP14	523	324
TP15	355	220
TP16	612	379
SP1	514	319
SP2	571	354
SP3	346	215
SP4	281	174
SP5	565	350
SP6	411	255
SP7	344	213
SP8	441	273

3.1.4 Soil Tests for Irrigation Network and Recovery Piping System

The purpose of the soil tests for the design and construction of the piping systems is to investigate the surface and subsurface condition of the soil, describe the soil profile within the

site, and to determine the physical and mechanical properties of the soil strata. This is to provide the designer with sufficient information for the design and construction of the irrigation network and collection piping system. The scope of the work included the following testing:

1. Excavation of 16 tests pits (4 m depth) shown in Fig. 3.1 where samples were taken every 0.5 m in depth.
2. Conducting laboratory testing on soil samples including:
 - a. Sieve analysis of 16 samples;
 - b. Determination of moisture content for 16 samples;
 - c. Determination of liquid and plastic limits for 16 samples.

The following is a summary of soil test results for irrigation network and recovery piping system design. *Appendix 4* includes the detailed soil test results for each location.

3.1.4.1 Sieve Analysis and Soil Classification

Table 3.7 shows the results of sieve analysis and soil classification according to Unified Soil Classification System (USCS).

Table 3.7: Soil classification based on the results of sieve analysis.

location	Classification
TP1, TP2, TP3	Poorly Graded Clayey Silty Sand
TP4, TP5, TP8	Poorly Graded Clayey Sandy Silt
TP6, TP7, TP9, TP12, TP13, TP14, TP16	Uniform Sandy Silt
TP10, TP11, TP15	Uniform Clayey Silt

3.1.4.2 Soil Plasticity

For the 16 locations, Atterberg limits were found using the Cone Penetration Method. Table 3.8 shows the soil plasticity results at each location.

Table 3.8: Soil plasticity.

location	Depth (m)	L.L	P.L	P.I
TP1	4	17	NP	NP
TP1	2	35	10	25
TP3	1	27	12	15
TP4	2	41	25	16
TP5	3	45	13	32
TP6	4	23	10	13
TP7	4	41	14	27
TP8	3	35	15	20
TP9	4	61	16	45
TP10	2	39	11	28
TP11	3	37	18	19
TP12	2	33	11	22
TP13	2	43	15	28
TP14	3	44	21	23
TP15	4	36	14	22
TP16	2	42	13	29

3.1.4.3 Natural Moisture Content

Table 3.9 shows the test results of natural moisture content.

Table 3.9: Natural moisture content.

Location	Depth(m)	WC %
TP1	2	9.8
TP2	3	1.3
TP3	4	1.0
TP4	2	16.3
TP5	3	14.7
TP6	4	7.6
TP7	4	14.1
TP8	3	16.7
TP9	4	18.6
TP10	2	16.8
TP11	3	14.7
TP12	2	10.0
TP13	2	16.5
TP14	3	20.6
TP15	4	19.6
TP16	2	20.4

3.1.4.4 Main Conclusions of Soil Test Result Regarding Piping Systems

The results indicate that the existing soil types are not suitable for backfilling, especially underneath and around the pipes within 50cm thickness. Clean sand must be used as a backfilling soil. Trench excavation must be at 1 vertical to 3 horizontal slopes, or proper shuttering system must be implemented.

3.1.5 Soil Tests for Structural Design

3.1.5.1 Tests

The scope of investigation includes conducting geotechnical tests for structural design of the booster pumping station, water tanks and other facilities. The testing included drilling 2 boreholes of 25 m and 2 boreholes of 15 m depths in locations shown in Fig. 3.2. SPT tests are conducted on site for each location every 2 m or change of layer up to 20 m depth. The following tests are conducted on selected samples in the laboratory:

- a. Sieve analysis
- b. Natural water content
- c. Atterberg limits for clayey soils
- d. Unconfined Compression Strength
- e. Consolidation test

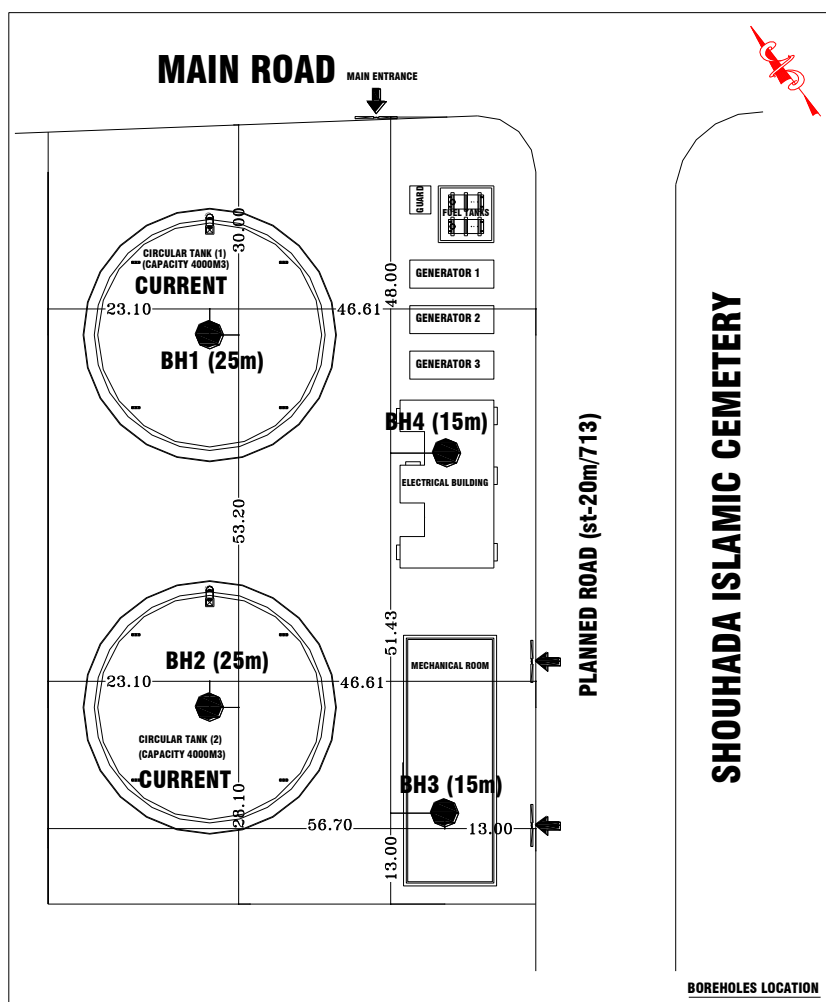


Fig. 3.2: Locations of soil bore logs for structural design.

3.1.5.2 Main Conclusions of Soil Test Result Regarding Structural Design

Mat foundation will be used for the storage tanks at net allowable bearing capacity of 100 kPa. The minimum granular structural backfilling replacement thickness is 2.5m below the mat base. General required recommendations that the rain water and facility water should be prevented to penetrate to foundation by paving the area of the site and no planting and tightening all pipes connections. The estimated total and differential settlements for the foundation at the center and the edge of the mat are less than the allowable limiting values of 51mm and 19mm, respectively.

Strip foundation are recommend for the design of the booster pumping station, electrical buildings and other facilities at net allowable bearing capacity of 75 kPa with minimum soil replacement by granular soil backfilling thickness of 1.5m below the base.

Regarding the road design, the top soil is clay of medium plasticity which is not suitable as sub-grade for road construction. It should be excavated up to 0.3m and replaced by kurkar fill with minimum CBR of 30%.

3.2 Hydrogeological and Water Quality Investigation

3.2.1 Introduction

Field investigations had been carried out on the proposed infiltration site by SWECO, 2003. In January 2010, PWA had finished the construction of 5 monitoring wells of the infiltration basin. Based on the information collected from the past investigations in the project area, five boreholes were drilled in a distance of 500 to 1000 m from the basin in the current project to complete the extension of the different geological layers in the aquifer.

The SWECO investigations results served as a fundamental source of information for the evaluations and conclusions of the EA report and the groundwater model. The new investigations were used to update the groundwater model and the assessment of the groundwater quality status. All information collected from SWECO, PWA investigation and the current investigations will be used in groundwater modeling and the design of the recovery wells. A summary of the most important results is given in this section. The complete report of the new hydrogeological investigation carried out under the current project (May, 2010) is presented in *Appendix 4*.

3.2.2 Testing Program

A summary of the hydrogeological investigation tests in the project area is given in Table 3.10. Fig. 3.3 shows the locations of these tests. The scope of the investigation included the following testing:

Table 3.10: Details of hydrogeological tests.

Item	Description	Unit	Sampling				Tests		
			Type	No. of locations	No. of samples per location	Total no. of samples	No. of tests per sample	Total no. of tests	
1	Hydrogeological Tests								
1.1	New Hydrogeological Tests								
1.1.1	Drilling pilot boreholes around 70-80 m depth	No.	Pilot boreholes **	5					
1.1.2	Hydraulic permeability testing (to reach to kurkar layer)	No.			1	3	1	3	
1.1.3	Laboratory permeability test (in ground water after reach GWL)	No.			1	3	1	3	
1.1.4	<u>Chemical Properties for well water (New)</u>								
1.1.4.1	PH & TDS	No.			1	5	2	10	
1.1.4.2	NO ₃	No.			1	5	1	5	
1.1.4.3	CL	No.			1	5	1	5	
1.1.4.4	N				1	5	1	5	
1.1.4.5	BOD				1	5	1	5	
1.1.4.6	COD				1	5	1	5	
1.1.4.7	P (Phosphorous)				1	5	1	5	
1.1.4.8	TSS				1	5	1	5	
1.1.4.9	NH ₄				1	5	1	5	
1.1.4.10	NO ₂				1	5	1	5	
1.1.4.11	O ₂				1	5	1	5	
1.1.5	<u>Mechanical Properties</u>				1	5	1	5	
1.1.5.1	Sieve analysis	No.			6	30	6	30	
1.1.5.2	Natural water content.	No.			6	30	6	30	
1.1.5.3	Liquid limit+ Plastic limit	No.			4	20	2	40	
1.1.6	Pumping – recovery test -2 days	No.		5			1	5	
Sub-total for 1.1- New Hydrogeological Tests				10		146		176	
** Same location used for sampling of both the pilot boreholes use and pumping recovery test different tests.									
1.2	Previous Hydrogeological Tests								

Item	Description	Unit	Sampling				Tests	
			Type	No. of locations	No. of samples per location	Total no. of samples	No. of tests per sample	Total no. of tests
1.2.1	BOD	No.	Existing wells and basins	7	1	7	1	7
1.2.2	COD	No.			1	7	1	7
1.2.3	NO ₃	No.			1	7	1	7
1.2.4	T.N	No.			1	7	1	7
1.2.5	CL	No.			1	7	1	7
1.2.6	Hco3	No.			1	7	1	7
1.2.7	Ca	No.			1	7	1	7
1.2.8	Mg	No.			1	7	1	7
1.2.9	K	No.			1	7	1	7
1.2.10	Na	No.			1	7	1	7
1.2.11	Heavy metals(Cu) Al Azhar	No.		1	1	1	1	7
1.2.12	Heavy metals(Cd) Al Azhar	No.					1	7
1.2.13	Heavy metals(Pb) Al Azhar	No.					1	7
1.2.14	Heavy metals (Boron) MoAg.	No.					1	7
1.2.15	Water level	No.		6	1	6	1	6
1.2.16	PH, EC & TDS	No.			1	6	3	18
1.2.17	Detergent	No.		5	1	5	1	5
1.2.18	F.C	No.			1	5	1	5
1.2.19	Helminthes eggs	No.			1	5	1	5
Sub-total for 1.2- Previous Hydrogeological Tests				18		98		137
Total for 1. Hydrogeological Tests				28		244		313

3.2.3 Lithology Description and Pumping Tests

The hydrological tests comprise the followings:

1. Drilling 5 pilot boreholes in the locations labeled on Fig. 3.3 BH-1 to BH-5. From past investigations it was found that the depth of groundwater level ranged between 45 m to 65 m and the depth of kurkar layer ranged between 65–85m. Based on that the depth of the new boreholes were on the range of 65-85 m which depends on the location of water

table. Each borehole is drilled to a distance below the water level and inside the kurkar layer equal to the length of the screen of the wells that is used for pumping test.

As noticed in the design section, the designed pumping rate of the recovery wells is 170 m³/hr. Therefore, a one step draw down test was carried out using the designed pumping rate with a borehole of a diameter of 18 inches is used in order to facilitate the insertion of 12 inch diameter filter, pipes and gravel backfilling. A step draw down test is performed at the first borehole BH3 where the test is conducted in 3 to 4 stages. Based on the results of the step test regarding to the well efficiency, the final Q for the long term pumping is identified. Step draw down test is performed by changing the pumping rate in successive steps, each lasting for sufficient period of time as shown in Table 3.11. The pumping is increased to 170 m³/h to check the behavior of the well and the aquifer. Pumping with certain rate is continued until steady state is satisfactorily reached. Basically tested yield/drawdowns shows the trend when test is successful. Results give basis for analyzing specific capacity of the wells.

Table 3.11: Step draw down test.

Step	Pumping rate	Duration
1	50 m ³ /h	2 hours
2	100 m ³ /h	2 hours
3	150 m ³ /h	2 hours

- Hydrogeological properties may not be the same in the planned well field area. To find out heterogeneity of the area, four constant rate pumping tests are carried out. Based on the results of the step drawdown test in BH3, pumping rate for the long term pumping test is 70 m³/hr. The four boreholes are of 10 inch diameter to insert 8 inch diameter filter to carry the rest of long pumping tests. Before the tests one measuring round is made in all of the observation points. The rates shown in Table 3.12 of water level measurements are applied in both step drawdown test and pumping tests. The test is terminated when the decline of water level in the observation wells is stopped which should did not exceed 48 hrs.

Table 3.12: Rates of water level measurements.

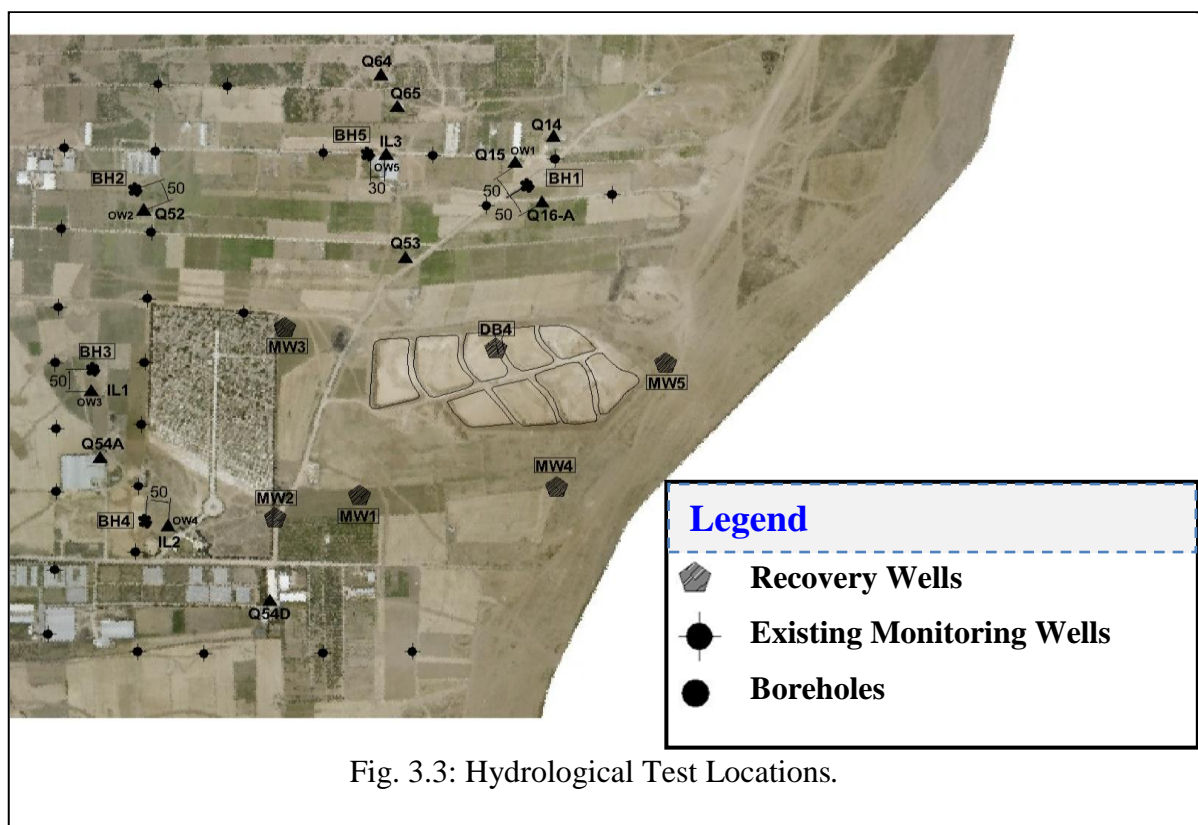
Time (since start of pumping) (min)	Time intervals between measurements (min)
0-60	2
60-120	5
120-240	10
240-360	30
360-1440	60
1440-termination*	480

3. The variation of the water level with time is measured in two wells, i.e. one in the pumping borehole and the second in the nearby agricultural well or infiltration basin monitoring well. In the same borehole used for pumping, water level is measured using steel pipe which inserted in the borehole with diameter 1 inch and at least 10m below the water level. Water level instrument is inserted in the pipe to determine the water level drawdown. In addition, agricultural wells in the area are used as an observation well at distance between 10m to 100m, from each pilot borehole. The wells are shown in Table 3.13.

Table 3.13: Boreholes and monitoring wells.

Borehole	Monitoring well	Distance in one direction	Distance in the other direction
BH1	Q14 – Q15	50m	60m
BH2	Q52	50m	0
BH3	IL1	50m	0
BH4	IL2	50m	0
BH5	Q54D	50m	0

4. Water depth observations and measurements are recorded in special forms included in *Appendix 4*. The pumping rate is monitored continuously by observing the time-flow readings at the flow meter. The flow readings are recorded in special forms included also in *Appendix 4*.



5. Testing of the hydraulic permeability at each pilot borehole at depths selected according to soil stratification encountered at site up to 10 m depth. This test is to measure the coefficient of permeability of soil layers which will help in the hydraulic characteristic of the unsaturated zones.
6. Sampling soils every 2 m or change of layer at each pilot borehole for soil classification above the water table.
7. The main emphasis is in layers below water table since the main objective of this investigation is designing wells. Below water table, soil samples are taken at every three meters. Soil samples are visually inspected and classified and depending on soil changes, reasonable amount of samples are sieved and the results documented. For the soil samples under water table at each pilot borehole, a laboratory permeability test is performed.
8. Each soil sample is sealed, labeled, and transported to the lab in accordance with relevant standards for laboratory testing.
9. Conducting laboratory tests on selected representative samples for mechanical properties of soil as follows:
 - i. Sieve analysis (No. 30)
 - ii. Natural water content (No. 30)
 - iii. Liquid and plastic limit for clayey soils. (No. 20)
10. Conducting the following laboratory chemical tests on water samples collected during the investigation at the end of pumping (for 5 wells):
 - i. PH, EC & TDS
 - ii. NO₃
 - iii. Cl
 - iv. NH₄
 - v. NO₂
 - vi. O₂

3.2.4 Recovery Test

The aquifer tests also include recovery tests where the recovery of groundwater level is measured after pumping from the well is stopped. The same measurement time interval which was used in the pumping test is used in measuring the recovering groundwater table.

3.2.5 Results of Pumping Tests

The detailed analysis and results of pumping tests are found in **Appendix 4**. The following is a summary of main findings.

To study the hydrologic properties of the aquifer, as part of the investigation program, the planned recovery well field area was studied with one step drawdown test (50–170 m³/h) which was carried out in BH3 and four constant rate pumping tests (70 m³/h). The aquifer tests included

also recovery of groundwater level test where the groundwater level was measured after pumping from the well is stopped.

The results of the step draw down test carried out in BH3 is shown in Table 3.14. Table 3.15 shows the results of the long pumping test results where it shows how aquifer parameters are derived from the drawdown data and corresponding type curves. Detail pumping test methods and results are shown in pumping test report (*Appendix 4*).

Table 3.14: Results of Step Draw Down test in BH3

Pumping Rate (m ³ /hr)	Well Losses (m)	Well Efficiency (%)	Specific Capacity (m ² /d)
50	0.138	86.86	1043.47
100	0.533	79.91	960.0
150	1.245	70.02	841.12

Table 3.15: Data extracted from pumping test

ID	T (m ² /d)	Sy (%)	b (m)	K _{mean} (m/d)
BH1	5557.21	17	75	78.22
BH2	6222.73	5.5	75	48.60
BH4	5259.48	18	75	75.14
BH5	8178.28	19	75	63.77
DB4*	4147.2	20	75	55.0
Mean	5872.98	16%		64.146

T = transmissivity, Sy = specific yield; b = thickness of aquifer which is used in computing the aquifer parameters; K = hydraulic conductivity. , * The data were collected based on SWECO soil investigation, 2003.

It is important to note that special care was taken in analysis comparing step drawdown results with the other constant rate pumping tests to make judgment of hydraulic properties on areal basis. Also there is valuable material about previous pumping test (SWECO, 2003) made near the infiltration basins in DB4. Altogether analyzing these pumping tests indicated the guidelines for best applicable recovery scheme.

3.3 Topographical Survey and Digital Maps

3.3.1 Background

The topographical survey carried out during the period March–April 2010 covered the piping network routes, booster pumping station, water tanks, service buildings, wells, and other associated facilities as shown in Fig. 3.4. The route topographic survey included two types; full corridor and spot elevations every 100 m. The total length of the route survey is about 80 km. The survey has reflected the coordinate system of the Palestinian Grid System (PALNET) and the levels be related to the Mean Sea Level (MSL).

3.3.2 Scope of Topographical Survey

The survey work was carried out in two phases:

Phase I: Full Corridor Survey for some roads as shown Fig. 3.4 of about 24 km in which all surface features were located on plans with elevations recorded every 20 meters.

Phase II: Spot Elevation Survey for other roads as shown Fig. 3.4 of about 56 km in which spot elevation every 100 meters is computed using Gaza Digital Terrain Model (DTM).

3.3.3 Digital Topographical Maps

The main results of the whole network survey for both the full corridor and the spot level topographical survey is shown in Fig 3.5. Fig. 3.6 shows the topographical survey for the area surrounding the site layout for the booster pumping station and associated facilities. Other surveys for the site, wells, etc. are determined after approving the design report. The digital topographical survey is enclosed in *Appendix 5*.

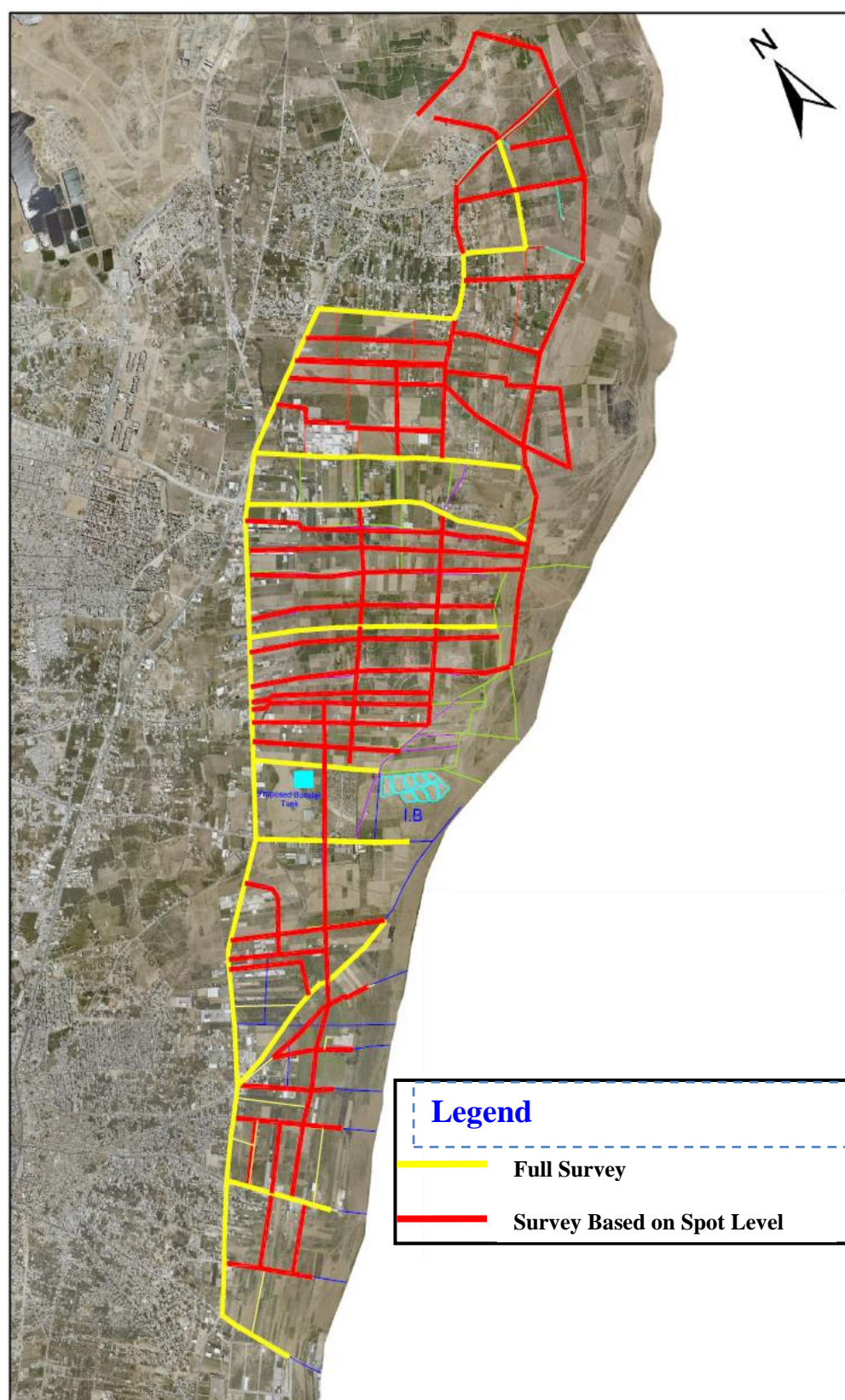


Fig. 3.4: Rout topographical survey.

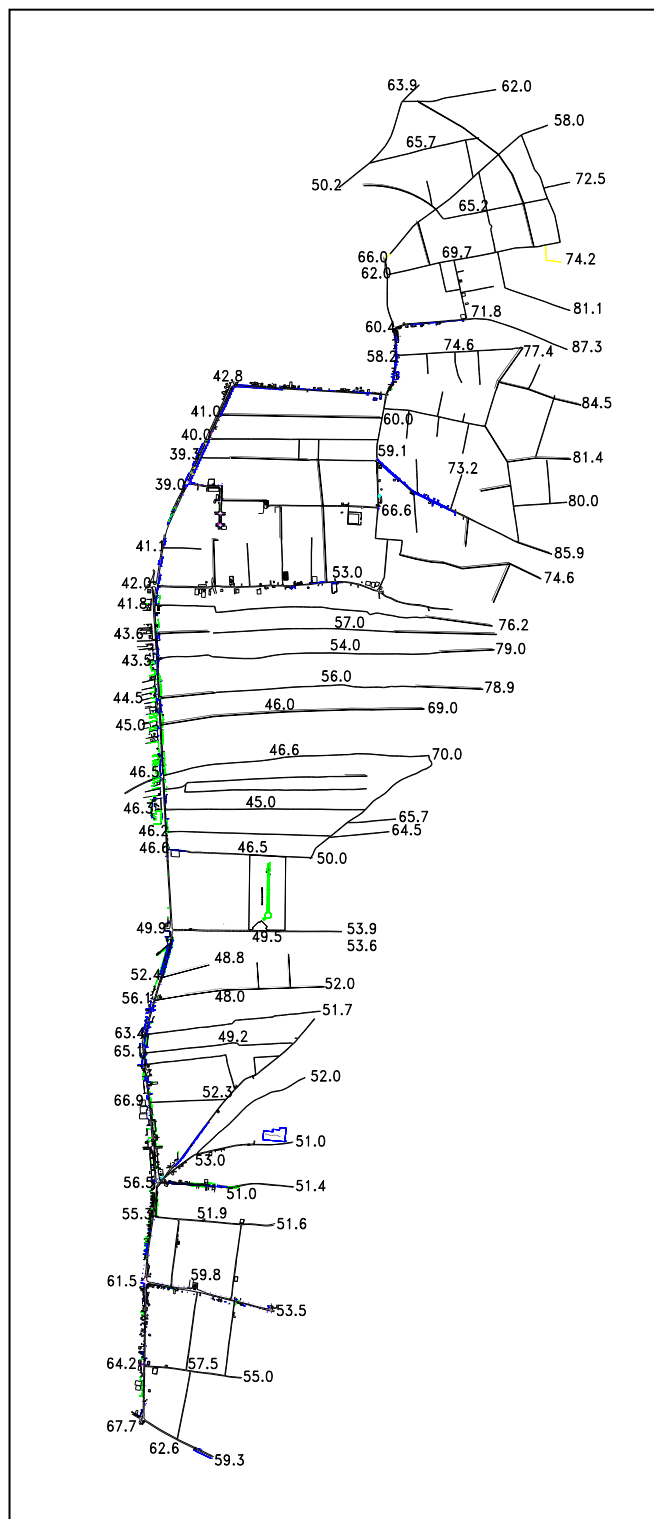


Fig. 3.5: Topographical survey for the project area (for full details refer to digital map in *Appendix 5*).

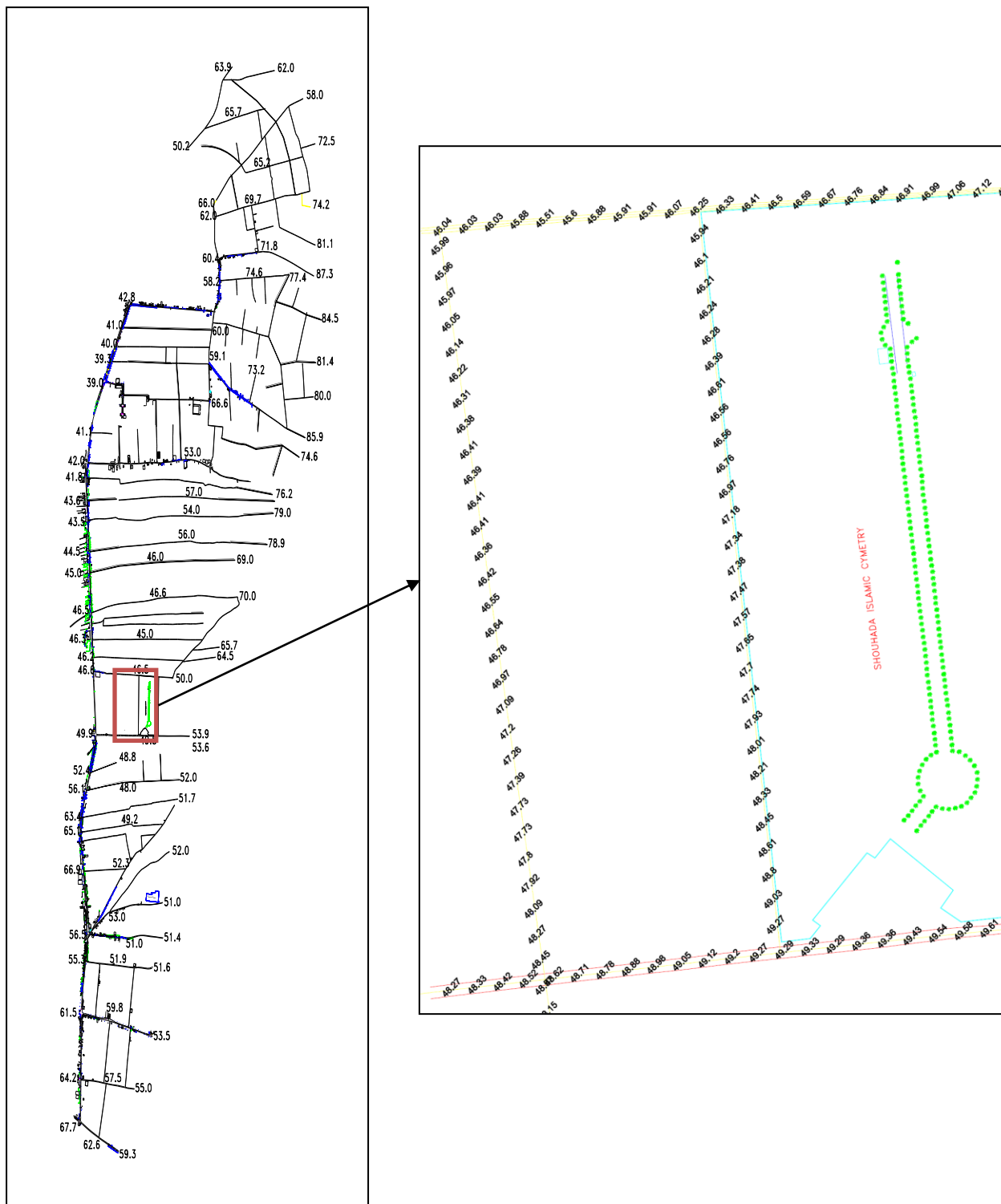


Fig. 3.6: Topographical spot level survey for the area surrounding the booster pumping station and associated facilities (for full details refer to digital map in *Appendix 5*).

3.3.4 Main Findings of the Topographical Survey

Fig. 3.5 indicates that in general, the topography of the project area is a flat sloping land descending at 1.5% to 2% towards the western direction in the northern part of the agricultural zone. Also, the land is descending at 1% towards the eastern direction in the southern part of the agricultural zone. Regarding the northern-southern direction, the land slopes at 0.5% to 1% towards either the north or the south directions. The highest and lowest topographical levels in the project area are 87m and 40m located at the northern-east and northern west sides of the agricultural land, respectively.

The site layout topography for the water tanks, booster pumping station and associated facilities is almost flat with less than 2 m difference and average level of 46m as shown in Fig. 3.6. The maximum difference in topographical levels between the booster pumping station and the irrigation net works is about 50m. While the maximum difference in the topographical levels between the recovery wells and water tanks is about 18m. The results of the topographical survey have been considered in the planning and design of the various project components in the project area.

4 EXPERIENCE FROM SIMILAR SYSTEMS

The purpose of this chapter is to present relevant experience from recovery and reuse schemes implemented elsewhere and describe the design criteria used in these schemes. Another aim is to bring out the positive and negative experience from previous projects. This information can provide lessons-learned for the Palestine project, remembering the local weather and environment on the coast of Mediterranean Sea.

4.1 Relevant Experience from Recovery Schemes

4.1.1 Finland Recovery Scheme Experience

FCG's experience of artificial groundwater comes from Northern Europe (Finland). Today, 60% of the water distributed by Finnish waterworks is groundwater, and the proportion of artificial recharge is about 20% of the total water use. Artificial groundwater is produced from lake water for household water supply purposes.

Lake water is pumped to be infiltrated through spreading basins or sprinkling areas, or sometimes lake bank filtration is used. Some of the biggest groundwater recharge plants and their design criteria are listed in Table 4.1.

Table 4.1: Finnish groundwater recharge plants.

Case 1: Jäniksenlinna DWTP

- Capacity: 12 300 m³/d (800 m³/h)
- Water source: surface water (Lake Päijänne)
- Recharge method: surface spreading in basin
- Infiltration area: 4 500 m²
- Infiltration rate: 2.7 m/d
- Aquifer hydraulic conductivity: 0.0012 m/s (100 m/d)
- Flow distance to uptake wells: 480–700 m
- Retention time: 36–51 days

Case 2: Jänneniemi DWTP¹

- Capacity: 20 000 m³/d (840 m³/h)
- Water source: surface water (Lake Kallavesi)
- Recharge method: lake bank filtration
- Infiltration line: waterfront 4,7 km
- Infiltration rate: 2 m/d
- Average retention time 150 d

Case 3: Kuivala - Utti DWTP

- Capacity: 17 000 m³/d
- Water source: surface water (Lake Haukkajärvi)
- Recharge method: surface spreading in basins
- Infiltration area: 9 200 m² area
- Infiltration rate: 1.8 m/d

Case 4: Rusutjärvi DWTP

- Capacity: 20 000 m³/d
- Water source: surface water (Lake Päijänne)
- Recharge method: sprinkling
- Aquifer hydraulic conductivity: 0.00081 m/s (70 m/d)
- Flow distance to uptake wells: 640–780 m
- Retention time: 35–65 days

The above mentioned four cases have been constructed in close co-operation with FCG Finnish Consulting Group Ltd.

Generally, Nordic experience of groundwater recharge systems has mainly been positive. Artificial recharge has managed to increase the groundwater resources with the help of natural water infiltration process (Fig. 4.1). Flow distance to uptake wells is typically in the range of 200 to 1000 m and retention time varies from 7 to 150 days. Recharge water is pumped (or bank filtrated) from natural lakes which contain a limited amount of chemical and biological impurities. Total organic carbon (TOC) concentration is relatively high (>5 mg/l) in source water lakes.

¹ Process report on Jänneniemi WaterTreatment Plant (FCG Finnish Consulting Group Ltd)

The dimensioning of groundwater recharge plants is determined by the quality of the infiltrated surface water (TOC less than 2 mg/l) and the flow characteristics that are dependent on the aquifer particle size distribution and hydraulic conductivity and by retention time long enough for the soil aquifer treatment process to reach water quality suitable for potable use.²

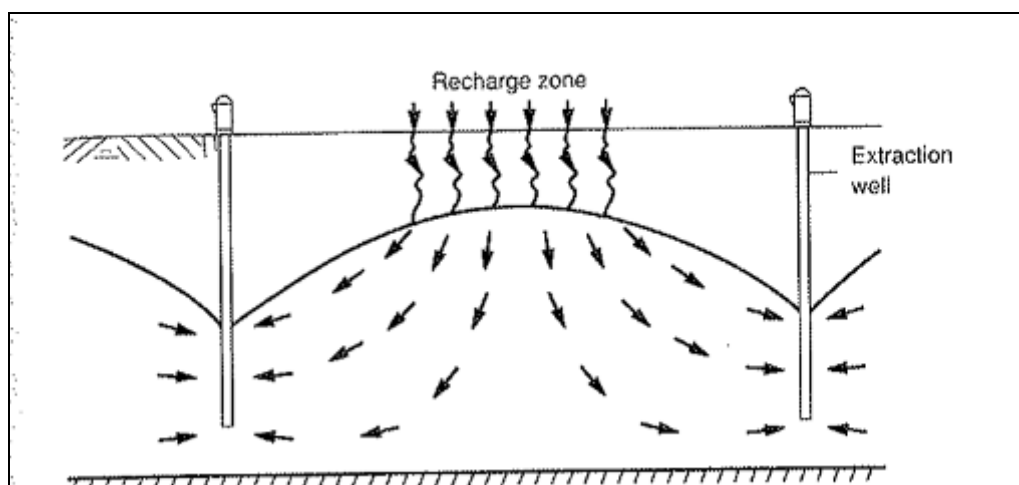


Fig. 4.1: Schematic sketch of Nordic groundwater recharge by surface spreading.

4.1.2 Lessons Learnt from Finland Recovery Schemes

1. The Finland experience has shown that the infiltrated water needs to be of good quality since groundwater recharge is sensitive to changes in source water quality. For example, It was necessary to improve pre-treatment before infiltration in Kuivala-Utti Recharge Plant (Case 3) because of the existence of organic and inorganic impurities in raw water. Otherwise, rapid clogging of infiltration basins would significantly harm the infiltration process.
2. Another lesson learnt from Kuivala-Utti recharge plant relates to groundwater quality. Infiltration and flow of water through sand/gravel ground might dissolve additional inorganic material (such as fluoride) from aquifer layers to groundwater. This might deteriorate water quality and prevent the use of water for drinking purpose.

4.2 Relevant Experience from Reuse Schemes

4.2.1 Israeli Recovery-Reused Experience

² Artificial recharge in Finland through basin and sprinkling infiltration (ISMAR) 2005

Israel National Water Company, Mekorot, returns over 50% of the reclaimed effluent to agriculture through 12 reclamation plants (Fig. 4.2). Annual consumption of treated wastewater reused for agriculture was 340 million m³ in 2005.³

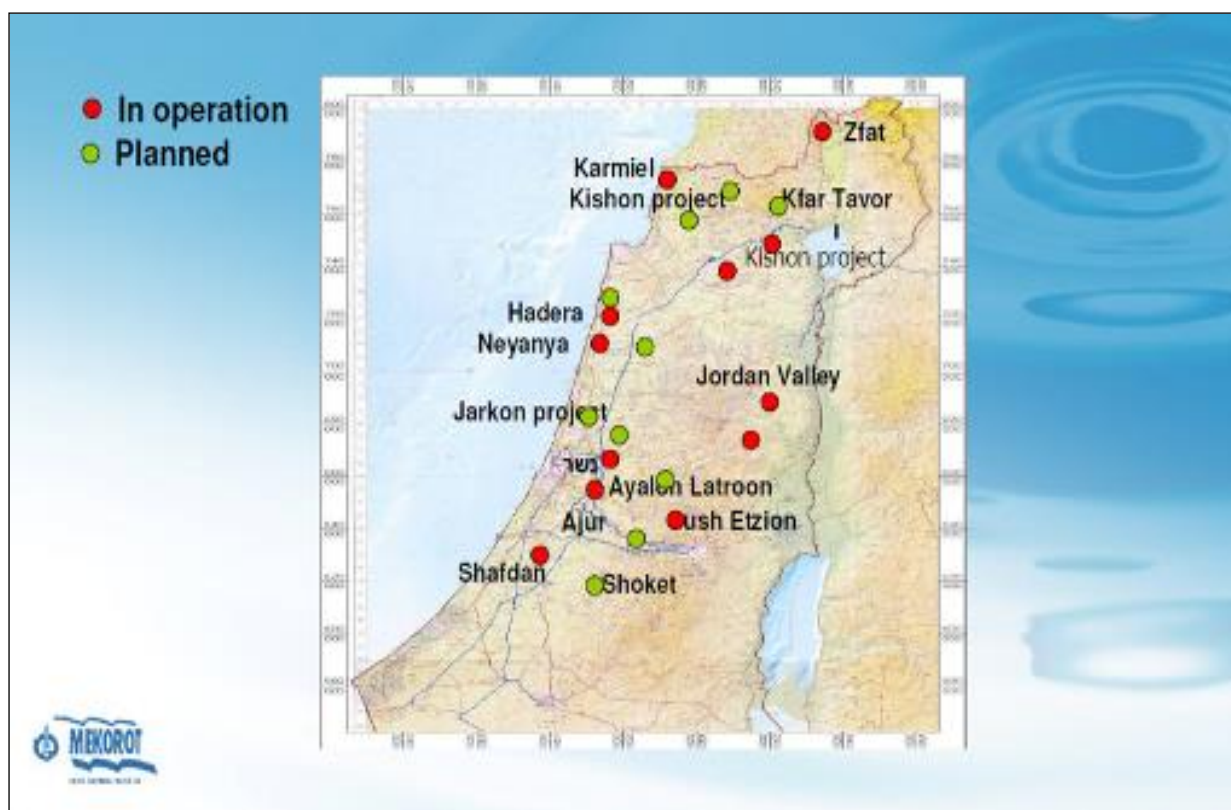


Fig. 4.2: Effluent reuse projects in Mekorot.

Israel is the one of the leading countries in recycling of wastewater with over 70% of effluent reused, followed by Spain and other semi-arid countries utilizing 12% or less of discharged wastewater (Fig. 4.3).

³ Wastewater Treatment and Effluent Reuse (Mekorot) May 2006

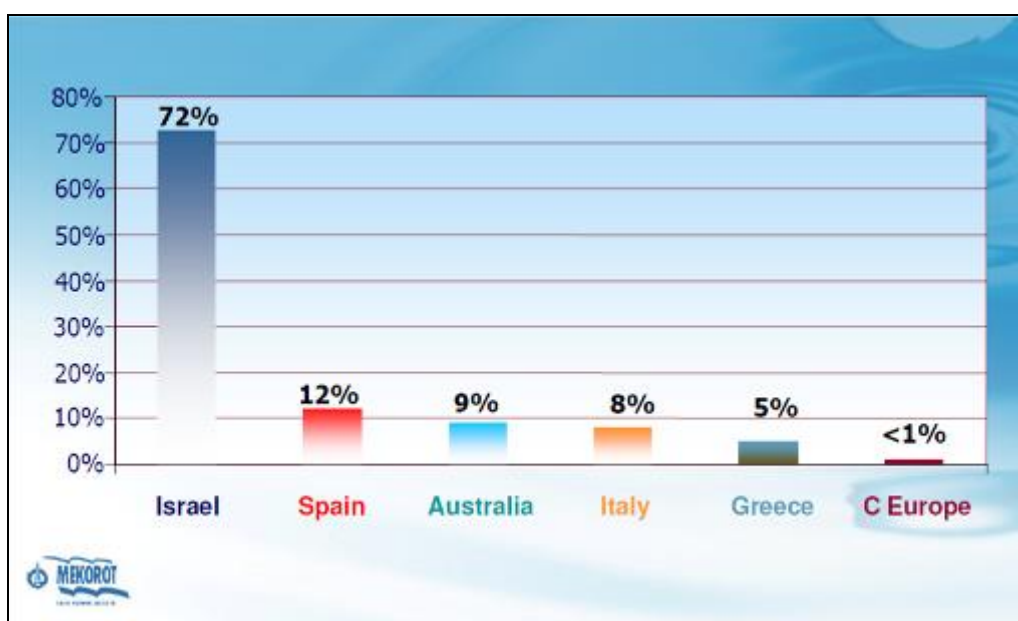


Fig. 4.3: Reused effluent in Israel in relation with other countries.

Groundwater recharge with municipal effluent has been investigated in Dan Region Reclamation project since 1977. The recharge-reclamation process is based on intermittent flooding and drying of the spreading basins, controlled passage of the effluent through the unsaturated zone and part of the aquifer, and subsequent pumping of the reclaimed water by means of production wells surrounding the recharge area. A separate zone is thus created within the regional aquifer, which is located beneath the recharge basins and is separated hydrologically from the rest of the aquifer by the well ring. This zone is dedicated to treatment and seasonal storage of the effluents (SAT, Soil Aquifer Treatment).

Shafdan WWTP treats about 130 million m³ of wastewater annually. Treated wastewater is presently infiltrated through 6 spreading basins (Fig. 4.4) and reclaimed through 150 recovery wells. The reclaimed water is used for unrestricted agricultural irrigation in the southern part of Israel conveyed through a 60 km long transmission line, the Third Line to Negev.

The following observations can be made until year 2008:

- Spreading basin area totals 105 ha;
- Hydraulic load varies between 64 and 242 m per year in spreading basins (0.2–0.7 m/d);
- Recharge regime includes 1–2 days of flooding (inflow) and 2–6 days of drying (no flow);
- The quality of WWTP effluent is high (BOD 5 mg/l and P 1.4 mg/l) because of the mechanical-biological treatment used since 1987;
- The SAT system provides additional treatment where over 70% reduction was obtained for: suspended solids, BOD, COD, DOC, UV absorbance, ammonia, Kjeldahl nitrogen, nitrate, phosphorus, phenol and copper. Moderate removal (50–70%) was obtained for filtered nitrogen, fluoride, cyanide and mineral oil;⁴

⁴ Groundwater Recharge with Municipal Effluent (Mekorot) 2008

- Average retention time is 9 months, but it was noticed that 3 months is enough for the SAT process to take place;
- Uptake from recovery wells is steady 36000 m³/d. During winter time not all wells are operated. Part of the storage required to balance the fluctuations in irrigation demand is provided by 3 large seasonal reservoirs at the Negev end of the system.

The recharge operation is accompanied by a comprehensive monitoring program, which includes both hydrological and water quality monitoring. Chlorine serves as a tracer of the movement of the recharged effluent in the aquifer (background level in the regional aquifer is low).



Fig. 4.4: Spreading basins drying and flooding at recharge site Soreq 2 of the Shafdan plant.

4.2.2 American Experience

According to one available national survey on municipal wastewater reclamation and reuse projects, there were 536 waste water reuse projects in the United States in 1975. Only 11 of these were in the category of groundwater recharge. The projects reused 2.570 million m³/d of wastewater (Table 4.2). Most of the wastewater reuse sites are located in the arid and semiarid western and southwestern states, including Arizona, California, Colorado, and South Carolina.⁵

⁵ Wastewater Engineering; Treatment; Disposal and Reuse (Metcalf&Eddy) 1991

Table 4.2: Municipal effluent reuse projects in the United States (US Dept. of Interior).

Category	Number of projects	Reclaimed water
Irrigation total	470	1.590 million m ³ /d
- Agriculture	150	
- Landscape	60	
- Not defined	260	
Industrial total	29	0.814 million m ³ /d
- Process		
- Cooling		
- Boiler feed		
Groundwater recharge	11	0.129 million m ³ /d
Other (Recreation etc.)	26	0.037 million m ³ /d
<hr/>		
Total	536	2.570 million m³/d

The first major groundwater recharge project with reclaimed wastewater was undertaken at Whittier Narrows in Los Angeles County, California, in the beginning of 1962. *After extensive health effects evaluation of more than 20 years of records, researchers concluded that there was no measurable adverse impact on the groundwater in the area.*

4.2.3 South African Experience (Atlantis 2002)⁶

The Atlantis (50 km from Cape Town) aquifer has supplied high quality groundwater for nearly 35 years in South Africa. Effluent recharge started in 1982. Low salinity storm water and treated wastewater has been led to an infiltration basin for recharge during the dry season, in order to increase the quantity and quality of groundwater.

In 1997 production capacity of Atlantis artificial recharge was 1.5–2 million m³ per year. The strategy is to improve the final quality of the water by soil-aquifer treatment and make 3 million m³ additional water each year. Geological cross section through the Atlantis aquifer is shown in Fig. 4.5.

The positive experience of Atlantis project is that good quality potable water could be produced from limited amount of natural water resources by utilizing the storm water and treated wastewater for groundwater recharge.

The negative experience is that the risk of contamination of groundwater has increased. Therefore, monitoring of infiltrated storm water and treated wastewater has been intensified. Moreover, sensitive groundwater recharge and catchment areas are partly protected.

Some operational problems in Atlantis have caused iron biofouling of wellfields. Also dissolved organic carbon concentrations have increased in some monitoring boreholes.

⁶ Atlantis Aquifer, Status Report (CSIR) July 2002

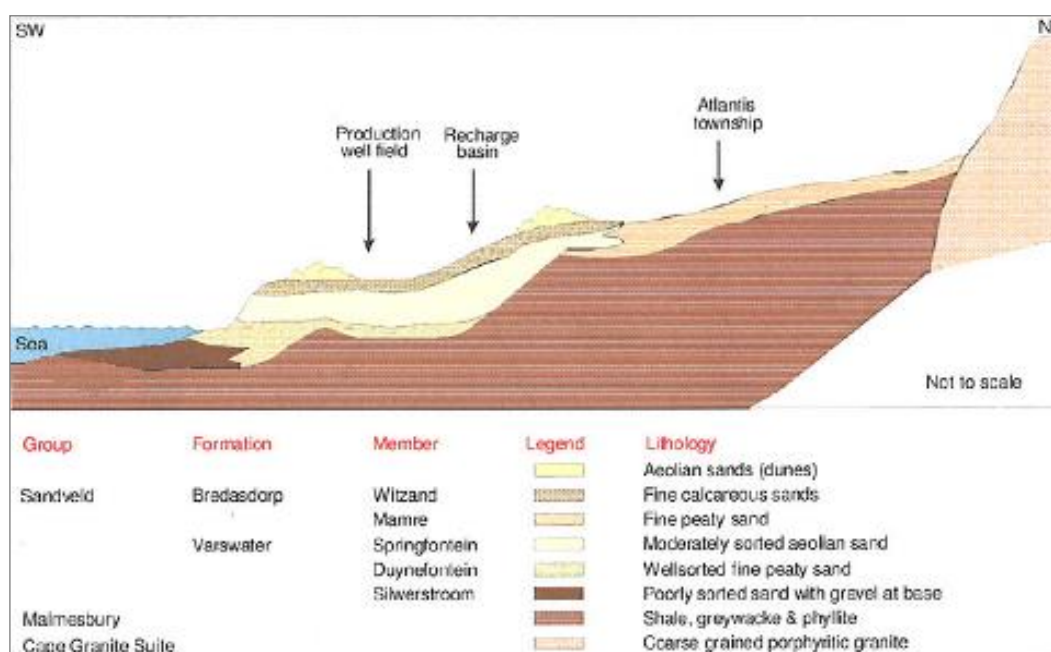


Fig. 4 5: Geological cross section through the Atlantis aquifer.

4.2.4 Lessons Learnt from Recovery and Reuse Schemes

- 1- Depending on the level of treatment, it is possible to reuse the treated wastewater in unrestricted irrigation. This is particularly true if SAT is used as the case of Shaf Dan in Israel.
- 2- Comprehensive monitoring program should be applied not only to observe the quality of groundwater table but also the water at the end user. This is to detect any signs of clogging to irrigation network.
- 3- Care should be applied in using of surface seasonal reservoirs which could degrade the quality of water. Solutions of such problems include using of covered reservoirs, growing of different types of fish, etc.
- 4- The existence of sand particles in pumped water from production wells may present a serious problem for the wells, especially the impellers, water networks and irrigation sprinklers or drippers. Gravel pack filter should be carefully designed and placed around the well screen.
- 5- The use of infiltration basins could require excessive land of appropriate hydrogeological characteristic. In countries of limited land availability infiltration may be replaced by other means of treatment such as membrane system.

4.3 Comparison with Palestinian Recovery and Reuse Project

The NGEST project being implemented in the northern of Gaza Strip has benefited from the experiences of other countries. The following are main project advantageous characteristics concerning recovery and reuse schemes:

1. Wastewater is treated to high quality before infiltrated to groundwater table.
2. SAT process is used thus recovered water can be used for unrestricted irrigation.

3. Groundwater is used as seasonal reservoir with appropriate retention time. Surface reservoirs are not used which eliminates various problems associated with surface reservoirs.
4. Groundwater modeling techniques is used in the study and design of the recovery system.
5. Closed water tanks are used as balancing tanks to hand daily variations.
6. Comprehensive monitoring system will be implemented to observe groundwater table, recovered water, and water quality at end user.

5 Hydrologic Assessment and Modeling

5.1 Introduction

The current chapter will be the base of design the recovery wells which will drilled around the infiltration basin that mainly capture the infiltrated water and pump to the irrigation scheme. The design of the recovery wells will first consider the assessment of the hydrological information obtained from the current hydrological investigations in addition to the past investigations in the project. Second the model will be used to verify the location of the wells with regards to their ability to capture the infiltrated water. The wells will be designed to mitigate against environmental, social, and public health impacts to the nearby communities caused by delays in implementation of the construction of the new wastewater treatment plant.

Therefore, the design of the recovery wells will consider several scenarios where the quality and the quantity of infiltrated treated wastewater are included. EA study was carried out based on a groundwater model of the northern area where the infiltration basin is included. In this study, the influence of transferring wastewater from the lake to the infiltration basin in the project area was studied. It was noticed that infiltrated water can be captured by a number of recovery wells surrounding the downstream of the infiltration basin. In this study an indicative number of recovery wells to mitigate the impact of worst case scenario were identified.

The objectives of this chapter are:

1. Review and analysis the data collected from the hydrological investigations carried out in the current project and past projects in the area. Preliminary, the configuration of the wells are in terms of number, location, discharge (pumping rate), depth of the well, operating hours will be determined as an output of the hydrogeological data assessment (Hydrogeological Approach). The modeling approach will be used also to verify and determine the exact spatial distribution of the recovery wells.
2. Review the existing models prepared in the previous modeling works. Evaluate the hydraulic studies, conceptual model, model boundaries, mesh size (grid spacing), calibration results, and simulations. Discuss and point out the accuracy, reliability, and shortcomings of the model.
3. Update the existing numerical model according to updated conceptual model and data. Make test runs and evaluate results (groundwater head, flow pattern, and mass balance). After this, calibrate the model. This means that model parameters are adjusted so that calculated steady state groundwater head is practically the same as the flow pattern and water level according to the field measurements. Evaluate the hydraulic water budget (mass balance) also.
4. When calibration results (steady state and transient) are satisfactory, perform simulation runs to study alternative well locations and to find the best solution for well field optimization and artificial recharge. Study capture zones with particle tracking method and pollution transport analysis (MT3D). Present accuracy of simulations. Utilize the final model to explore the optimal configuration of recovery wells taking into consideration any land ownership issues.
5. In addition, based on the hydrogeological assessment and model results, design a monitoring program which includes the number and location of monitoring wells,

temporal frequency of the monitoring activities, and the parameters to be monitored. This sub-activity makes use of the existing monitoring program.

5.2 Design of Wells based on Hydrogeological Assessment

5.2.1 General Geology of the Coastal Aquifer

Gaza aquifer is part of the regional coastal aquifer which lies along the southeastern edge of the Mediterranean Sea and extends from the foothills of Mt. Carmel southward to Gaza and northern Sinai. It is composed of Pliocene-Pleistocene age calcareous sandstone, unconsolidated sands, and layers of clays. In the Gaza Strip, the aquifer extends about 15–20 km inland, where it overlies Eocene age chalks and limestone or the Miocene-Pliocene age Saqiye Group. The Saqiye Group is a 400–1000 meter thick sequence of marls, marine shales, and claystones. Approximately 10 to 15 km inland from the coast, the Saqiye Group pinches out, and the coastal aquifer rests directly on Eocene chalks and clastic sediments of Neogene age. Fig. 5.1 presents a generalized geological cross-section of the coastal aquifer.

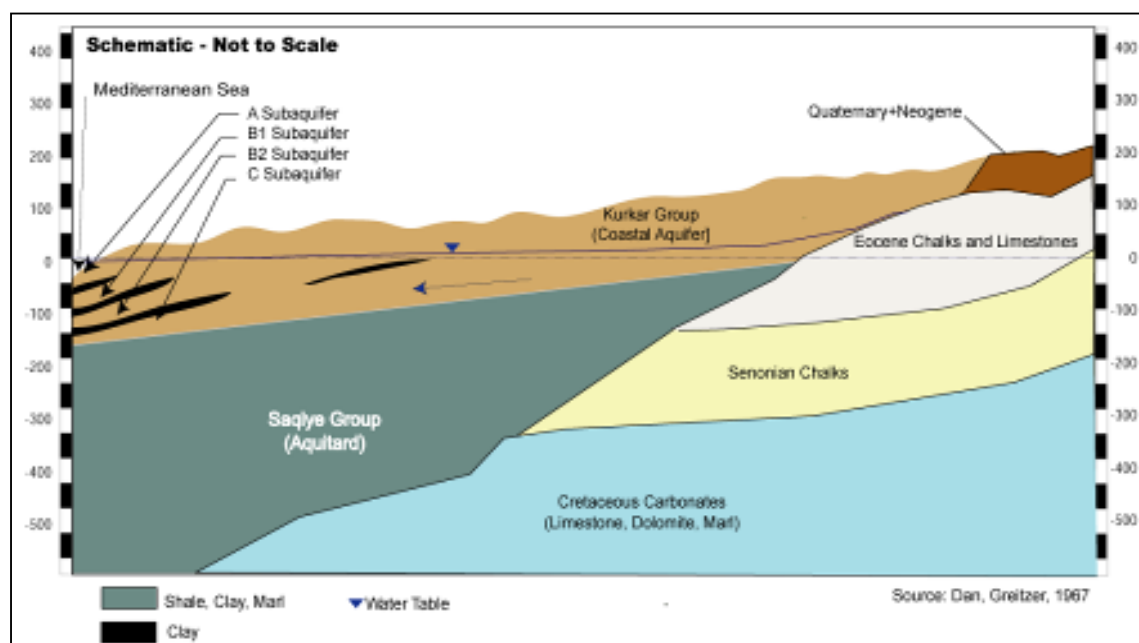


Fig. 5.1: Generalized geological cross section of the coastal aquifer.

Near the coast in the Gaza Strip, clay layers subdivide the coastal aquifer into four separate sub-aquifers (Fig. 5.1). They extend inland about 2 to 5 km, depending on location and depth. Further east, the marine clays pinch out and the coastal aquifer can be regarded as one hydro-geological unit.

Within the Gaza Strip, the thickness of the Kurkar Group increases from east to west, and ranges from about 70 m near the Gaza border to approximately 200 m the coast. Low permeable layers are found in the Kurkar group. These layers are more predominant closer to the coast.

5.2.2 Geotechnical Assessment of Recovery Wells Area

Although the greater part of the Gaza strip has a topsoil of stiff clay, the hydrogeological investigations at the site (August 2002 in well DB4 and tests carried out under the current project May-2010) clearly indicates that the aquifer is an unconfined, phreatic aquifer. The water level data indicated that the flow direction is from east to west as shown in Fig. 5.2. The recovery wells should be located around the infiltration basin and should be concentrated in the west and the north direction of the basin as indicated in Fig. 5.2, where two geological cross sections were made. Fig. 5.3 shows a cross section that connects BH1, BH5 and BH2 and Fig. 5.4 shows a geological cross section for BH2, BH3 and BH4.

The sections indicate that clayey and silty layers have been found below the ground surface. The layers are found both in the unsaturated and saturated zone. Below these layers sand to coarse sand (Kurkar) and they are also found below the depth of the water level. The depth of water level ranged between 45 to 65 m below the ground surface. The average depths of sand and kurkar layers in the range between -37 m and -70 from the ground surface. The recovery wells screens are located inside the sand and kurkar layers. The screen length will be designed based on the pumping rate of each well which is expected to be 170 m³/hr as shown in section 5.2.3. The exact depth of the screen will be also considering the drawdown of the groundwater level. Figs. 5.3 and 5.4 show the location of the screen of the wells which will be ranged between -50 to -84 below the ground surface.

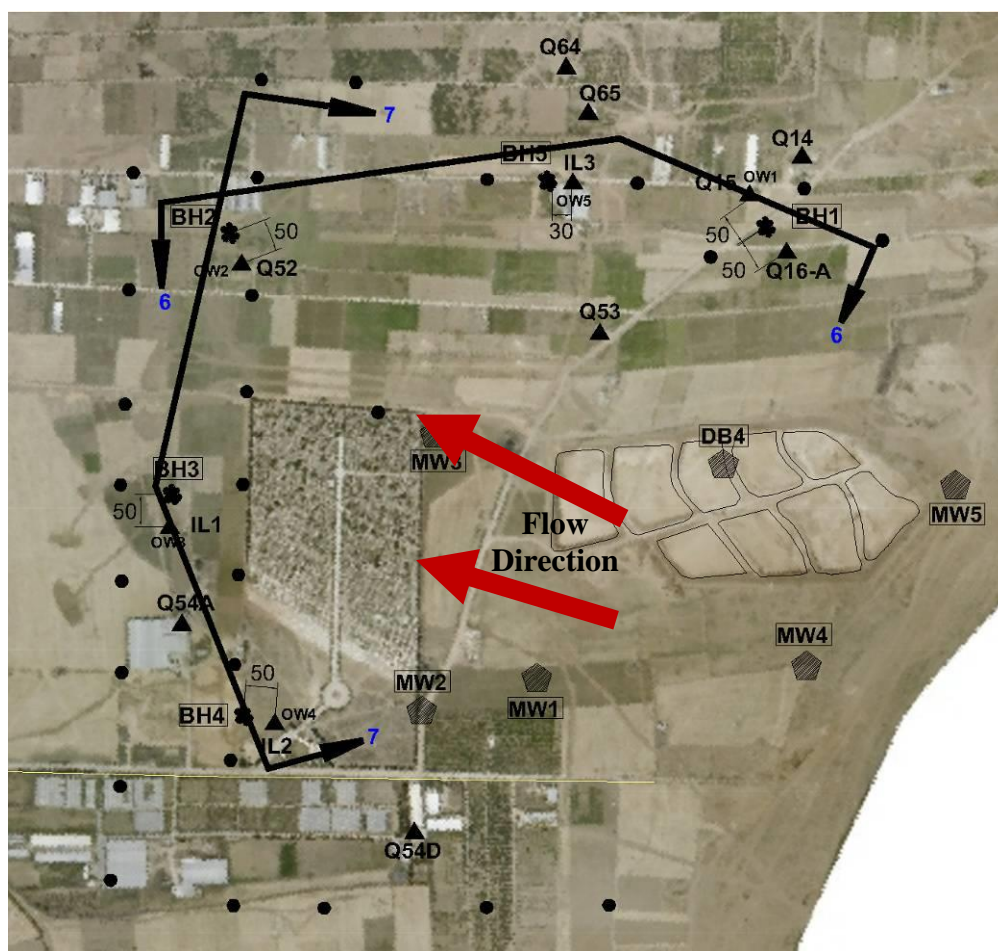


Fig. 5.2 Flow direction and location Hydrogeological cross sections

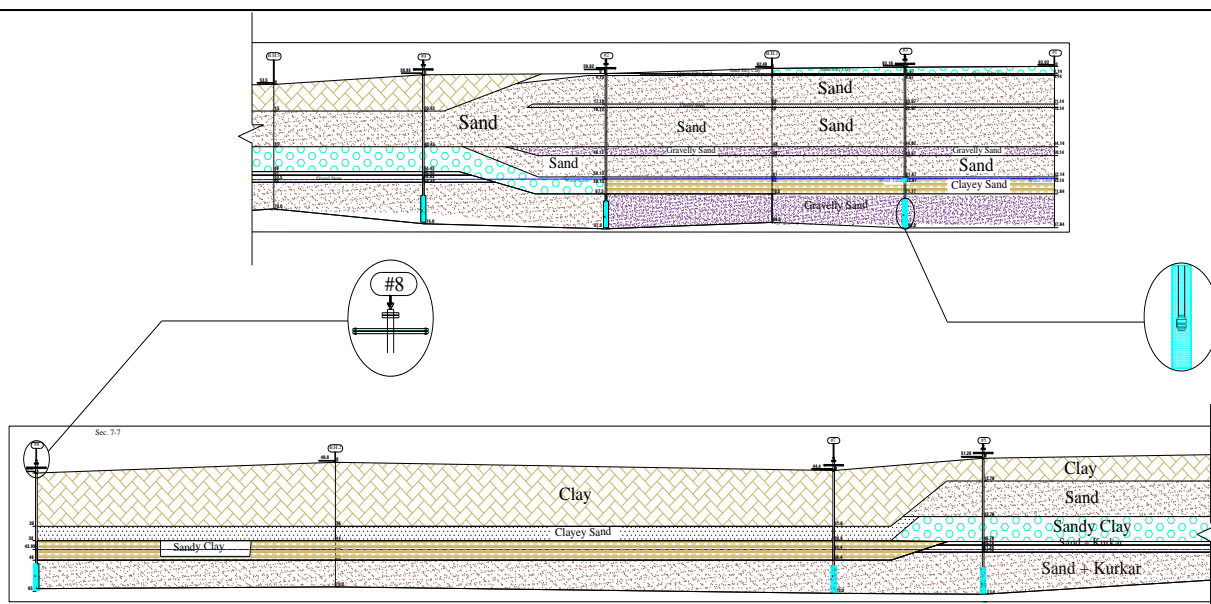


Fig. 5.3: Hydrogeological cross section of BH1, BH5, and BH2.....

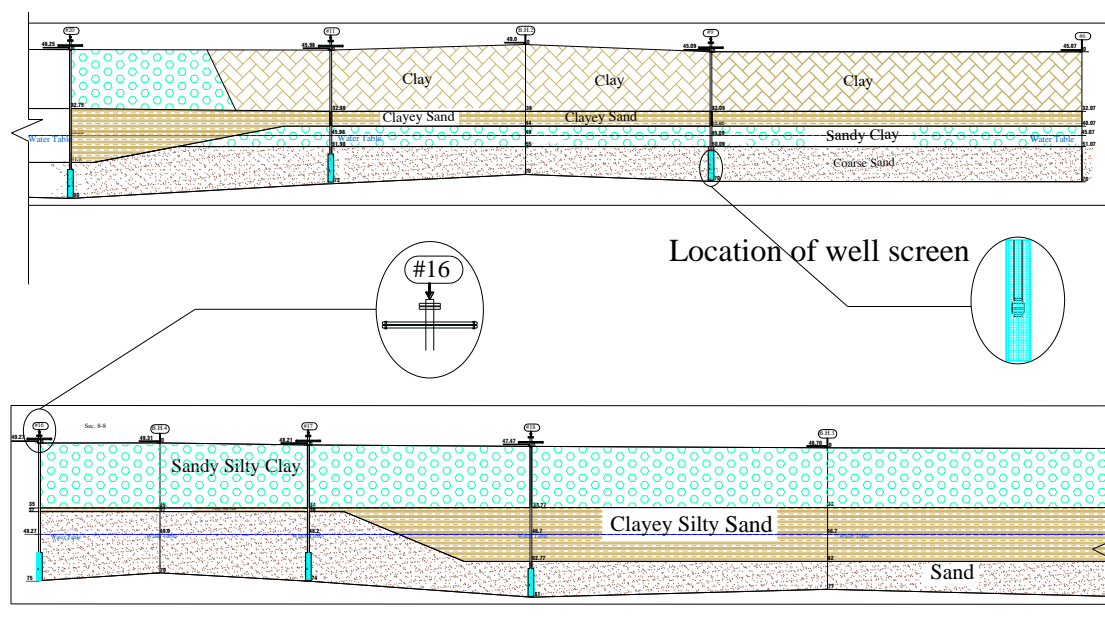


Fig. 5.4: Hydrogeological cross section of BH2, BH3, and BH4.....

5.2.3 Number of Recovery Wells

Based on the agricultural report in *Appendix 1*, three scenarios have been made for both the infiltration water quantity and consequently the quantity of water to be recovered. Table 5.1 shows the daily recovered water quantities which should be extracted by the recovery wells and pumped through the irrigation networks. The values presented in Table 5.1 considered the values of the water requirements for irrigation multiplied by 1.15 (15% extra) to account for non-farming activities and potential climatic change. In this concern three phases can be distinguished, in Phase one the average amount of water to be recovered will be **16500 m³/d** where the minimum value will be in October and the maximum value will be in June with an amount of **21140 m³/d**. For Phase three the maximum value to be recovered will be in June with an amount of **50885 m³/d**. An alternate recommendation for maintaining the pump operating at its design capacity throughout the year, pumping hours should be adjusted monthly, with maximum 12 hours operating in the month of June (Table 5.2) and 8 hours for the month of October. Based on Table 5.1 and Table 5.2 the required number of recovery wells is computed for the month of June to be 25 wells as shown in Table 5.3. For the other months, a similar approach has been used to calculate the number of wells for each month. Table 5.3 shows the number of wells for each month for the three scenarios. For computing the number of recovery wells and based on the pumping test, **170 m³/hr** as a constant pumping rate have been used. The maximum number of recovery wells will be **25 wells** which will be fully operated in June under scenario III.

Table 5.1: Daily recovered water (m³/day)

Scenario	I	II	III
Recovered	16500m ³	23100m ³	39160m ³
Jan.	14799	20718	33081
Feb.	15091	21127	35816
Mar.	14010	19614	34995
Apr.	13997	19595	34204
May	19634	27488	46622
June	21140	29596	50885
July	20262	28367	50136
Aug.	21269	29777	49073
Sept.	17476	24466	40290
Oct.	12459	17443	30187
Nov.	13147	18406	31484
Dec.	14716	20602	33146
Average	16500	23100	39160

Table 5.2: Pumping hours and rates for scenarios I, II and III, for the June peak month.

	Scenario I	Scenario II	Scenario III
Working hours	12	12	12
Pumping Rate	1761 m ³ /hr	2466 m ³ /hr	4240 m ³ /hr

Table 5.3: Required number of Wells

Month	Scenario I	Scenario II	Scenario III
Jan.	9	12	19
Feb.	9	12	21
Mar.	8	12	21
Apr.	8	12	20
May	10	13	23
June	10	15	25
July	10	14	25
Aug.	10	15	24
Sept.	9	12	20
Oct.	7	10	18
Nov.	8	11	19
Dec.	9	12	20
Average	9	12	21

5.2.4 Aquifer Soil Sampling and Testing

During the drilling, permeability tests, soil sampling, and water sampling were carried out. Samples were subsequently analyzed in the laboratory as noticed in Section 3. Water quality analysis were carried out to check the quality of groundwater in the recovery well areas and verify the groundwater quality model.

5.2.5 Current Water Quality

This section is based on the aquifer water quality baseline survey and the water sampling of the aquifer close to the basin carried out during the current project. The baseline water quality survey was carried out for the Implementation of Environmental Management Plan (EMP) for the North Gaza Emergency Sewage Treatment Project (NGEST). EMP was part of the project EA study. The scope of EMP was to: sampling and analysis of groundwater in 25 wells (13 in existing wastewater treatment plant (BLWWTP) area and 12 in NGEST area), and sampling and analysis of BLWWTP effluent for major ions, microbiological parameters and heavy metals. The sampling and analysis were conducted in October to November 2007, January to February 2008, May 2008 and July 2008.

The current sampling program was carried out in two rounds which concentrated on the wells around the basin as shown in Fig. 5.5. Field visits to obtain water samples from agriculture wells, and wells within infiltration basins were carried out on 12th and 13th December 2009 for wells no. Q/53, Q/15, Q/54B, DB4. The measured parameters in the first round sampling were: PH, E.C., T.D.S., Nitrate, Chloride, and Calcium.

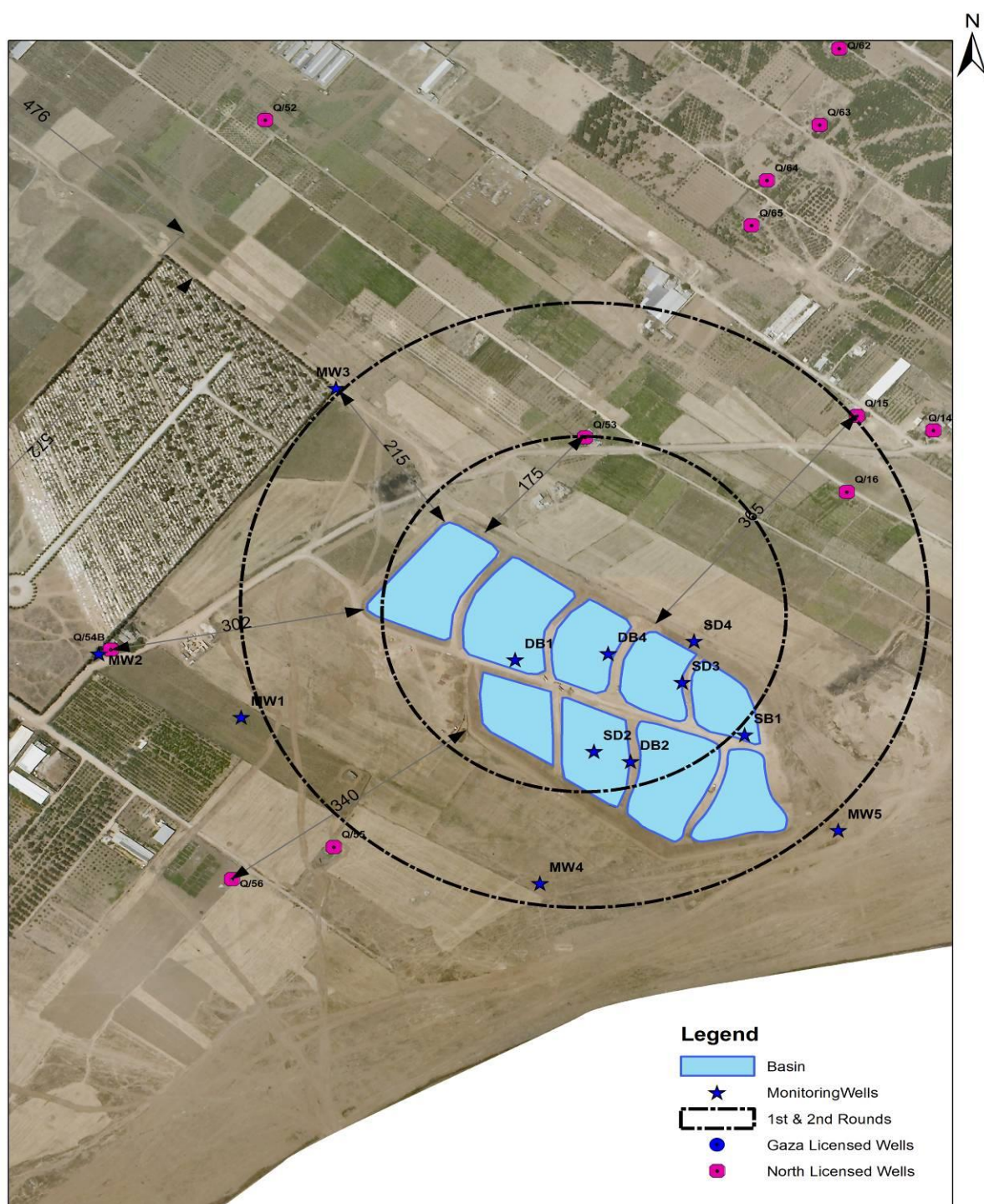


Fig. 5.5: Location of Sampled Wells in the First and the Second Round.

Based on the first round of water well sampling, a second round was started on 12–13 January 2010 to investigate the quality of wells in wider distance and other directions from the basins. The second round sampling wells and the tested parameters are shown in Table 5.4. Some of the wells which were proposed to be sampled were replaced by other wells, for example, MW1 was not ready for sampling and Q55 was destroyed. These wells were replaced by MW2 and Q56. MW5 was not sampled since it is very close to the border. The location of the sampled wells is shown in Fig. 5.5.

Table 5.4: The sampling wells and parameters in the second round

Well No. Tested Parameters	Q/15, Q/53, Q/56 and DB4	MW2 MW3	Basin
W1	x	x	
PH	x	x	
EC	x	x	
TDS	x	x	
BOD	x	x	x
COD	x	x	x
NO ₃	x	x	x
T.N	x	x	x
Cl	x	x	x
Detergent	x		x
F.C	x		x
Helmithes eggs	x		x

Fig. 5.6 shows the results of the Cl in the wells close to the infiltration basin. The chloride concentration ranges between 400 to 600 mg/l in the wells surrounding the infiltration basins up to the end of year 2009. The trend of the chloride concentration seems to be steady since year 2007 in some wells and it getting closer to effluent value in other wells. The effluent value is lower than the base concentration in the area due the transboundary flow from the eastern border.

Fig. 5.7 shows results of the Nitrate in the wells close to the infiltration basin. The nitrate concentration for the same period ranges between 25 to 35 mg/l which is less than in WHO standards. Fig. 5.7 also shows that there is a drop of the nitrate concentration in the aquifer surrounding the basin; this may due to the reduction of agricultural practice in the area in the last two years due to the insecurity situation in the area surrounding the infiltration basin.

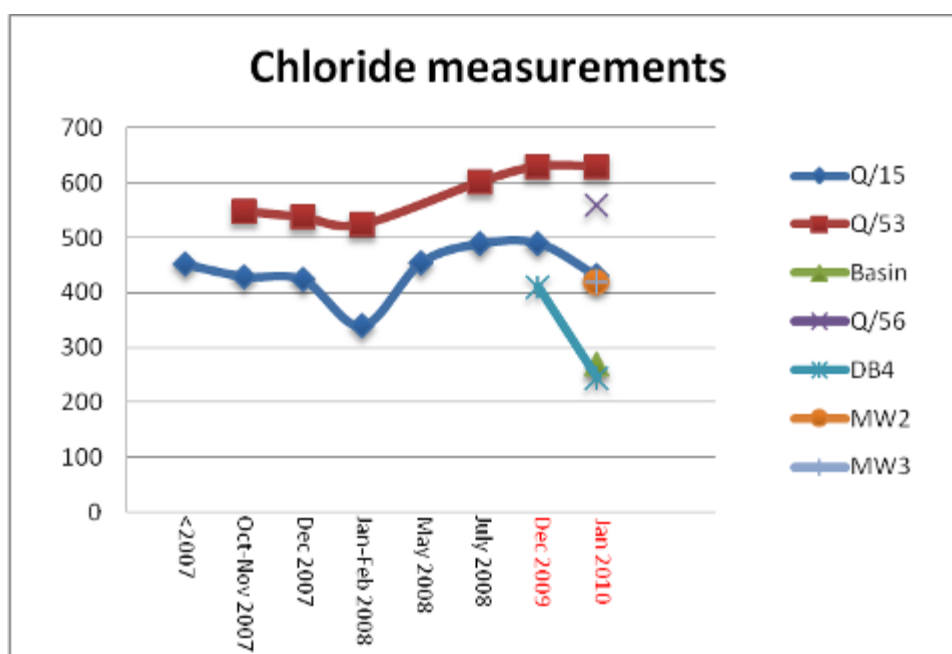


Fig. 5.6: Cl Concentration in the Wells Close to the Infiltration Basins.

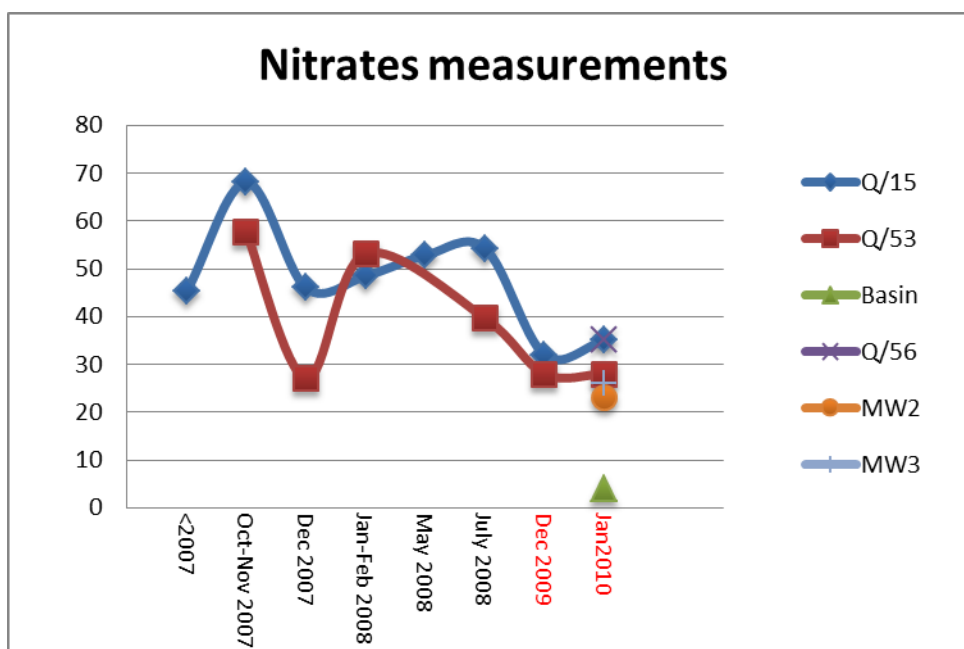


Fig. 5.7: NO₃ Concentration in the Wells Close to the Infiltration Basins.

Table 5.5 shows the results of the water quality parameters tested in the wells close to the basin in 12–13 January, 2010. The TDS ranges between 1000 to 1800 mg/l. The BOD of the water in the infiltration basin was measured as 58 mg/l, in the DB4 well of a value equals to 25 mg/l where in well Q/56 is 15 mg/l. The TKN in the basin was 40 mg/l where in the rest of wells was around zero. The detergent was measured in the basin as 4.6 MBAS where in the wells the value ranges from 0.01 to 0.05 which is below the WHO standards. The results of the bacteriological pollution tests for well Q15 show the existence of Helminth eggs and the F.C. is over the

recommended standards. The reason of such FC existence may be attributed to the direct pollution through the well pipe from animal wastes such as birds. This justification could be applied also to the case of MW3.

Table 5.5: The groundwater quality in a selected samples taken on the site.

	T.D.S (mg/l)	BOD (mg/l)	COD (mg/l)	TKN (mg/l)	Detergent (MBAS)	F.C. (cfu/100ml)	Helminthes	HCO ₃ ⁻ (mg/l)	Ca ²⁺ (mg/l)	Mg ²⁺ (mg/l)	K ⁺ (mg/l)	Na ⁺ (mg/l)
Q15	1425	0	10	1	0.034	TMC	2	440	15	39	2	410
Q53	1705	0	10	0	0.011	0	0	350	29	49	3	440
Basin	----	58	125	40	4.6	6x10⁵	3	630	66	60	20	320
Q56	1730	15	45	0.9	0.018	5	0	410	17.5	45	3	425
DB4	1100	25	70	0.9	0.05	0	0	425	6	52	3	260
MW2	1400	0	10	0	-----	-----	-----	440	15	28	2	360
MW3	1375	0	10	0.9	-----	TMC	-----	425	19	27	3	360

TMC: Too Many to Count

During the pumping test, water quality samples were taken from the five boreholes (BH1, BH2, BH3, BH4, and BH5). Table 5.6 shows the results of the laboratory analysis. The samples were taken in the period between June to August 2010. Table 5.6 shows that the level of NO₃ is between 30 to 44 mg/l which is the same range as in the wells sampled in January 2010. In addition, the Cl ranges between 410 mg/l to 730 mg/l which is greater than the values in the wells sampled in January 2010. For example, the Cl concentration in Q15 was 420 mg/l whereas the Cl concentration in BH1 which is very close to Q15 (50 m distance) is 585 mg/l. In addition the Cl concentration was 550 mg/l in Q56 in January 2010 whereas it increased to 674 mg/l in BH5 which is very close to Q56. The increase of Cl is also shown between MW2 and BH4 which are close. The difference in Cl concentration between the two wells is around 300 mg/l. This could be an indication of the influence of the infiltrated water to the groundwater in the project area which starts to make dilution of the chloride concentration in MW2 area since the effluent chloride concentration was 250 mg/l. In addition, it seems that the effect of the effluent doesn't reach BH4 area yet. .

Table 5.6: Water Analysis Tests Results of the Boreholes

Item	Results				
	BH1	BH2	BH3	BH4	BH5
pH	7.31	7.05	7.40	7.31	7.16
EC $\mu\text{S}/\text{cm}$	2190	2220	1850	2620	2300
TDS (mg/l)	1315	1330	1110	1570	1380
TSS (mg/l)	150	1.2	4	2	2
BOD as O_2 (mg/l)	0	0	0	0	0
COD as O_2 (mg/l)	0	0	0	0	0
DO (mg/l)	2.75	2.3	2.7	2.5	3.65
NO_3^- (mg/l)	36	44	30	44	43
TKN as $\text{NH}_3\text{-N}$ (mg/l)	0	0	0	0	0
$\text{NH}_4\text{-N}$ (mg/l)	0	0	0	0	0
Cl^- (mg/l)	585	697	410	730	674
$\text{PO}_4\text{-P}$ (mg/l)	0	0	0	0	0
NO_2^- (mg/l)	0	0	0	0	0

5.3 Assessment of the Existing Groundwater Model

5.3.1 Groundwater Modeling

5.3.1.1 Software Description

Previously, there have been three modelling exercises related to the study area. Groundwater model of the Northern area for NGEST project under EA 2006 study. Visual Modflow (VMF) version 4.2 and its integrated modules were chosen. VMF is based on the finite-difference code MODFLOW (Harbaug & McDonald 1988) and contains four integrated modules: MODFLOW – Groundwater flow model, ZONE BUDGET – Water balance within user defined zones, MODPATH – Particle tracing and MT3D (Model Tracking 3D) – Substance or solute transport.

For the current work, the model used by EA is considered as the base of further modeling activities in this project. Details of modeling procedures carried out are presented in *Appendix 1*. Therefore, the following conceptual model is considered valid, however, the modeling procedures were repeated for the seek of further calibration and verification of the model by input of new data from year 2004 until 2008.

5.3.2 Conceptual Model

5.3.2.1 Model domain and boundaries

The model domain and boundary is used as in EA modeling efforts which is presented in *Appendix 1*. Fig. 5.8 shows the selected model domain as part of the coastal aquifer.

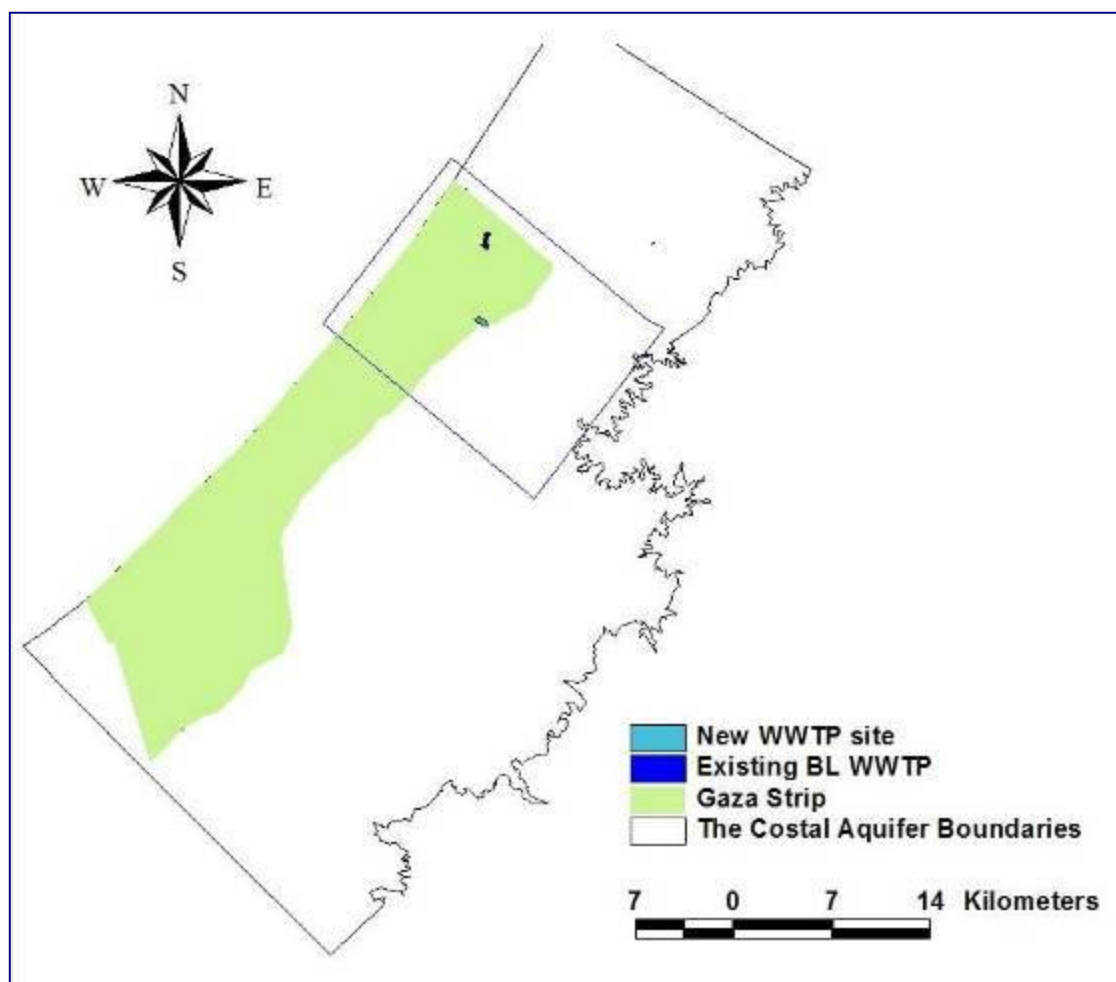


Fig. 5.8: Model domain and boundaries

The model domain is divided into a horizontal grid with cell size 50x50 m at the BLWWTP site and 20x20 m at the new NGWWTP site and the cell size then increases gradually towards the model boundaries (Fig. 5.9). The same model boundaries in previous model was used as follows

- East: General Head Boundary
- West: Constant Head Boundary
- North and South: No Flow Boundary.

The lower boundary of the model consisted of Sakiye's surface. This has been adopted based on the regional DYN model consideration and the results from geophysical investigations, and borehole investigation at the site (DB4, BH1 to BH5).

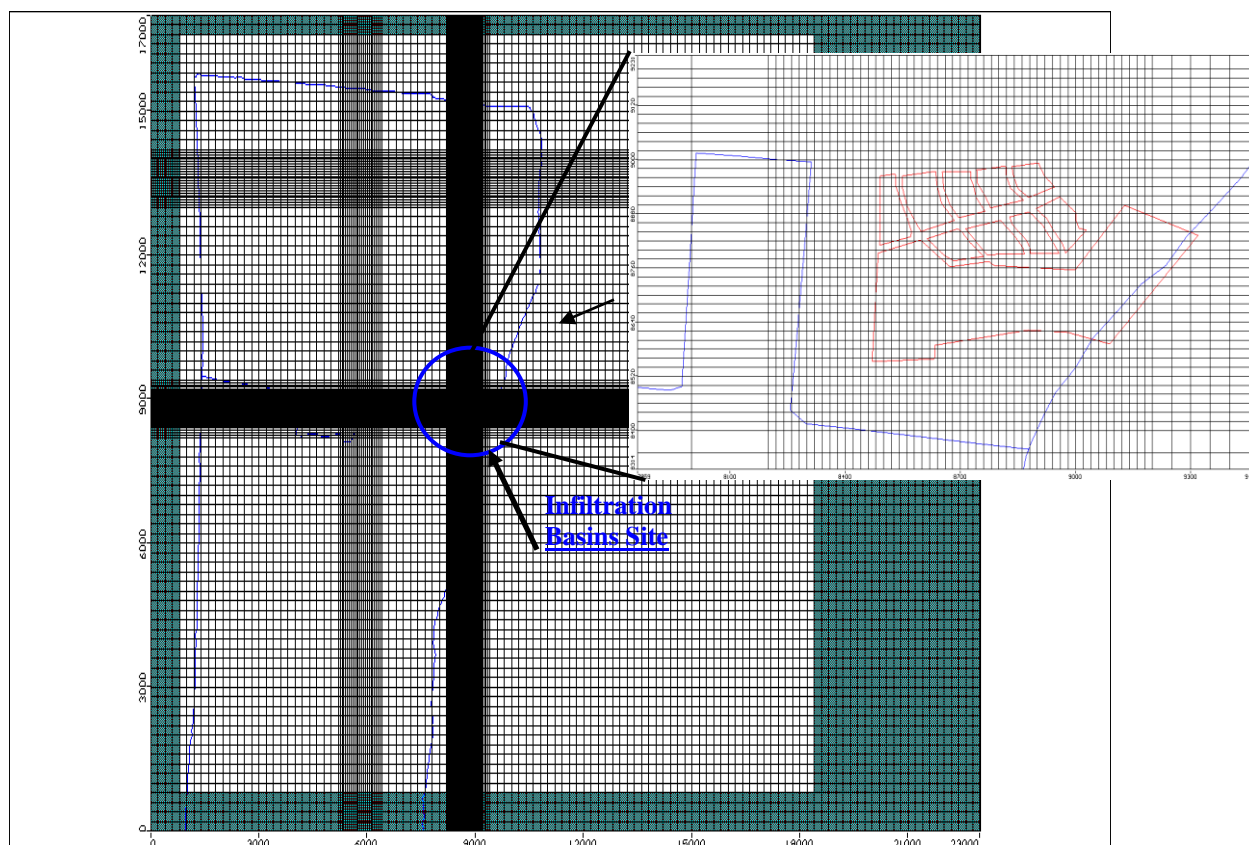


Fig. 5.9: Model grid and the grid of the Infiltration Basin

The arial continuity and hydraulic permeability of these layers do, however, not lead up to the conclusion that the aquifer is divided into several hydraulically separate subaquifers. Instead, the one aquifer approach is supported.

This report used the same assumption used previously by CAMP model final report which indicated that the top clay layer extends up to 2 km inland. The second clay layer extends up to 1.5 km and the third deep clay layer extends up to 3.5 km inland. The average depths of those layers are -60, -100, and -130 to -60, respectively. As motioned above, all wells, screens are located above the deep clay layer.

Recharge Components which were used in EA modeling procedures were used in the current modeling. The components included the recharge from rain, irrigation, unpiped wastewater, piped wastewater and water supply network losses. The GIS recharge rate distribution of the 2003–2004 hydrologic in winter and summer is shown in *Appendix 1*. It was seen from Fig. 5.10, the difference in rainfall in the period between 2000 to 2003 and the period 2004 to 2007 is small. The difference was adjusted in the abstraction of agricultural wells which was reduced comparing the values in the EA model

Based on the GIS recharge grid distribution, 24 recharge zones (Fig. 5.11) were considered for the MODFLOW input. Each zone carries different values based on annual and seasonal recharge values.

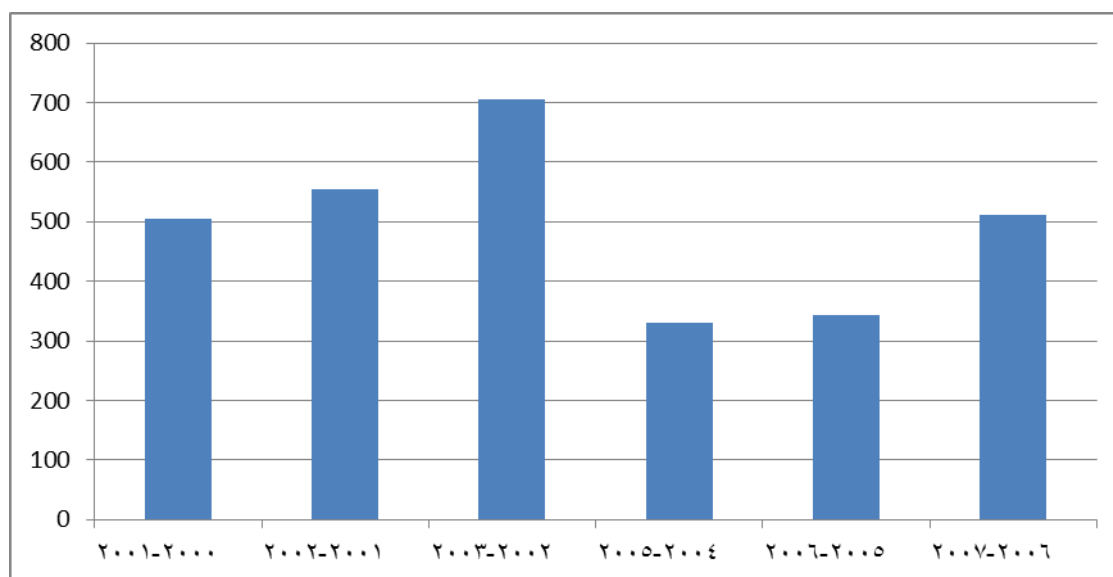


Fig. 5.10: Average Rainfall in the Northern Area between 2000 to 2007.

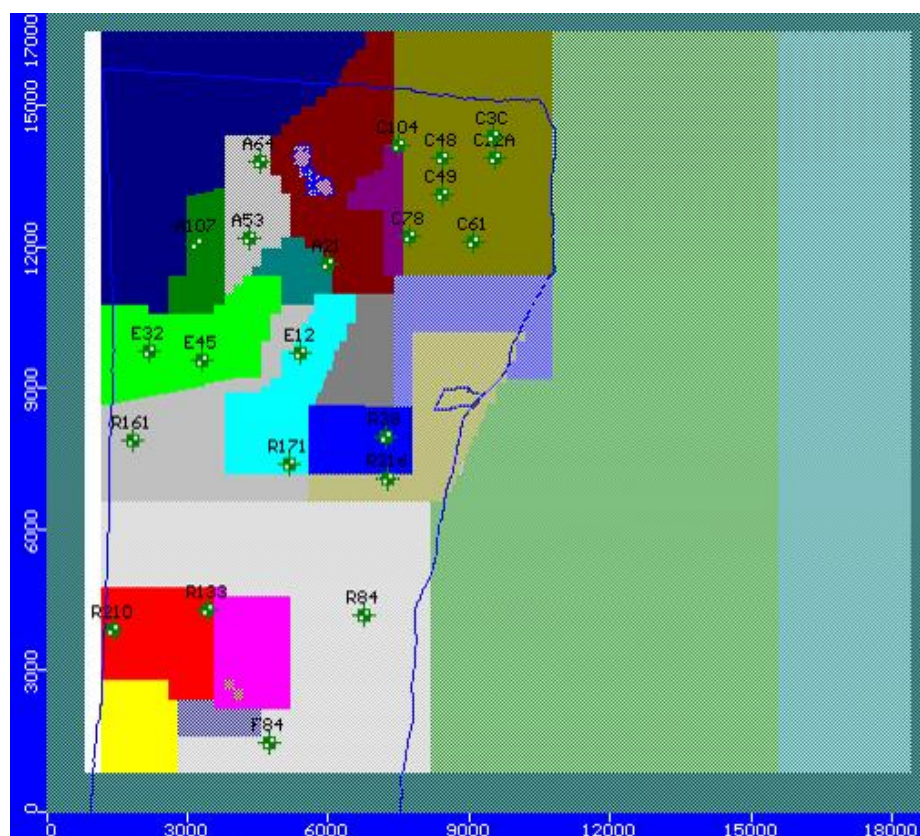


Fig. 5.11: Head observation wells and MODFLOW recharge zones

5.3.2.2 Abstraction Components

Within the model area, 1185 agricultural wells have been defined and parameterized with a given average discharge based on available data (data from PWA and Ministry of Agriculture). In addition 62 domestic wells were also recorded based on data from Coastal Municipality Water Utility (CMWU). The abstraction from domestic wells is recorded monthly. Table 5.7 shows the yearly abstraction from domestic wells whereas the average daily abstraction of each municipal well in the northern area are shown in *Appendix 2*. Very limited data is available about agricultural wells abstraction. In most of agricultural wells the abstraction rates were estimated based on information from Ministry of Agriculture about irrigated areas, crop patterns, and crop water requirements.

The 26 wells which were selected as head observation wells for the model regional calibration in the previous model is still used in the following calibration procedures. The selection was based on the availability of good hydrograph for these wells. More details are presented in the calibration section.

Table 5.7: Yearly Abstraction Municipal Wells

	Yearly Total Abstraction (m ³ /year)				
	2004	2005	2006	2007	2008
48 wells	40,297,825	38,292,697	42,208,089	42,260,758	42,147,0975

Source: CMWU and PWA Data, 2008

5.4 Groundwater Model Update

5.4.1 Aquifer Properties

The default model parameters were set based on the calibrated parameters from the EA study (EA, 2006). The new pumping tests carried out in May 2010 indicated the following parameters based on which the model was recalibrated. K_{xy} has been initially set with a general value of 60 m/day in the proximity of the proposed infiltration site and 35 m/d else where in the model domain. Little adjustments have been made thereafter in specific zones in connection to model calibration. In the same way, K_z has been set for 3 m/day, S_y for 0.15, S_s for 0.00002 m⁻¹, n_e for 0.25, and total porosity for 0.35.

5.4.2 Steady State Model Calibration

Data from year 2004 to 2006 was used for the steady state calibration at year 2004. The recharge and abstraction rates were estimated based on 2004–2006 data, as specified in section 4.3.2. The modeled waterlevel was then calibrated based on year 2004 water level records for 26 observation wells distributed throughout the model domain (Fig. 5.11).

Fig. 5.13 shows the steady state water level contour map for the year 2004. In general, the modeled contour map shows a good agreement with the previous modeling results of the EA study for the same period. However, the area that covers the -3 m drop in the EA model is larger

than in the case of the current model. This is due to that the current model used a real data of the abstraction wells whereas the previous model used estimated data.

Fig. 5.12 compares the modeled results with the observed water level values. Except for few wells close to the seashore and far away from the basin (e.g. R/161, R/210, and E/32, C3C), the modeled values shows 90% (Correlation Coefficient = 0.90) agreement with the observed value. In addition the model shows a good agreement in the wells close to the infiltration basin such as for well R84 the correlation is 0.98. This indicated that the model would perform very well within at least 6.0 kilometers radius from the infiltration site.

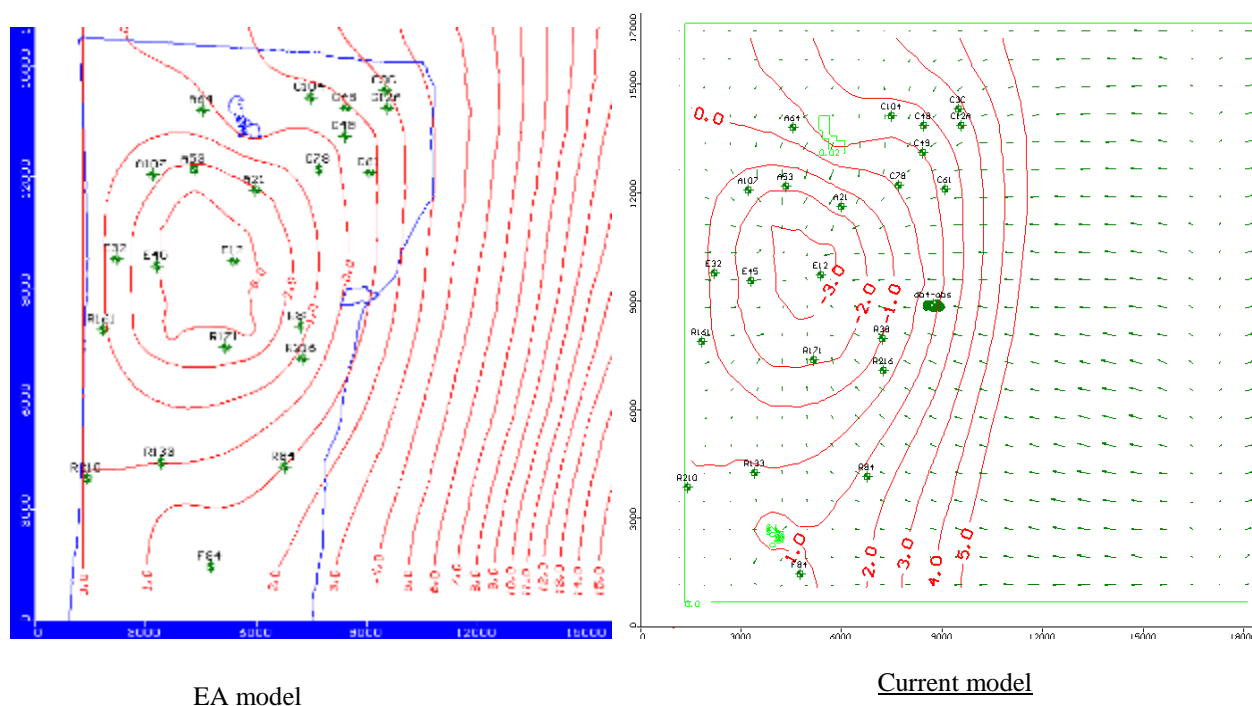


Fig. 5.12: Comparison between the steady State water level contours in EA study and the current model (year 2004)

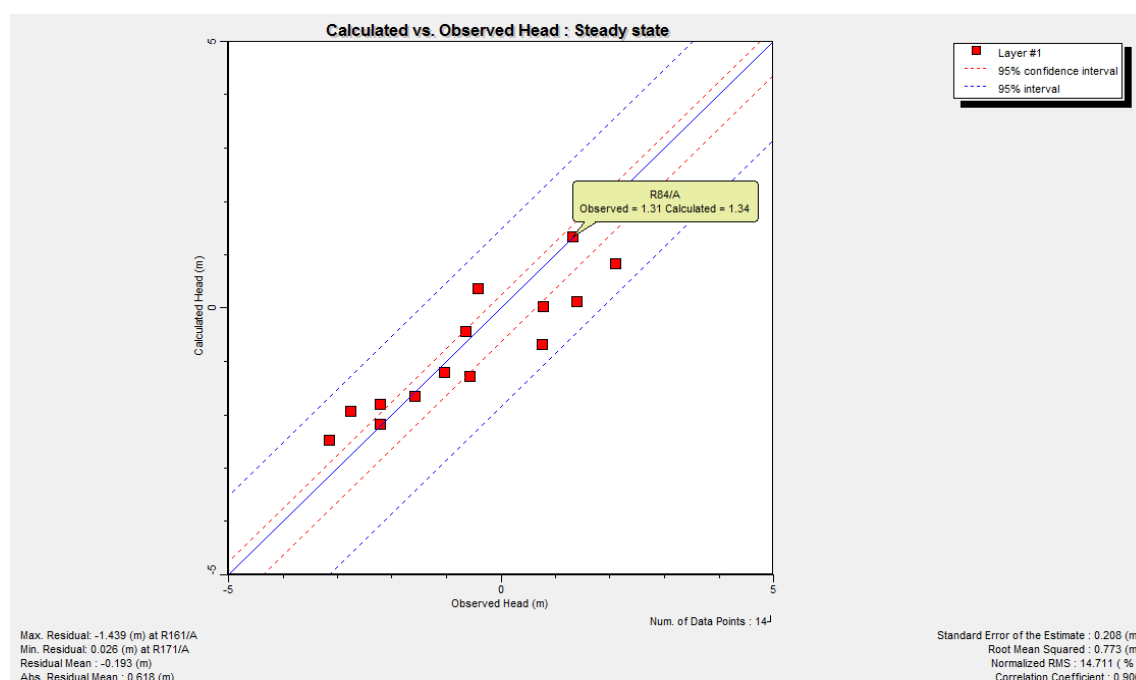
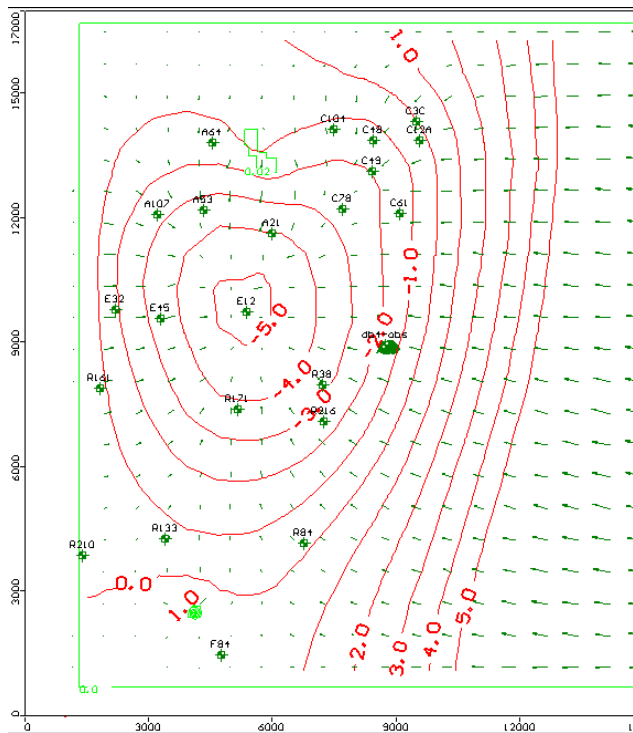


Fig. 5.13: Steady state calibration results

5.4.3 Transient Model Verification

Data from the period 2006–2008 was also used for the transient model calibration. The abstraction and recharge components were earlier discussed in section 4.3.2. The same graph of the distribution of recharge rate used in the steady state is used in the current transient model. Since the aquifer properties were set based on the CAMP DYN model and the model developed by SWECO INT, and the EA model, the calibration was mainly performed based on the change of the abstraction of the wells whereas the recharge rate is assumed to be the same as in year 2003 to 2004. It was seen from Fig. 5.10, the difference in rainfall in the period between 2000 to 2003 and the period 2004 to 2007 is small. The difference was adjusted in the abstraction of agricultural wells which was reduced comparing the values in the EA model.

The time step for the transient model was set daily. Fig. 5.14 shows the modeled groundwater level contours at the end of year 2007 and the observed water level in the same year.



(b): Modeled Groundwater level contours in year 2007

Fig. 5.14: (a) Observed and (b) Modeled Groundwater Level Contours in Year 2007

Figs. 5.15 and 5.16 show the observed versus modeled water level hydrograph for wells E/45 and A/53. Notice the summer and winter fluctuation of water level. Similar graphs are available for other wells in the model domain. The modeled water level showed good agreement with the observed water level both in the trend and in the value.

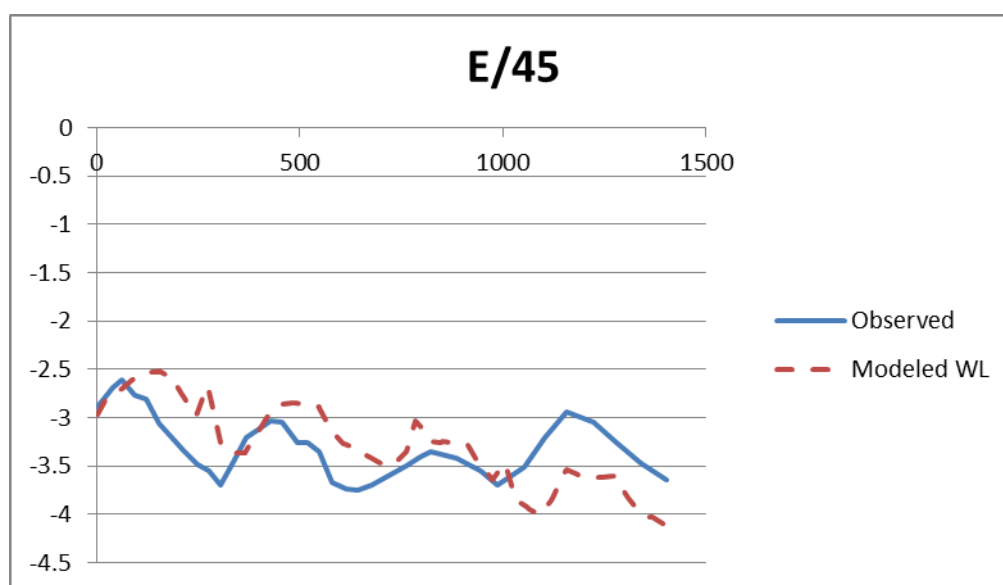


Fig. 5.15: Observed vs. modeled water level for well E/45. Category axis shows days since 2004

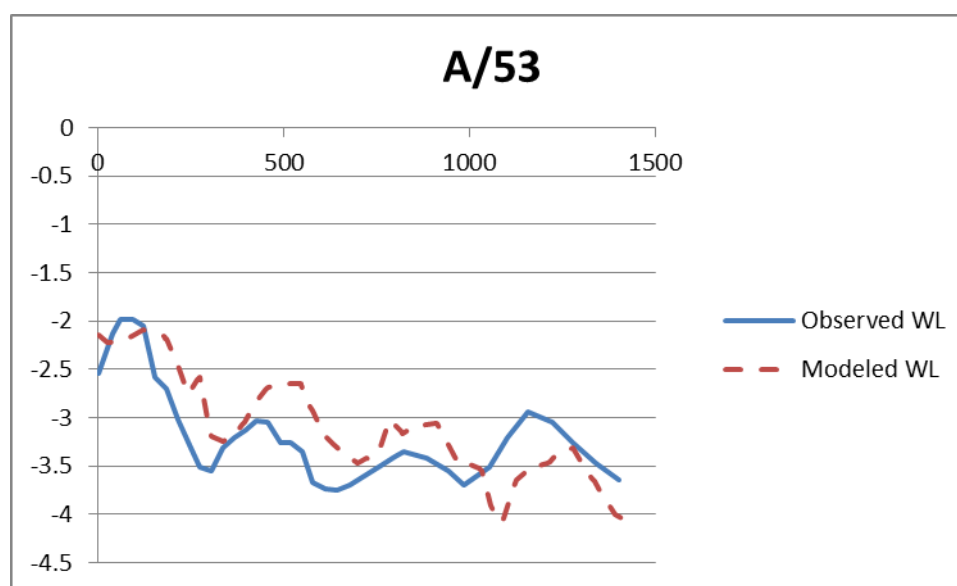


Fig. 5.16: Observed vs. modeled water level for well A/53

Fig. 5.17 shows the observed versus modeled water level hydrograph for well R/216. The well is close to the infiltration basin. There is a good correlation in the year 2004. The observed water level then started to get higher than the modeled water level through the end of verification period. This is typical in all the wells located in areas affected by Israeli incursion activities. In these areas, the trees have been uprooted and the abstracted water was less than the modeled abstraction. This was the same case in several agricultural wells located north and east of the Gaza Strip close to the borders with Israel when these wells are shut down.

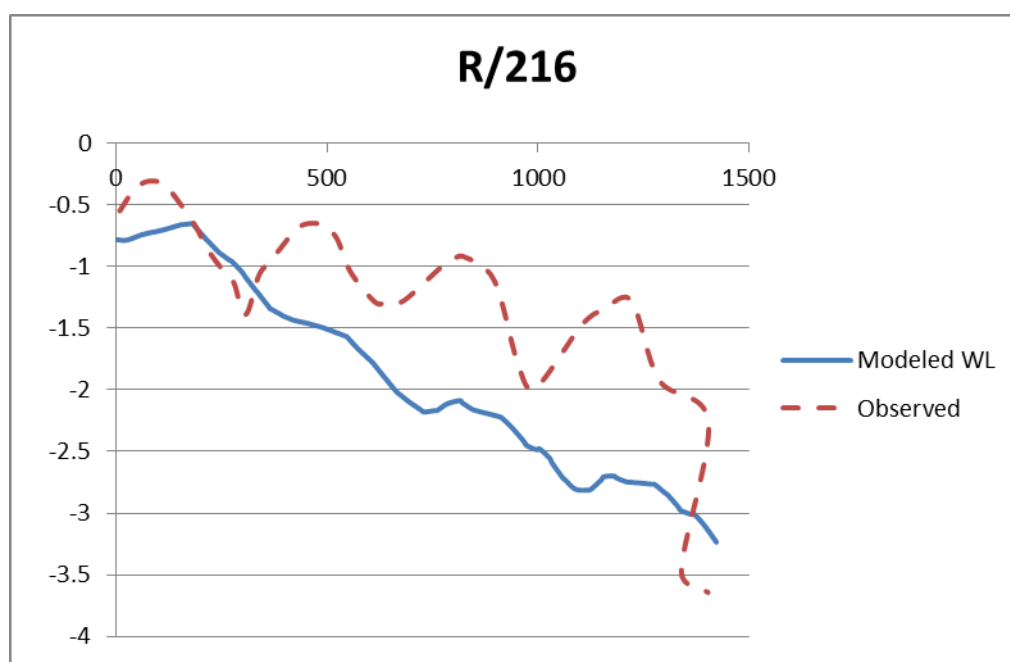


Fig. 5.17: Observed vs. modeled water level for well R/216

5.5 Recharge Scenarios

5.5.1 Flow Model

In order to study the impacts of the proposed infiltration basins on the aquifer, a prediction model, starting from year 2004 till the year 2025, was designed taking into consideration the calibration results of both the steady state and the transient models. Hence, the aquifer parameters are set as in the transient model. The long term seasonal recharge rate for summer and winter is considered to represent the seasonal differences in recharge through each year. The time step is chosen 1 day to study the impact of infiltration in greater details. Regarding the abstraction the following assumption are made:

- No change in agricultural abstraction due to the limitations in expanding agricultural activities (same assumption was made in CAMP Model).
- In each well, the municipal abstraction increases 3.0 % annually (same as the average population growth rate based on PCBS1997 predictions). Also there is an upper bound for the well abstraction which is equal to 170 m³/hour (Metcalf & Eddy, 2000).

5.5.2 Infiltration Scenarios

- The main objective of this concept is to develop a recovery plan that accounts for possible infiltrated wastewater, recovery scheme, and the demand for crops up to year 2025 or 2030 (based on consultant prediction) which could reach the identified capacities of treated wastewater (69,000 m³/d).
- Two phases based on generated wastewater quantity (35,600 m³/d and 69,000 m³/d) regardless of the target year (around 2012 and 2025) will be considered. Detail design of the recovery scheme is for 35,600 m³/d as infiltrated treated wastewater. Future extension

of infiltration basins and recovery scheme to accommodate 69,000 m³/d of treated wastewater will be suggested.

The main influencing factors for the design of the scheme are:

- i. The quality of the pumped wastewater (treated and partially treated) to the infiltration basin depends on the efficiency of the treatment in the existing BWWTP and/or the construction of the NGEST.
 - ii. The demand patterns of the crops.
- The whole amount of infiltrated water within one year should be recovered within one year where 10% extra should be abstracted to ensure the capturing of all infiltrated quantity.
 - The optimal recovery scheme for the 35,600 m³/d that will be implemented should satisfy these requirements by considering the following several relevant scenarios.

5.5.3 Scenarios for Infiltration and Recovery Scheme

The scenarios are classified into two major scenarios that considered the construction of NGEST or not. The sub-scenarios considered the quality of the wastewater which determines the quantity of wastewater which should be pumped to the infiltration basin.

5.5.3.1 SC1: NGEST is not constructed

SC1.1: if the quality of the pumped wastewater from the existing BWWTP was not improved nor worsen (BOD and SS are between 70 to 100 mg/l), then the allowable quantity of water to be infiltrated will be 15,000 m³/d up to year 2025. This scenario could be the pessimistic one.

SC1.2: if the quality of the pumped wastewater from the existing BWWTP is improved (BOD and SS are between 40 to 70 mg/l) by upgrading the BWWTP by year 2013. Three years were assumed necessary to reach a decision to improve the quality of BWWTP and then to implement the improving requirements. The allowable quantity of water to be infiltrated will be 15,000 m³/d with current quality (BOD and SS are between 70 to 100 mg/l) up to 2013. Then, 21,000 m³/d with improved quality (BOD and SS are between 40 to 70 mg/l) from year 2013 up to year 2025. The logic of the 21,000 is to keep the same amount of BOD load in the water that results from the 15,000 (i.e. $15000 \times 100 / 70 = 21,000$). This means that the quality of the groundwater due to infiltration of 15,000 will remain the same even if the quantity of infiltrated water was increased to 21,000.

SC1.3: If the quality of wastewater from the existing BWWTP was worsened, then no infiltration to groundwater should be allowed. This is to protect the quality of groundwater from further deterioration. It should be mentioned that recent groundwater quality tests showed negative influence of pumping partially treated water with current quality. So the 70 to 100 mg/l limit is justified and should not be changed. Therefore this Sc1.3 is discussed to justify its elimination and will not be considered by the consultant.

5.5.3.2 SC2: NGEST is Constructed

SC2.1: if NGEST is implemented in year 2014 (planned year for operation), the quality of the pumped wastewater will be good (BOD and SS are 10 mg/l). Then the allowable quantity of partially treated wastewater to be infiltrated with current quality will be 15,000 m³/d up to year 2013 and 35,600 of fully treated wastewater from year 2014 to year 2025.

SC2.2: if there will be a delay in construction of the treatment plant to about 5 years, i.e. NGEST is implemented in year 2020, the quality of the pumped wastewater will then be good (BOD and SS are 10 mg/l). In this scenario, the allowable quantity of partially treated wastewater to be infiltrated will be 15,000 m³/d up to year 2019 and 35,600 of fully treated wastewater from year 2020 to year 2025.

SC2.3: if NGEST is implemented in year 2020, and the existing BWWTP is upgraded in year 2013, then the quality of the pumped wastewater will be different (BOD and SS are 10 mg/l after 2020 and 40-70 mg/l between 2014 to 2019). In this scenario, the allowable quantity of partially treated wastewater with current quality to be infiltrated will be 15,000 m³/d up to year 2013. Then 21000 with improved quality from year 2014 to year 2019 and 35,600 with fully treated wastewater from year 2020 to year 2025.

5.5.3.3 Other Scenarios

Other scenarios could be identified to represent conditions between the above mentioned time intervals. However, by inspection it can be concluded that such scenarios will not influence the optimal recovery scheme since the aforementioned considered scenarios cover the extreme conditions. Thus conditions in between will be covered the considered scenarios. Therefore, no other scenarios will be considered at this stage. Other conditions may be later verified using the optimal infiltration and recovery scheme to be implemented.

Table 5.8 shows the prediction of generated wastewater in the northern governorate. It can be seen that the CEP prediction is quite similar to the prediction carried out by SWECO study which was the basis of the NGEST and the infiltration basin design. Table 5.8 to Table 5.13 include the predictions of generated, infiltrated, and recovered partially or fully treated wastewater relevant to identified scenarios.

Table 5.8: Wastewater generation in the northern area between 2010 to 2025

Year	Average (M&W Study) (m ³ /d)	Max (M&W Study) (m ³ /d)	Average SWICO Study (m ³ /d)	CEP Prediction (m ³ /d)
2010	24,556	28,242	31,049	32,455
2011	28,023	32,230	33,325	33,719
2012	31,490	36,218	35,600	34,982
2013	34,957	40,206	38,867	36,246
2014	38,424	44,194	42,134	37,510
2015	41,893	48,182	45,403	44,315
2020	53,140	61,117	55,368	61,688
2025	58,257	67,003	65,336	64,412

Table 5.9: NGEST is not constructed (SC 1.1: BOD and SS is between 70-100 mg/l)

Year	Infiltration Quantity (m ³ /d)	Conc. Of BOD and SS (mg/l)	Recovered Water (m ³ /d)
2010-2030	15,000	70-100	16500

Table 5.10: NGEST is not constructed (SC 1.2: BOD and SS is between 40 to 70 mg/l)

Year	Infiltration Quantity (m ³ /d)	Conc. Of BOD and SS (mg/l)	Recovered Water (m ³ /d)
2010-2012	15,000	70-100	16500
2013-2030	21,000	40-70	23100

Table 5.11: NGEST is constructed (SC 2.1: NGEST will be operated in 2015)

Year	Infiltration Quantity (m ³ /d)	Conc. Of BOD and SS (mg/l)	Recovered Water (m ³ /d)
2010-2014	15,000	70-100	16500
2015-2030	35,600	10	39160

Table 5.12: NGEST is Constructed (SC 2.2: NGEST will be operated in 2020)

Year	Infiltration Quantity (m ³ /d)	Conc. Of BOD and SS (mg/l)	Recovered Water (m ³ /d)
2010-2020	15,000	70-100	16500
2021-2030	35,600	10	39160

Table 5.13: NGEST is constructed (SC 2.3: NGEST in 2020 with improvement in the existing BWTTP)

Year	Infiltration Quantity (m ³ /d)	Conc. Of BOD and SS (mg/l)	Recovered Water (m ³ /d)
2010-2020	15,000	70-100	16500
2021-2030	35,600	10	39160

5.6 Flow Model Results

5.6.1 Steady State Risk Simulation

If recovery is not implemented, the results show that the rising water table reaches steady state conditions after approximately 10 years for the infiltration rates for the minimum recharge (15,000 m³/d), and approximately 30 years for the maximum recharge (35,600 m³/d) respectively. The effect on groundwater levels caused by infiltration is best described in Figs. 5.18 and 5.19 after steady state conditions were reached. The simulations show that the groundwater level under the infiltration area will rise to about 2 m for the minimum recharge quantity while it will rise to 10 m if the maximum recharge quantity increase. The results show that at the long run hundreds of wells will be affected by the resulted water mound. The resulted water mound will extend about 2,300 m in the west and north-west direction from the infiltration basin in the case of maximum quantity.

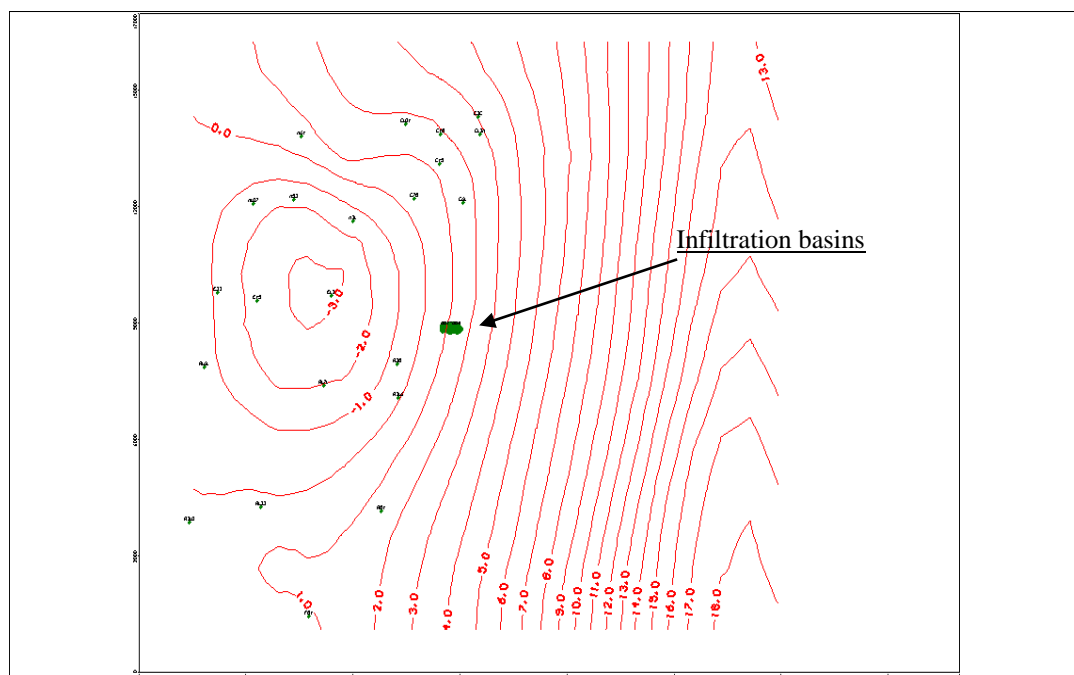


Fig. 5.18: Steady state water level contours in case of infiltration 15,000 m³/d

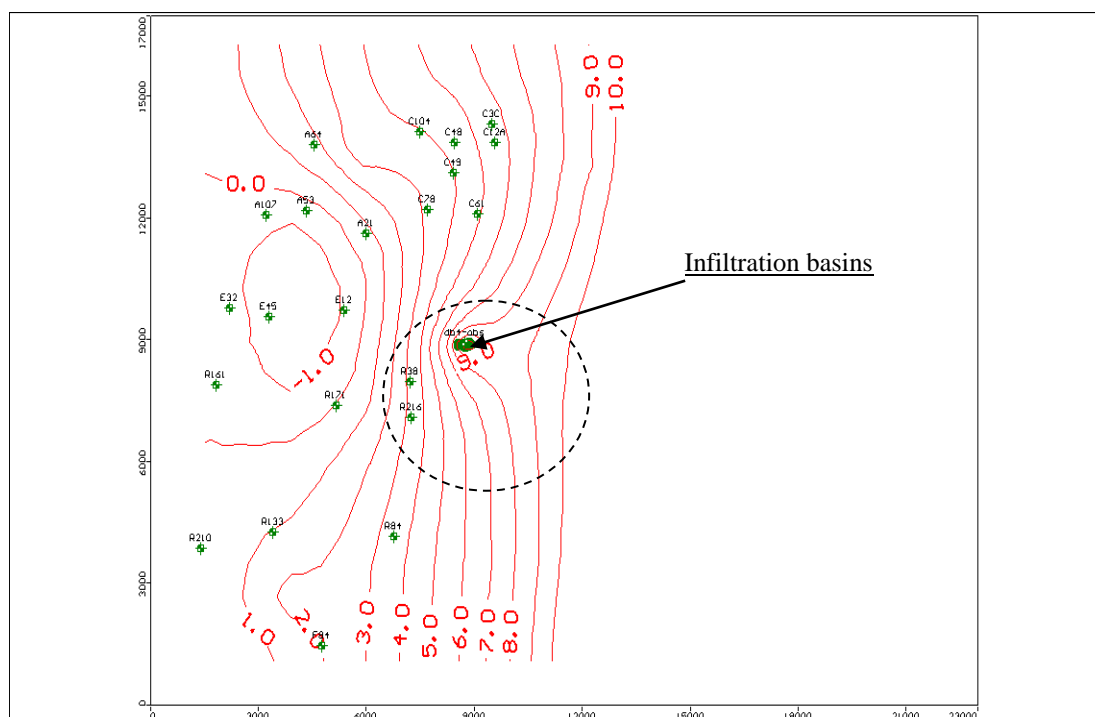


Fig. 5.19: Steady state water level contours and the extent of water mound in case of infiltration 35,600 m³/d

In order to study the lateral groundwater flow across the borders the model domain is divided into 3 different zones (Fig. 5.20). Zone 1 represents the aquifer beneath the infiltration basins

and the nearby surrounding areas (800 m from the infiltration site which is the maximum distance of the location of the recovery wells).

Table 5.14 shows that 10% from 35,600 m³/d infiltrated water may cross the borders with Israel while 90% of this amount will flow in the direction of west and north-west. Very small quantity (0.2%) will cross the Israel's border in case of 15,000 m³/d quantity is recharged. The lateral flow in the reverse direction will reduce to half due to the infiltration.

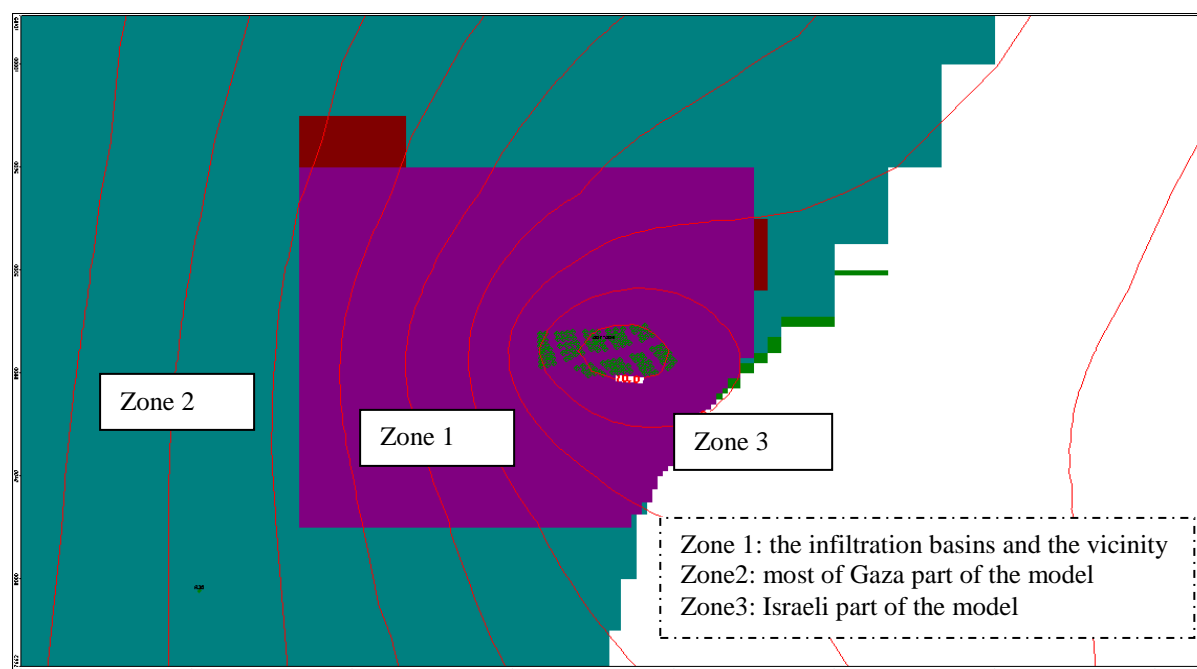


Fig. 5.20: Modeling zones for zone budget

Table 5.14: Lateral groundwater flow across the borders in the vicinity of the site.

	Infiltration (15,000 m³/d) MCM/Y	Infiltration with (35,600 m³/d) MCM/Y
Zone 1 total recharge rate	6.1	13,65
Flow from Zone1 to Zone2	6.0	12.33
Flow from Zone1 to Zone3	0.01	1,32

5.7 Recovery Well Scheme

5.7.1 Verification of the Location of Recovery Wells for Scenario I

As mentioned in Table 5.3, the maximum number of wells required to recover the infiltrated amount of water under scenario I (15,000 m³/d as infiltrated water and 16500 m³/d as recovered water) is 10 wells in June. In order to check and specify the location of these wells, Modpath module was run under steady state conditions. Fig. 5.21 shows the pollution pathlines without the operation of recovery wells under steady state. Fig. 5.22 shows the pathlines extensions after five years of infiltration which will exceed the first row wells and will be very close to the second row of recovery wells.

In Fig. 5.23, infiltration is fully captured by the 12 wells. The optimal location of the wells was selected after several runs of the model on the base that they should be able to capture all pollution. Therefore, the wells will be from the first row and the second row which are concentrated in the direction of flow. The wells will be located in the first row with a distance of 550 m from the infiltration basin and the second row will be around 750 m from the basin as shown in Fig. 5.23.

The groundwater level under the recovery wells will be 0 m with a drawdown equal 2 m. It can be concluded that the proposed wells are optimal since they will recover the entire infiltrated water quantity during a year, prohibit the escape of all pollution, and direct the flow to the wells.

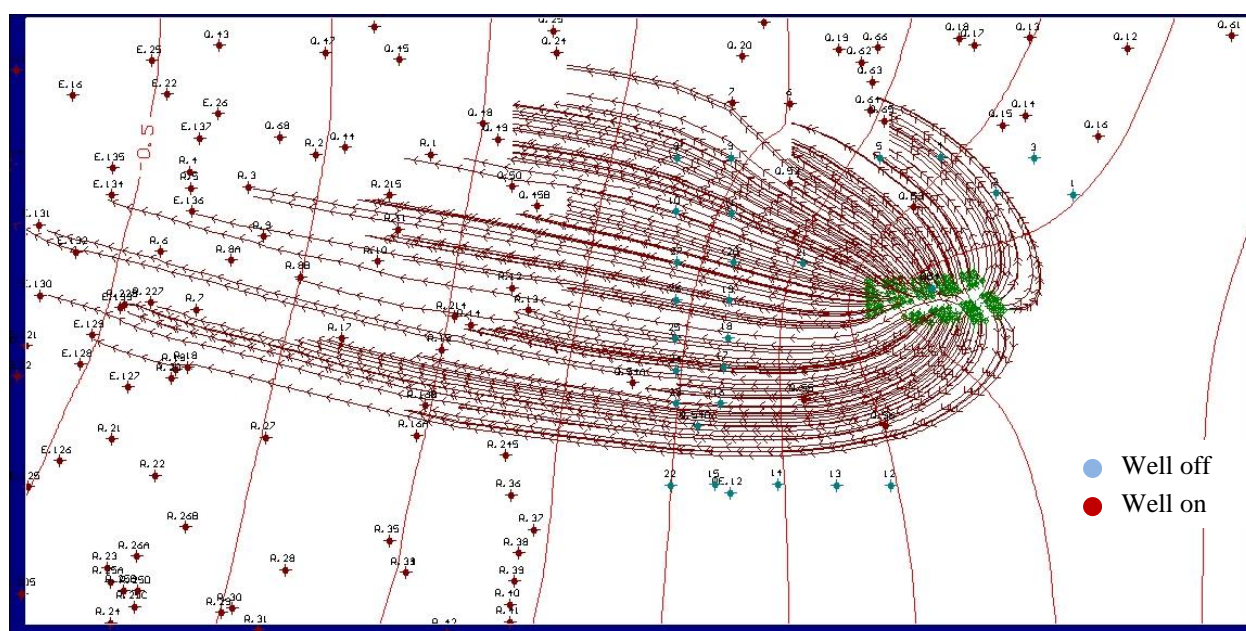


Fig. 5.21: Pollution path lines without recovery system under scenario 1

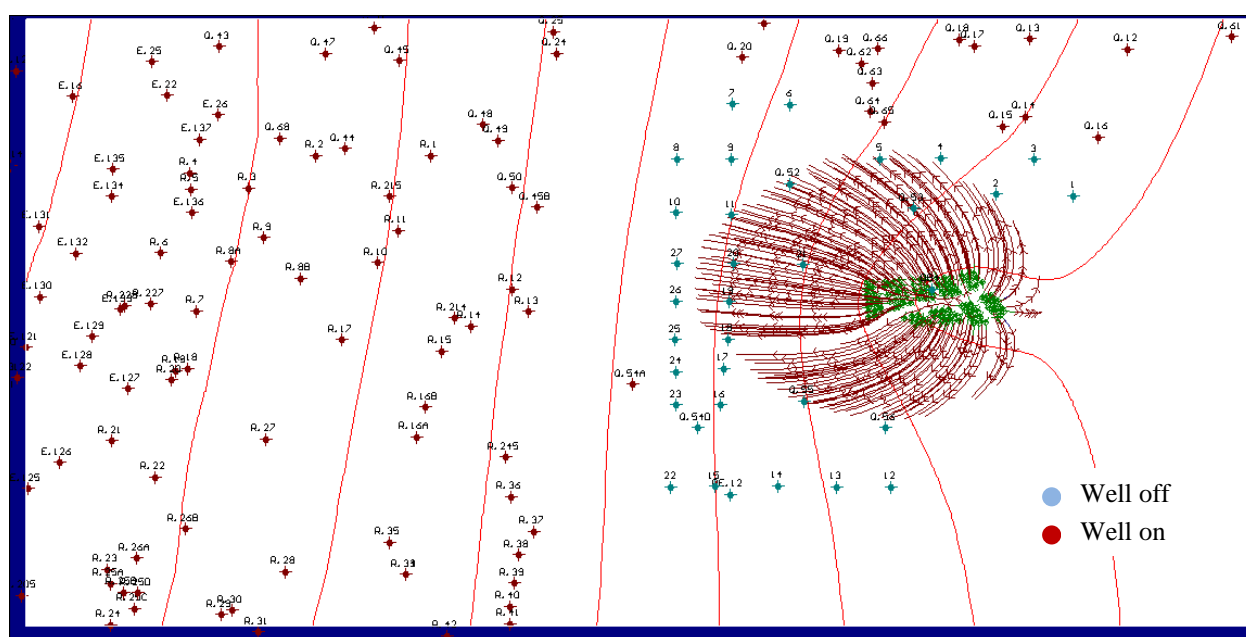
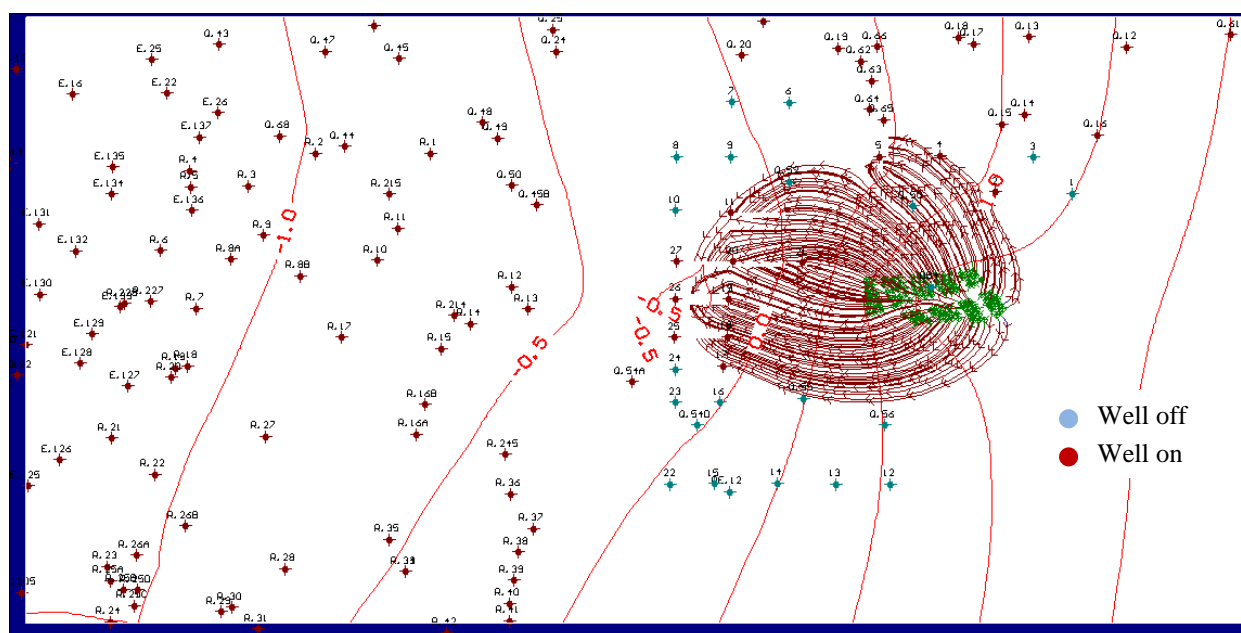


Fig. 5.22: Path lines after five years of infiltration under scenario I.



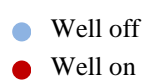


Fig. 5.24: Pollution pathlines without recovery system under scenario III

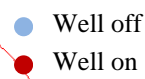


Fig. 5.25: Pathlines after five years of infiltration under scenario III without recovery

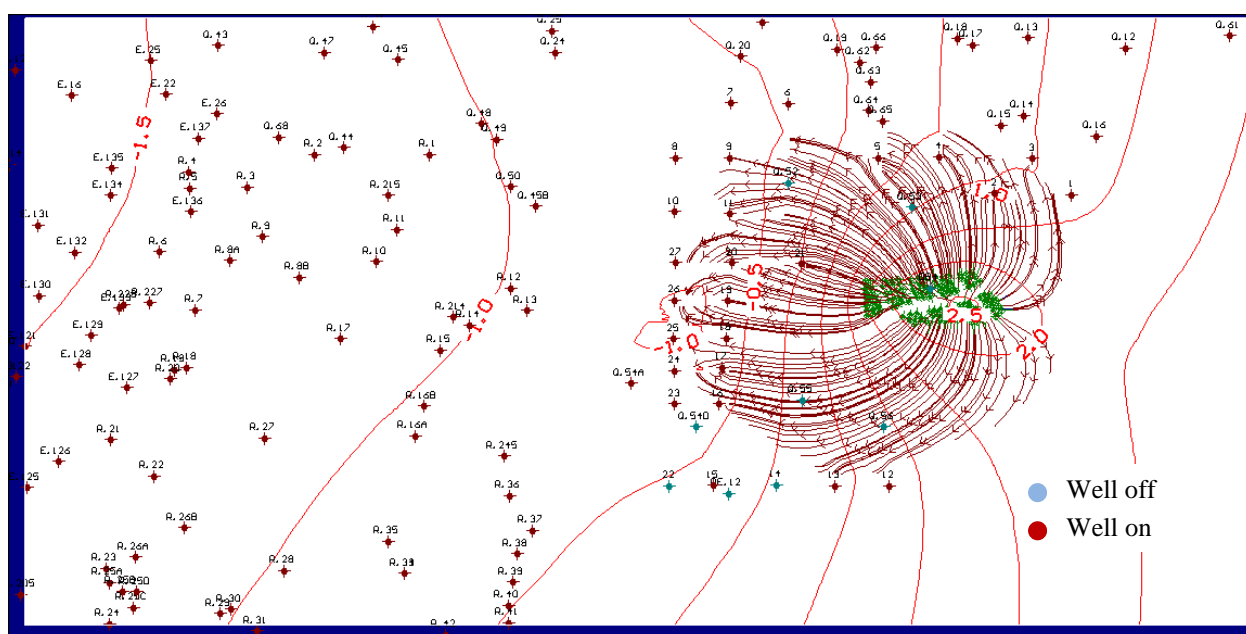


Fig. 5.26: Recovery wells and pollution path lines under scenario III.

5.8 Transport Model

In order to study which part of the aquifer that will be directly influenced by the infiltration, the module Modpath was used to simulate the advective transport. Thereafter the dispersion was examined by simulation of pollution in the infiltration water using the MT3D module for labelled water containing soluble, non-reactive contaminant.

The parameters that principally influence mass transport in the flow model are effective porosity and dispersivity. The effective porosity, n_e , has been set to 25 %. The uncertainty for the parameter is considered to be small, approx. 5 % (SWECO INT., 2003, EA, 2006). Reducing n_e will result in increased particle velocity which affects the time aspect in advective transport.

Dispersivity has been set to values ranging from 3 m to 12 m calculated by the following equation (SWECO INT., 2003):

$$D_L = 0.83 \log L^{2.414}$$

where D_L concerns longitudinal dispersivity and L is the length of the mass transport plume considered. Comparison of simulations shows that this difference in dispersivity does not result in any measurable changes of the diffusion plume.

5.8.1 Impacts of the Infiltration

In order to study the transport due to advection-dispersion, MT3D module simulation has been performed using a pollution tracer which could be Chloride, $\text{NO}_3\text{-N}$ or any chemical. However the BOD was considered as indicator for the influent which has a range of 10 to 100 mg/l which indicated the good quality and bad quality of water. The pollution concentration in the aquifer was set to 0 mg/l. This simulation allowed for a clear picture of the spreading of the labelled water, since, any deviations from the zero level is a direct effect of the infiltration. For example partially treated wastewater from BLWWTP is characterized by high N-content in all forms.

Lacks of aeration in the aerated lagoon hinder the formation of nitrate and degradation of the organic matter. Moreover the lagoon system is unfit for denitrification process. Using large area infiltration basins with good management system will enhance the nitrification process in the soil top layers and denitrification in the deeper layers. The partially treated wastewater will supply Carbon to the soil deeper layers enhances the denitrification process, but this may not go further than few meters. Hence there will not be effective denitrification process during the emergency phase treatment or passage through the unsaturated and saturated zones.

Consecutive drying of the flooded basins will supply enough oxygen that will enhance the nitrification process. As a result it is assumed that 90% of the Kjeldal nitrogen will end up as nitrate in the aquifer. This may lead to an overestimation of the resulting concentration of nitrogen compounds in the groundwater, but there are no data available to support. This effect is presently difficult to quantify. The transport model was run using the scenarios presented as in Tables 5.8 to 5.13.

Fig. 5.27 shows that the pollution will be extended to a distance of 1000 m in the west and north-west direction of the basin in year 2015 if bad quality of water ($15,000 \text{ m}^3/\text{day}$) is infiltrated in the basin starting from year 2009. In addition, around 20 agricultural wells will be negatively influenced.

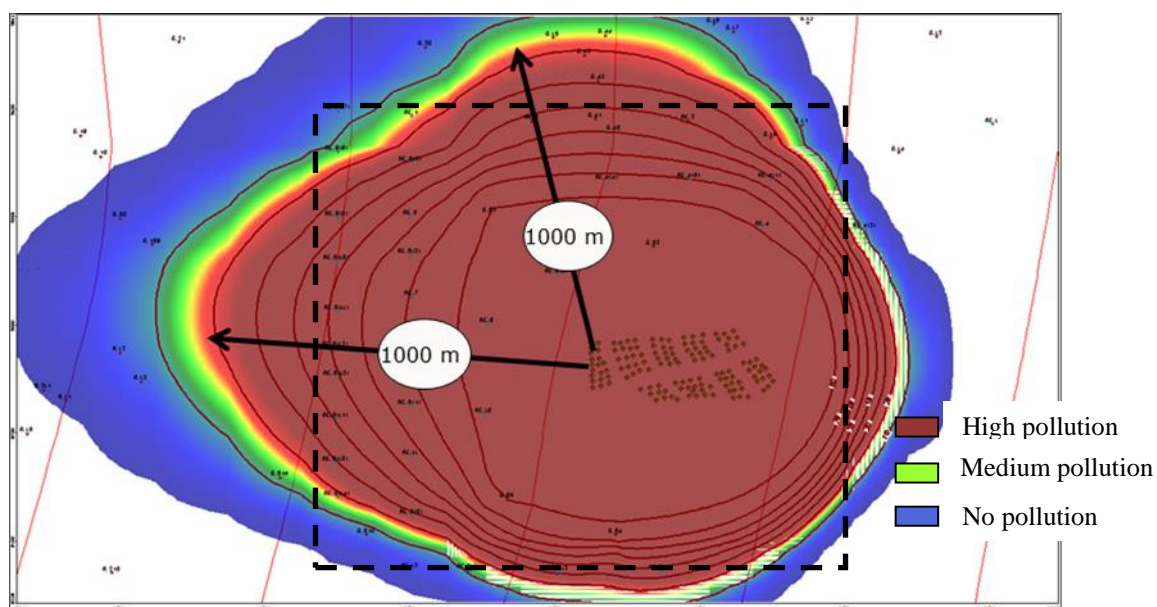


Fig. 5.27: The pollution plume in year 2015 (current infiltration, no recovery)

Fig. 5.28 shows that the pollution will be extended to a distance of 1800 m in the north-west direction of the basin in year 2022 if bad quality of water ($15,000 \text{ m}^3/\text{day}$) is infiltrated in the basin starting from year 2009. In addition, around 35 agricultural wells will be negatively influenced.

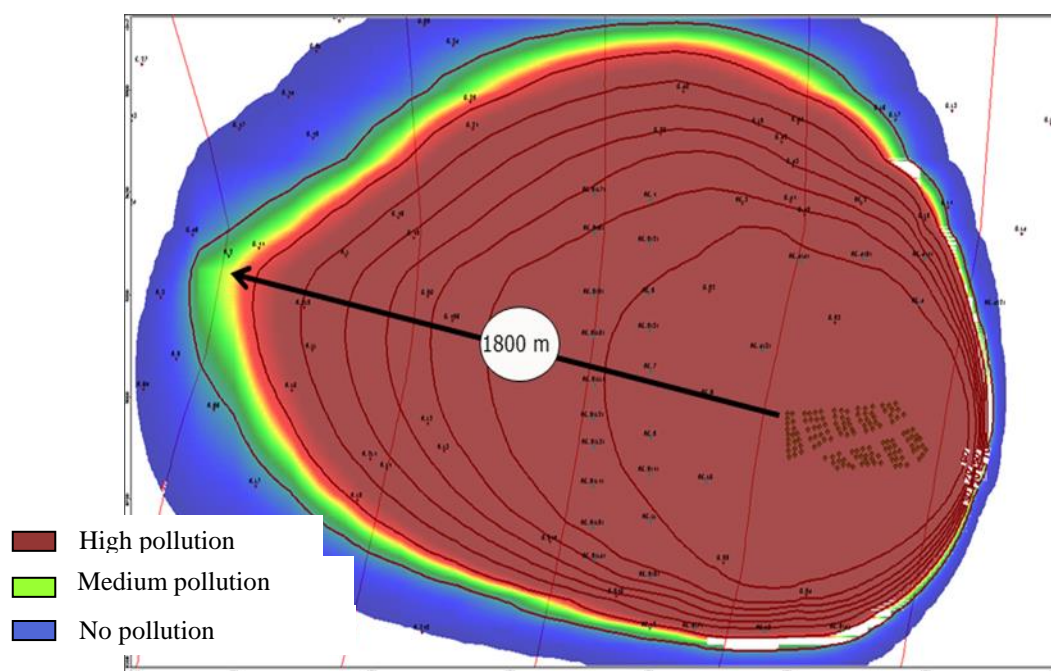


Fig. 5.28: The pollution plume in year 2022 ($15000 \text{ m}^3/\text{day}$ with bad quality)

Fig. 5.29 shows that the pollution will be extended to a distance of 2300 m in the west and the north-west directions of the basin in year 2025 if bad quality of water ($35,600 \text{ m}^3/\text{day}$) is infiltrated in the basin starting from year 2015. In addition, around 55 agricultural wells and some of municipal wells will be negatively influenced.

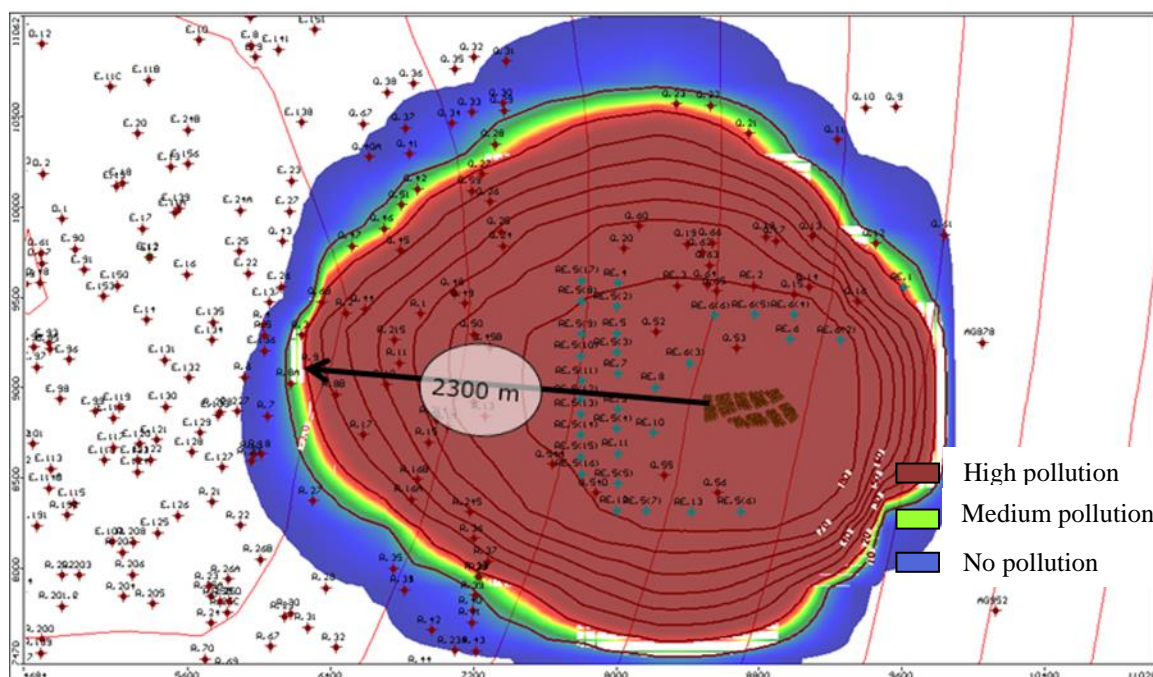


Fig. 5.29: The pollution plume for year 2025 if 35600 m^3 is infiltrated with bad quality

5.9 Monitoring Program

5.9.1 Monitoring Strategy and Plans

Before preparing a groundwater monitoring plan, the overall strategy of the groundwater monitoring program should be defined to guide the development of the plan. In this sense, “strategy” refers to the manner in which a hypothetical release from a regulated unit will be detected or measured. Examples of issues that should be addressed when developing a monitoring strategy include:

- (1) The type of monitoring data needed;
- (2) The locations (both horizontal and vertical) from which the samples are to be collected (i.e., definition of “target monitoring zones”);
- (3) The manner in which the samples will be obtained; and
- (4) The ability of the monitoring features to rapidly detect a change in groundwater quality. For detection monitoring programs,

The types of data needed are usually defined by regulation; for other types of monitoring programs, the types of data needed are typically based on site-specific considerations.

Development of a groundwater monitoring strategy is illustrated in Figs. 5.30 and 5.31. As shown in these figures, the potential sources of contamination and the aquifers of concern should be characterized before developing a groundwater monitoring strategy because selection of target monitoring zones cannot be made until the source and the aquifer of concern have been evaluated, usually through a detailed hydrogeologic evaluation of the site. When evaluating the ability of a monitoring system to rapidly detect a release from the potential source, the impact of preferential flow paths and vertical gradients should be carefully evaluated; a two-dimensional analysis of groundwater elevation may not reveal actual upgradient or down gradient locations of groundwater flow. The presence of vertical gradients may significantly affect the selection of monitoring locations.

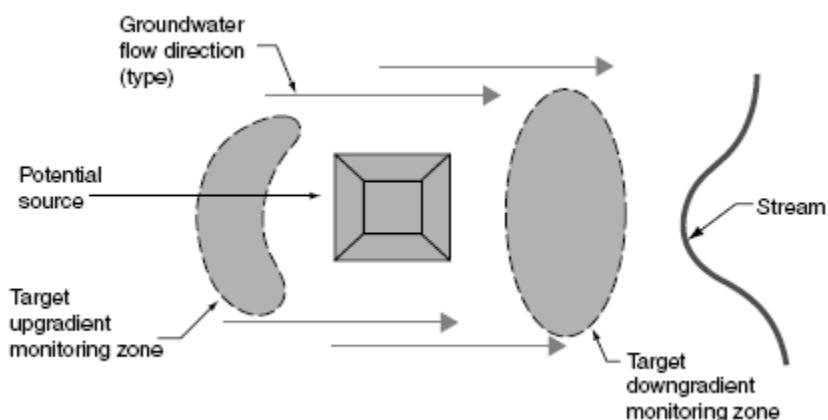


Fig. 5.30: Plan view of typical unconfined aquifer groundwater monitoring system.

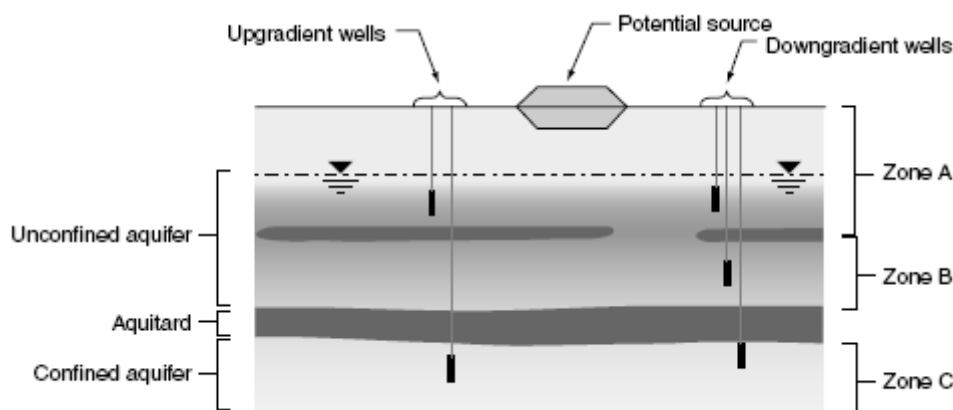


Fig. 5.31: Vertical cross section of target monitoring zones.

5.9.2 Monitoring Locations and Parameters:

Locating the appropriate monitoring point locations is essential in designing a monitoring network capable of providing data of adequate quality to achieve the program objectives. At times, monitoring well locations may be prescribed by the regulations under which the groundwater monitoring program is being developed. For example, some regulations require monitoring Locations be placed at a designated “point of compliance,” which is often at the property boundary or a groundwater discharge location. For other groundwater monitoring programs, the groundwater professional should select monitoring locations that provide the most reliable data needed to detect or assess a groundwater contaminant plume. To verify that the monitoring network can accomplish this goal, target monitoring zones must be selected based on the site hydrogeologic conditions and anticipated contaminant pathways. Fig. 5.32 shows the recommended locations of the monitoring wells which was set up based on the location of the recovery wells.

The overall strategy of the groundwater monitoring program in this project to evaluate the status of the groundwater quality after infiltration of partially treated and treated wastewater. The monitoring wells are distributed in two rows: around 400 to 500 m from the infiltration basin and the second row will be of 1100 to 1200 m from the basin. The first monitoring well row should be located before the first row of the recovery well in the direction infiltration basin, and the second row of the monitoring wells should be located after the second row of the recovery wells to check the quality of groundwater outside the recovery wells areas. The monitoring network will also use the existing 5 monitoring wells constructed recently by PWA and used to monitor the infiltration basin. In addition, the recovery wells will be part of the monitoring network as shown in Fig. 5.32.

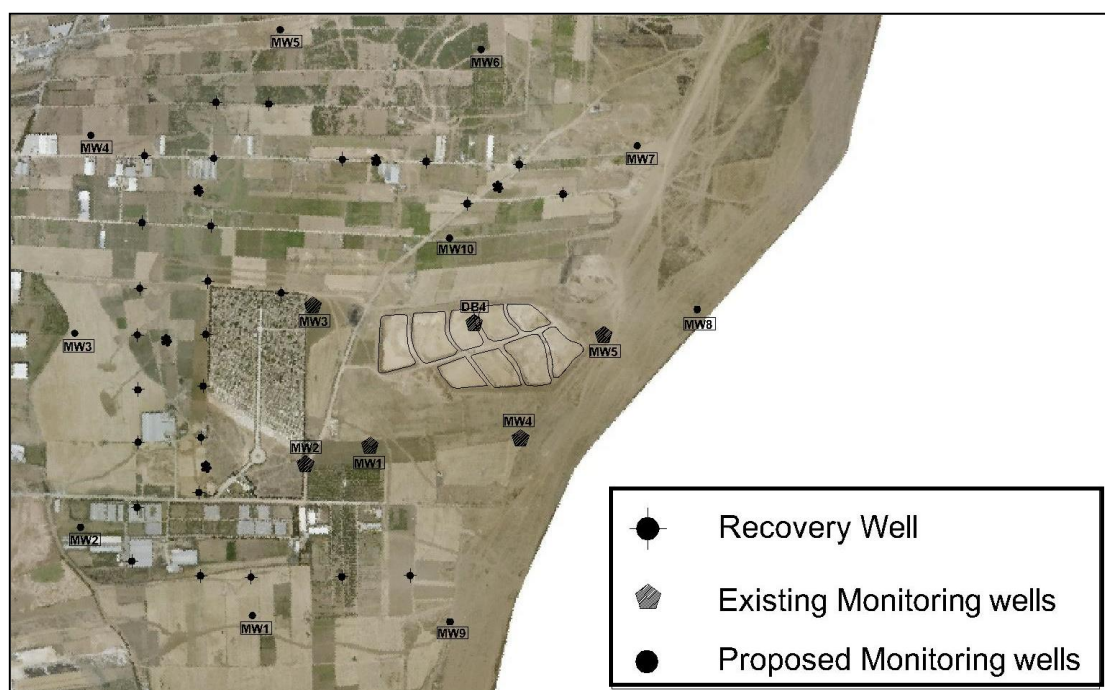


Fig. 5.32: Monitoring wells location

After determining the number and location of observation wells, the parameters to be monitored should be specified. The main objective of monitoring is to check the groundwater quality after infiltration and check the operation of Soil Aquifer Treatment process. The consultant made extensive reviews of similar projects such Gosh Dan Project where several parameters are monitored. Among these parameters, the consultant proposed in Table 5.15 some parameters which could reflect the status of groundwater after infiltration of partially treated wastewater and could be analyzed in Gaza Strip laboratories.

Table 5.15: Monitored Parameters and Frequency of Monitoring

Parameters	Frequency of Monitoring
Water Level	Monthly
pH	Four Times a year
TDS	Four Times a year
BOD	Four Times a year
COD	Four Times a year
DOC	Four Times a year
TC	Four Times a year
Ammonia as N	Four Times a year
NO₃	Four Times a year
NO₂	Four Times a year
T.N	Four Times a year
Cl	Four Times a year
Detergent	Four Times a year
F.C	Four Times a year
Phosphrous	Four Times a year
Heavy Metals	Four Times a year
O₂	Four Times a year
Nitrogen and Oxygen Isotopes	Four Times a year
Mg	Four Times a year

To determine what action should be taken to reduce nitrate contamination of the aquifer due to the recharge of untreated or partially treated wastewater, it is important to assess to what extent the problem is due to present of the nitrate from the source of infiltrated wastewater. Samples will be collected from the monitoring wells to characterize the geochemistry of groundwater. The nitrogen and oxygen isotopes of groundwater nitrate will be used in conjunction with other geochemical data to place constraints on potential nitrate sources. The $\delta^{18}\text{O}_{\text{nitrate}}$ values of the in the monitoring wells will indicate that the nitrate is primarily derived from nitrification of ammonium in the soil. The $\delta^{15}\text{N}_{\text{nitrate}}$ values suggest that direct wastewater sources predominate; however, the influence of wastewater can be seen in the elevated $\delta^{15}\text{N}_{\text{nitrate}}$ values of some of wells. The value of $\delta^{15}\text{N}_{\text{nitrate}}$ will distinguish water contaminated by infiltrated wastewater as opposed to agricultural land use.

6 DESIGN BASIS AND PARAMETERS

This chapter includes the basis, parameters and methods used for the design of the recovery (recovery wells, collection pipes, observation wells and associated facilities), and the reuse (water tanks, booster pumping station, irrigation water network and associated facilities) schemes.

6.1 Demand for Irrigation Water for System Design

The system design requires the determination of irrigation water demands, especially during the peak and the lowest summer and winter seasons. A comprehensive study was carried out for the determination of the irrigation plan in the project area. The study has taken into consideration main influencing factors and requirements such as crop patterns, water quality, agricultural zones, irrigation scheduling and demands, soil characteristics, environmental factors, weather, climate change, leaching requirements, losses, etc. According to the study the total agricultural land in the project area is about 15,000 dunoms. The agricultural land was subdivided into six zones (zones A, B, C, D, E and F) of almost equal size averaging 2500 dunoms each. Each zone is to be irrigated once each 6 days. The results of the study were submitted to the client as a special report (agriculture report) and for convenient is also included in *Appendix 1* in this report. As indicated in the agricultural report in *Appendix 1*, there are variations in the demand during the year. Moreover, a field survey indicated that there will be a variation in the demand during the day as most farmers prefer to irrigate in the morning. Therefore, the variations in irrigation demands across the year and during the day will influence the design of the physical components of the reuse scheme that includes the water tanks, booster pumping station, and irrigation networks.

In reference to the agricultural report in *Appendix 1*, the monthly irrigation demand for water for the 2015 design phase is shown in Table 6.1 (Scenario III). The peak demand of 50,885 m³/day is in the month of June and the lowest demand of 30,187 m³/day is in the month of October. Furthermore, in spite of the constant pumping from the recovery wells during the irrigation hours, there will be a variation in the water demand during the day due to irrigation preference by farmers. The design and the operation of the project components need to consider all of these variations, especially the peak and the lowest values. For this purpose the consultant has made special study to determine the variation during a peak day to acquire farmer irrigation preferences and farm sizes.

Table 6.1: Daily recovered water (m³/day) (from the irrigation report in *Appendix 1*).

Scenario	I	II	III
Jan.	14799	20718	33081
Feb.	15091	21127	35816
Mar.	14010	19614	34995
Apr.	13997	19595	34204
May	19634	27488	46622
June	21140	29596	50885
July	20262	28367	50136
Aug.	21269	29777	49073

Sept.	17476	24466	40290
Oct.	12459	17443	30187
Nov.	13147	18406	31484
Dec.	14716	20602	33146
Average	16500	23100	39160

6.1.1 Variation in the Irrigation Demand during the June Peak Summer Day

The hourly variation in irrigation demands during a summer day in the peak month of June has been determined based on the number and size of farms as well as the irrigation preferences by farmers. For this purpose the consultant has carried out a comprehensive field survey to determine the number and sizes of farms in each of the six irrigation zones within the project area. In addition, the consultant has prepared a special questionnaire to acquire irrigation preferences by farmers during a 12 hour working day. Thirty farmers have completed the questionnaire.

Irrigation time preference: The results of the questionnaire shown in Fig. 6.1 clearly indicate farmer preference to irrigate in the morning hours. This preference has been considered in the irrigation plan discussed in this section.

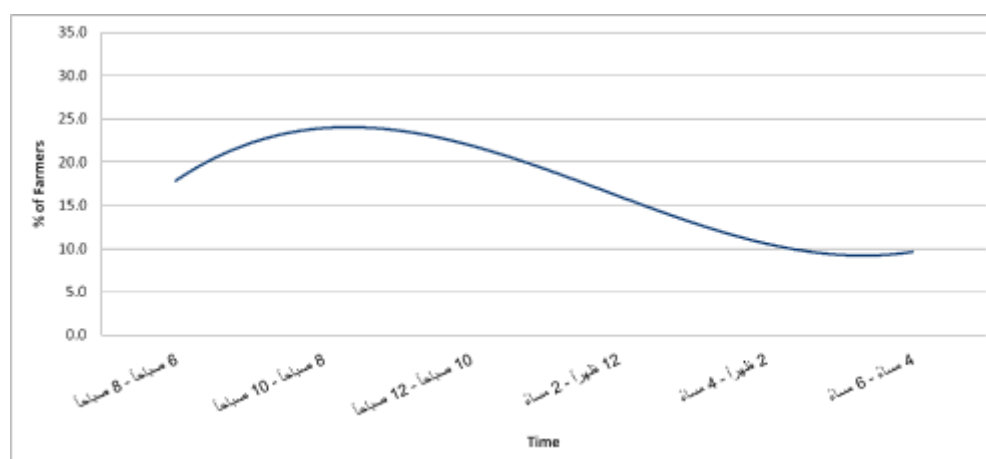


Fig. 6.1: Irrigation time preference by farmers.

Farm sizes and numbers: Tables A1.1 to A1.12 in *Appendix 1 (Water Demand for Irrigation)* include the results of field survey concerning the number and size of farms, the irrigation plan and the storage capacity required for each of the six irrigated zones (i.e. Zones A, B, C, D, E, and F). For clarity, Figs. A1.1 to A1.6 in the same appendix includes the cumulative water demand and supply and thus the storage capacities for each of the six irrigation zones. The results of the field survey indicate that most of farms are of small size. It should be mentioned that the consultant was not able to investigate some agricultural land (less than 10% of the total land) beside the border with Israel, since these lands were previously excavated by Israeli army. To overcome this problem, these lands were reasonably assumed to have same distribution similar to the remaining farms within each irrigated zone.

Irrigation Zone F: The details of the irrigation tables and figures in *Appendix 1 (Water Demand for Irrigation)* are explained using Tables 6.2 and 6.3 (Tables A1.11 and A1.12 in *Appendix 1*) and Fig. 6.2 (Fig. A1.1 in *Appendix 1*) for the irrigated Zone F as an example. It should be

mentioned that the study showed that the requirements of Zone F is the most critical for the design. It should also be mentioned that numerous trials have been investigated until reaching the final results shown these tables and figures and discussed as follows:

1. Input data for all irrigation zones: The irrigation time starts at 7AM and ends at various times depending on the farm size. The minimum irrigation period for the smallest farm size of less than 1.5 donum is 4 hours. The irrigation period increases by one hour for each 1.5 donum increase in the farm size until reaching the maximum of 12 hours for farms larger than 12 donums where irrigation starts at (7 AM) and ends at 19 (7 PM). The irrigation demand which is also equal to the irrigation supply from the recovery wells for each zone is 50,885 m³/day. It should be mentioned that the average area of the six irrigation zones is 2500 donums with maximum difference of less than 4% compared to actual zone areas. This small difference is insignificant since actual zones will include nonfarm areas such as roads, buildings, etc. that was assumed to comprise of about 20% of total area.
2. Table 6.2 shows that most of farms (more than 56%) are of small size with areas less than 6 donums and few farms (less than 20%) have areas larger than 12 donums. This distribution trend of farm sizes can be found in all irrigation zones. The day water demand for each irrigation period and the hourly demand for each hour are also shown in Table 6.2. It should be noted that the hourly demand for each irrigation period is equal to the day demand subdivided by its relevant period of irrigation. For example, for the first irrigation period the 152.7 m³/day is supplied in 4 hours ($Q \text{ per hour} = 152.7/4 = 38.2 \text{ m}^3/\text{hour}$).
3. Table 6.3 shows the water demand for each hour from all periods. For example, the water demand for the first hour (from 7 to 8) is equal to the summation of the hourly demands from all irrigation periods ($38.2 + 397.4 + 1009.2 + \dots + 2110.6 = 5574 \text{ m}^3/\text{hr.}$). This is so since there is irrigation in this hour for all farms, i.e. all irrigation periods start at 7 AM. On the other hand, for the water demand for the last irrigation period from 18 (6 PM) to 19 (7 PM) is equal to demand from the last irrigation period only (2110.6 m³/hr.) since there is irrigation in this hour for larger size farms (> 12 donums) only, i.e. only last irrigation period ends at 19 (7 PM). The water supply is constant for all hours and equal to 4240.4m³/hr. as shown in Table 6.3. For the purpose of calculating the water storage capacity, the accumulative water demand and supply have been calculated in Table 6.3 and depicted in Fig. 6.2. Subsequently, the storage capacity is determined as the maximum difference between the accumulative demand and supply.
4. The results of this analysis for the critical irrigation Zone F, the maximum irrigation demand equals to 5574 m³/hr. is required during the morning hours where all farms are being irrigated. The minimum irrigation demand equals to 2110.6 m³/hr. is required during the last hour from 18 to 19 where only farms larger than 12 donums are being irrigated. The water storage capacity was found equal to 7528 m³.

Table 6.2: Farm irrigation requirements for Zone F during the peak month of June.

(Area = 2432 dunoms Q demand = 50885 m³/day = 50885/2432 = 20.932 m³/dunom)

Irrig. Period (Hrs.)	Irrigation Time		Farm Area (duns.)	No. of Farms	Farm Area within each Irrigation Period (du.)	Q Demand per day "irrigation period" (m ³ /day)*	Q Demand per 1 hr. (m ³ /hr)*
	From	To					
4	7	11	(<1.5)	5	7.3	152.7	38.2
5	7	12	(1.5<3)	35	95.0	1987.1	397.4
6	7	13	(3<4.5)	65	289.4	6055.5	1009.2
7	7	14	(4.5<6)	34	213.5	4467.7	638.2
8	7	15	(6<7.5)	19	153.4	3209.5	401.2
9	7	16	(7.5<9)	17	165.8	3469.1	385.5
10	7	17	(9<10.5)	13	150.1	3140.0	314.0
11	7	18	(10.5<12)	11	147.0	3075.7	279.6
12	7	19	(>12)	49	1210.5	25327.7	2110.6
Summation				248	2432.0	50885.0	

* Amount of irrigation required in 12 hr working day is supplied during the "irrigation period". For example for the first irrigation period, the 152.7 m³/day is supplied in 4 hours (Q per hour = 152.7/4 = 38.2 m³/hr).

Table 6.3: Hourly irrigation demand and storage for Zone F during the peak month of June.

Time	Q Demand (m ³ /hr)	Q Supply (m ³ /hr)	Cumulative Demand (m ³)	Cumulative Supply (m ³)	Difference = Demand - Supply (m ³)	Storage = max. difference (m ³)
7-8	5574.0	4240.4	5574.0	4240.4	1333.6	7527.6
8-9	5574.0	4240.4	11148.0	8480.8	2667.1	
9-10	5574.0	4240.4	16721.9	12721.3	4000.7	
10-11	5574.0	4240.4	22295.9	16961.7	5334.3	
11-12	5535.8	4240.4	27831.7	21202.1	6629.7	
12-13	5138.4	4240.4	32970.1	25442.5	7527.6	
13-14	4129.1	4240.4	37099.3	29682.9	7416.4	
14-15	3490.9	4240.4	40590.2	33923.3	6666.8	
15-16	3089.7	4240.4	43679.9	38163.8	5516.1	
16-17	2704.2	4240.4	46384.1	42404.2	3980.0	
17-18	2390.2	4240.4	48774.4	46644.6	2129.8	
18-19	2110.6	4240.4	50885.0	50885.0	0.0	
Sum	50885.0	50885.0				

- (Q_{max}/Q_{ave}) = 5574.0/4240.4 = 1.31

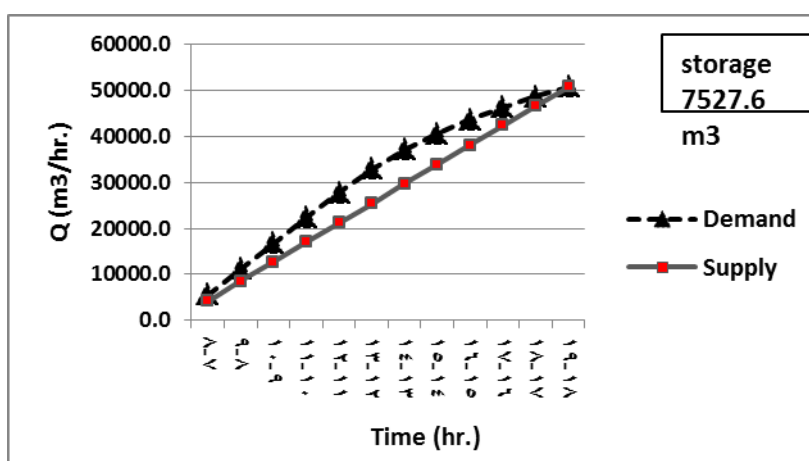


Fig. 6.2: Accumulated irrigation demand and supply for Zone F during the peak month of June.

6.1.2 Design Criteria and Parameters for a Peak Summer Day in June

The following design criteria and parameters are determined based on the analysis results for all irrigation zones given in Tables A1.1 to A1.12 and Figs. A1.1 to A1.6 in *Appendix 1 (Water Demand for Irrigation)*. As mentioned earlier irrigation Zone F was found to have critical design requirements for the considered peak summer day.

1. Two water storage tanks of 4000 m³ each (8000 m³ altogether) are used where the slight increase compared to maximum required volume (7528 m³) in the size is to allow for proper connections, etc. For other irrigation zones, this increased storage capacity will provide additional flexibility in the operation of the system. The capacities of water tanks satisfy the hydraulic and mechanical operational requirements.
2. Maximum hourly pumping rate is 6000 m³/hr. where the slight increase in the rate (7%) compared to maximum required (5574 m³/hr.) is taken as a factor of safety and to allow for more flexibility in the operation.
3. The minimum hourly pumping rate is 2100 m³/hr. It should be mentioned that it is not necessary to reduce this rate in the design as a factor of safety as has been done for the maximum rate. This is since the pumping capabilities and control system will ensure that the minimum pumping rate will never be exceeded.
4. The 6000 m³/hr maximum and the 2100 m³/hr minimum hourly pumping rates are considered in the design of pumping station, trunk lines, irrigation and networks for the six irrigation zones, and associated facilities. However, the design of each irrigation zone will be checked against the actual maximum and minimum required values given in Tables A1.1 to A1.12 in *Appendix 1 (Water Demand for Irrigation)*.

6.1.3 Influence of Lowest Demand during Winter Season

Similar analysis has been carried out for the lowest demand period given in Table 6.1 during the month of October. Numerous trials have been tried in order to reach optimal irrigation plan which is considered in this report. The results of this analysis are given in Tables A1.13 to A1.24 and Figs. A.1.7 to 6.12. The following notes are related to this analysis:

1. The irrigation time starts at 8 AM and ends at various times depending on the farm size. The minimum irrigation period for the smallest farm size of less than 2.5 donum is 4 hours. The irrigation period increases by one hour for each 2.5 donum increase in the farm size until reaching the maximum of 8 hours for farms larger than 10 donums where irrigation starts at (8 AM) and ends at 16 (4 PM).
2. The irrigation demand which is also equal to the irrigation supply from the recovery wells for each zone is 30,187 m³/day.
3. The design criteria and parameters applied during the peak demand summer month were found adequate for the lowest demand month. The difference is in the operation plan only where the working day is 8 hours instead of 12 hours.

6.1.4 Summary of Water Demand for Irrigation of the Six Zones

Table 6.4 includes the results of analysis for the water demand for the six irrigation zones for both the peak and lowest months of June and October, respectively which are detailed in *Appendix 1 (Water Demand for Irrigation)*. The table shows the maximum and lowest hourly

demands and the storage requirements for each of the zone during the months of June and July. This information is necessary for the design of the project physical components as well as for operation purposes. It should be mentioned that the irrigation hours for the June and October months are 12 hr. and 8 hr. respectively.

Table 6.4: Summary of water demand for the six irrigation zones for June and October months.

Irrigation Zones	Peak June Month					Lowest October Month				
	Working hours	Constant Supply (m ³ /hr.)	Max. Demand (m ³ /hr.)	Min. Demand (m ³ /hr.)	Storage (m ³)	Working hours	Constant Supply (m ³ /hr.)	Max. Demand (m ³ /hr.)	Min. Demand (m ³ /hr.)	Storage (m ³)
Zone A	12	4240.4	4544	3580.2	1789.8	8	3773.4	3920.6	3391.9	672.9
Zone B			4922.3	2846.8	4142.9			4115.3	2764.6	1660.8
Zone C			4731.6	3389.2	2848.7			4026.7	3086.5	1202.6
Zone D			4921.9	2870	3987.8			4107	2842.3	1550.9
Zone E			5149	2256.5	5688.7			4215.4	2355.1	2189.1
Zone F			5574	2110.6	7527.6			4461	2162.5	3223

6.1.5 Operation Plans through the Year

The developed design criteria and parameters have shown to satisfy the peak and lowest irrigation demands. It is recommended to adopt these two operation plans during the whole year. The first plan during the summer season extends for five months from May to September in which the monthly irrigation demand varies from 40,290 m³/day to 50,885 m³/day. During the summer season operation plan, the 12 hour working day plan is applied. The second plan during the winter season extends for seven months from October to April in which the monthly irrigation demand varies from 30,187 m³/day to 35,816 m³/day. During the winter season operation plan, the 8 hour working day plan is applied. Other operation plans could be investigated after the approval of the system design. However, the best plan can only be determined based on actual data after the construction and operation of the system.

6.1.6 Flexibility in the Operation

The operation plans are flexible and allow for any variation in accordance with actual conditions of the project upon implementation. The flexibility is provided as follows:

1. It is possible to increase the irrigation duration for even more than 12 hours if needed which will supply more water for irrigation or for any purpose. In this case the recovery wells and pumping station can work for longer times.
2. The networks for all irrigation zones can transport larger quantities of water (up to 6000 m³/hr) than indicated in the operation plan and in the same time the maximum allowable velocities are not exceeded.
3. The water storage tanks during the whole year (apart of the control peak month of June for Zone F) have larger capacities than needed for any irrigation zone.

4. An emergency pipe connecting the pump station to the infiltration basins is suggested to allow pumping of recovered water back to the infiltration basin if necessary. This situation may arise in case there was a delay in irrigation for long periods.
5. It is possible for farms to irrigate for longer or shorter periods than indicated in the operation plans. In such a case, irrigation duration for each period is changed since at the end of each day the whole quantity for the irrigation demand is provided and the control system will ensure the relevant minimum and maximum design limitations.
6. Extra two recovery wells are to be used to safeguard against any malfunction in the wells or in case of larger amounts of recovered water are needed.
7. The recovery wells will be designed to have a slightly higher capacity than that specified for operation.

6.2 Bases and Parameters for Hydraulic and Mechanical Design

6.2.1 Wells

Well design is the process of specifying the physical materials and dimensions for a well. A good well design depends on many factors, the type of aquifer and its characteristic, the depth of water level, the pumping rate, and the type of pumps. Fig. 6.3 shows the various well sections of a typical well design. The purpose and design of these well sections, and their position in the well, are discussed as follows:

6.2.1.1 Casing Section

The pump housing is the upper section of blind casing that supports the well against collapse, and in which the pump is installed. The length of the pump housing should be chosen so that the pump remains below the water level in the well, for the selected discharge rate, under all conditions, and over the total lifetime of the well. Pump housing is always required when submersible pumps are used. The diameter of the pump housing should be large enough to accommodate the pump with enough clearance for installation and efficient operation. It is recommended that the pump housing be two pipe sizes larger than the nominal diameter of the pump; the diameter of the pump depends on the selected discharge rate and the pump type (Delleur, 2007)⁷.

⁷ Delleur J. W., (2007). *The Hand Book of Groundwater Engineering*, Second Edition.

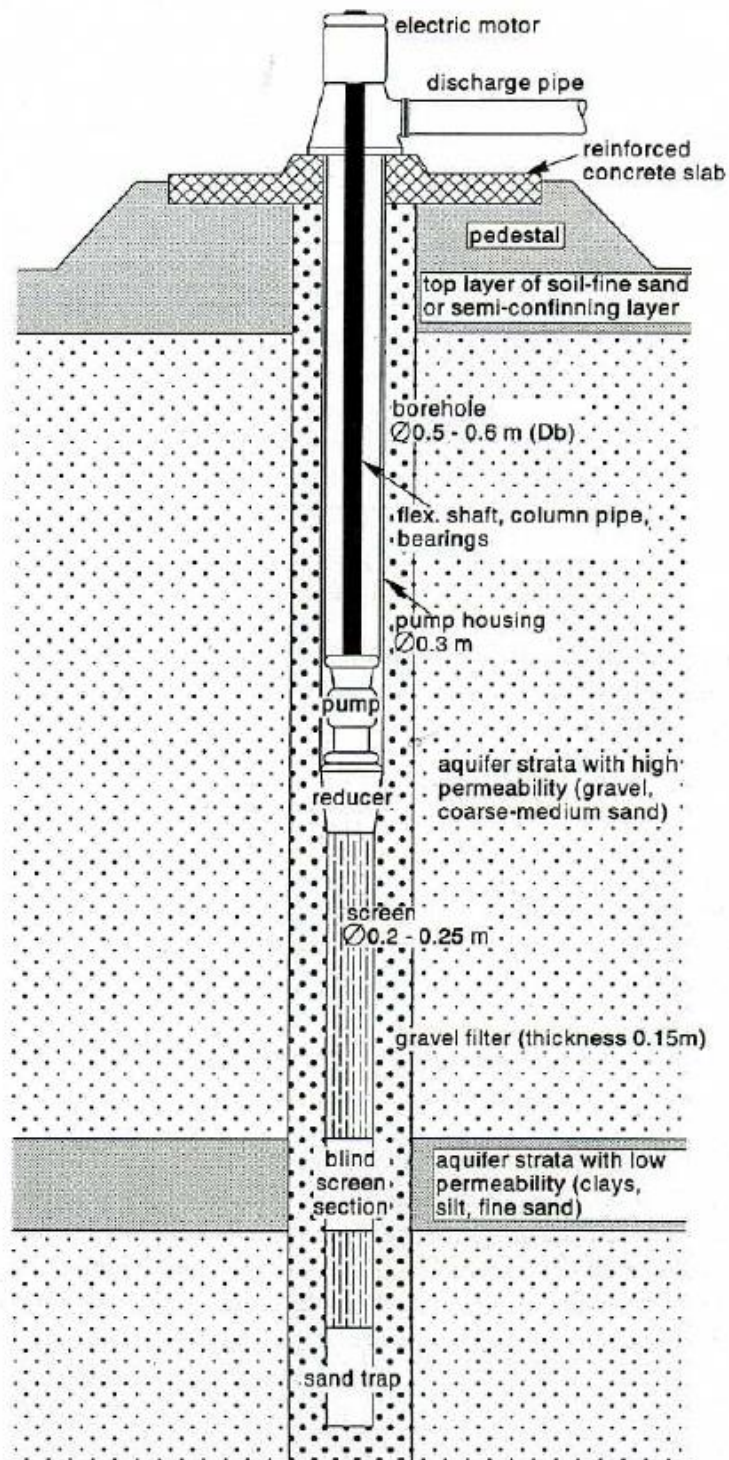


Fig. 6.3: Typical well sections.

The production casing is the lower section of blind pipe between the bottom of the pump housing and the top of the aquifer. The production casing is not required in unconfined aquifers at shallow depth where the pump housing reaches sufficiently deep into the top section of the aquifer. The length of the production casing depends on the thickness of the aquitard overlying the pumped aquifer. To minimize the head losses in the production casing itself, the upward velocity of the pumped water should be less than 1.5 m/s. Based on this criteria, Table 6.5 shows casing sizes recommended for various pumping rates; for the pipe sizes and pumping rates shown in this table, the head losses will be small. Moreover, the diameter of the production casing should be smaller than the diameter of the pump housing and should be larger or equal to the diameter of the underlying screen section.

Table 6.5: Maximum Pumping Rates for Standard-Weight Casing, Based on an Upward Velocity of 1.5 m/s.

Casing Size		Maximum Pumping Rate (m ³ /d)
(inch)	(mm)*	
4	102	1090
5	127	1690
6	152	2450
8	203	4250
10	254	6700
12	305	9590
14	337	11700
16	387	15500
18	438	19800
20	489	24700
25	591	36100

*Actual inside diameter

(Data from Driscoll, F. G. 1986. Ground water and Wells. St. Paul, Johnson Divison, Minnesota, 1089 p.)

6.2.1.2 Length of Pump Housing

The actual length of the pump housing is primarily determined by the required depth of the pump. The location of the pump depends on 1) the expected depth to which the water level inside the well will drop for the selected design discharge rate, 2) the procedure to determine the maximum expected water level depth inside the pumped well, 3) the expected drawdown of the water level inside the well should be determined that includes the aquifer losses and the well losses as presented in Fig. 6.4) the drawdown S_1 corresponding to the linear aquifer loss can be expressed as given in Eq. 6.1:

$$S_1 = B_{1(rw,t)} Q \quad (\text{Eq. 1})$$

where

B_1 : is the linear aquifer losses coefficient in day/m²

Q : is the pumping rate in m³/hr.

B_1 can be calculated using the result of long term pumping test in which the Transmissivity T and storativity S can be used to calculate the B_1 values as a function of r_w and t . Well losses are divided into linear and nonlinear head losses. Linear well losses are caused by damaging the aquifer during drilling and completion of the well. They comprise, for example, head losses due to the compaction of the aquifer material during drilling; head losses due to plugging of the aquifer with drilling mud, which reduces the permeability near the bore hole; head losses in the gravel pack; and head losses in the screen. The drawdown S_2 corresponding to this linear well loss can be expressed as given in Eq. 6.2.

$$S_2 = B_2 Q \quad (\text{Eq. 6.2})$$

where

B_2 : is the linear well losses coefficient in day/m^2 .

Among the nonlinear well losses are the friction losses that occur inside the well screen and in the suction pipe where the flow is turbulent, and head losses that occur in the zone adjacent to the well where the flow is usually also turbulent. All these losses responsible for the drawdown inside the well are much greater than one would expect on theoretical grounds. The drawdown S_3 corresponding to this nonlinear well loss can be expressed as given in Eq. 6.3.

$$S_3 = C Q^P \quad (\text{Eq. 6.3})$$

where C is the nonlinear well loss coefficient in $\text{day}^P/\text{m}^{3P-1}$, and P is an exponent. The general equation describing the drawdown in a pumped well as function of aquifer/well losses and discharge rate thus reads as given in Eq. 6.4.

$$S_w = (B_1 + B_2)Q + C Q^P = BQ + C Q^P \quad (\text{Eq. 6.4})$$

where $S_w = S_1 + S_2 + S_3$.

Jacob (1947) used a constant value of 2 for the exponent P . According to Lennox (1966), the value of P can vary between 1.5 and 3.5. The value of $P = 2$ as proposed by Jacob is, however, still widely accepted. Values of the three parameters B , C , and P can be found from the analysis of step-drawdown tests.

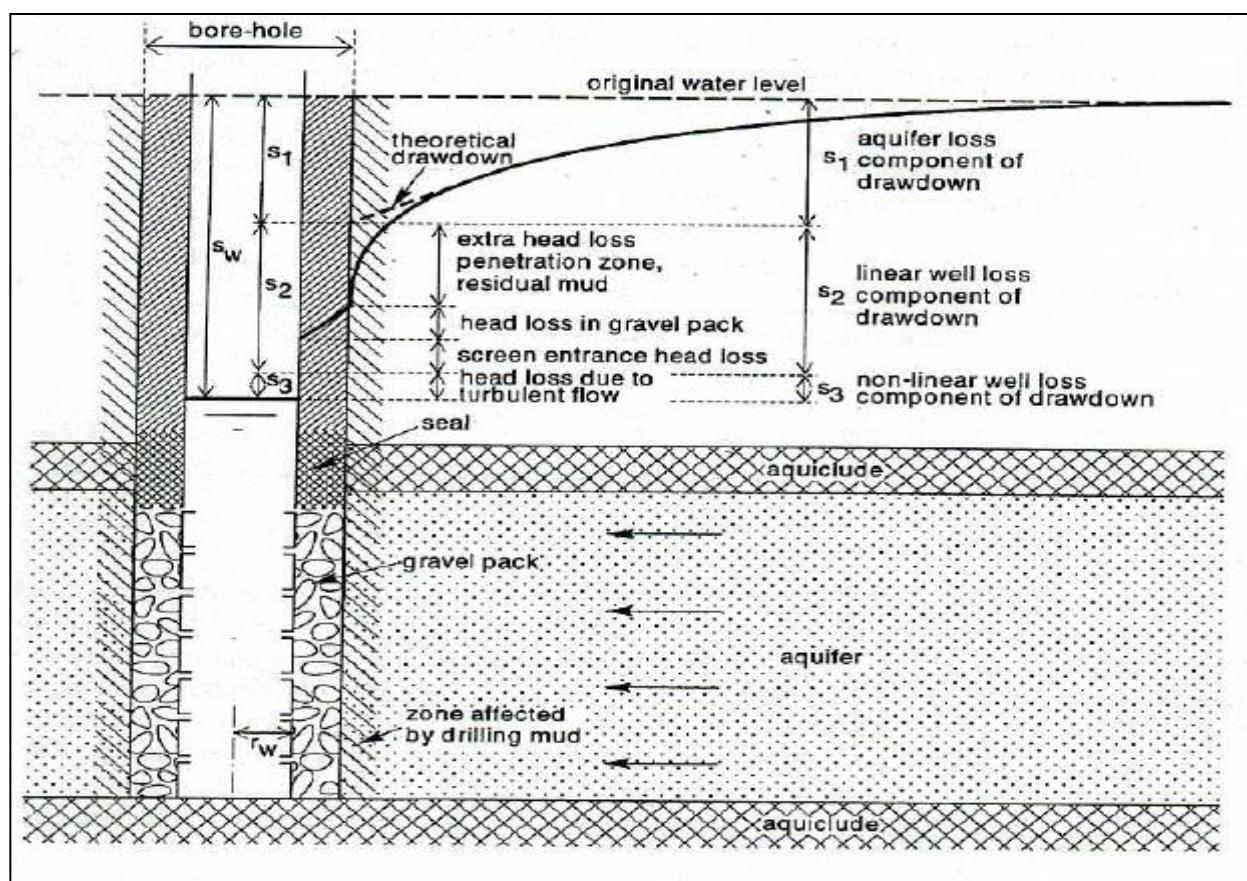


Fig. 6.4: Various components of head losses in a pumped well (Delleur, 2007).

6.2.1.3 Screen Section

Important properties of the screen are that it prevents sand and fine material from entering the well during pumping, has a large percentage of open area to minimize the head loss and entrance velocity, supports the wall of the well against collapse, and is resistant to chemical and physical corrosion by the pumped water. The screen includes the following characteristics.

1. Screen Material: PVC and fiberglass screens are lighter and more resistant to corrosion by chemically aggressive water, but have lower collapse strength than steel screens and casings. In practice, PVC and fiberglass-reinforced screens and casings will be technically and economically attractive for wells in alluvial aquifers, where wells are placed at moderate depths of up to 400 m. Steel screens are required in deep wells drilled in hard rock aquifers. Stainless steel screens combine both strength and resistance to corrosion and chemically aggressive water, but are more expensive. In the current project, taking into consideration the special requirements, the project sensitive nature and the chloride concentration in the ground water, stainless steel screens will be used. The extra cost will be compensated by the better durability and the saving in screen length where larger opening percentages (reduced screen length) can be used in this case.

2. Screen slot size: The selection of the screen slot size depends on the type of aquifer and the use of a gravel pack. The screen slot size must be selected to ensure that most of the finer

materials in the formation around the borehole are transported to the screen and removed from the well by bailing and pumping during the well-development period immediately after the borehole has been constructed and the screen and casing have been installed.

3. Screen length: The screen length should be chosen so as to ensure that the actual screen entrance velocity is in accordance with the prescribed entrance velocities as listed in Table 6.6. From these screen entrance velocities, the minimum length of the well screen can be calculated from Eq. 6.5.

$$Q = 86400 V_e L_{\min} A_0 \quad (\text{Eq. 6.5})$$

where

- Q: the discharge rate of the well in m³/day
- V_e: the screen entrance velocity in m/s
- L_{min}: the minimum screen length in m, and
- A₀: the effective open area per meter screen length in m²/m.

In determining the effective open area per meter screen length, it is often assumed that 50% of the actual open area is clogged by gravel particles (Huisman, 1975). The actual open area per meter screen length depends on the type and diameter of the selected screen type. Conventional slotted screens have open areas not exceeding 10% in order not to weaken the column strength, whereas more expensive continuous slot screens of stainless steel or modern PVC screens could have an open area of up to 30 to 50%. In current project, stainless steel open screen area of 30% will be used.

So the minimum total screen length is determined by the maximum screen entrance velocity and the actual screen type. The optimum length of the screen may differ from its minimum length. Determining the optimum screen length is rather complex; it depends on the following factors:

- (1) All the cost factors that determine the costs of pumping the required discharge.
- (2) The total thickness of the aquifer. In very thick aquifers, which is not the case of current project, the deeper penetration of the well will result in a smaller drawdown, which reduces the pumping costs but increases the investment costs in the borehole; and
- (3) The selected pumping rate.

The total length of the required screen section is found by adding to the actual screen length, as outlined above, the total length of sections of blind (unperforated) pipe used to case off unproductive layers in the aquifer. The total length of blind pipe depends on the distribution of hydraulic conductivity in the aquifer (i.e., the distribution of layers of higher and lower hydraulic conductivity). This stratification can be determined from the driller's log, geophysical logs, and sieve analysis.

Table 6.6: Recommended screen entrance velocities.

Hydraulic Conductivity of Aquifer (m/d)	Screen Entrance Velocities (m/s)
>250	>0.03
250-120	0.03
120-100	0.025
100-40	0.02
40-20	0.015
< 20	< 0.01

6.2.1.4 Gravel Pack

The effect of gravel-packed wells is to ensure that the zone around the well screen is made more permeable by removing some formation material and replacing it with specially graded material. This relatively narrow zone separates the screen from the formation material and increases the effective hydraulic diameter of the well.

A gravel pack is chosen to retain most of the formation material; ***a well screen opening is then selected to retain about 90% of the gravel pack after development.*** Gravel pack material should ideally be clean, rounded, siliceous sands or gravels; carbonate material, shale particles, or soluble material such as gypsum should not exceed 5% of the total. Gravel pack material should be well sorted to assure good porosity and hydraulic conductivity of these materials around the screen.

The gravel pack is designed on the basis of sieve analyses of aquifer samples. If aquifer samples from different depths show considerable variation in gradation, the gravel-pack design should be based to be stable against the finer-grade samples. Numerous investigators and agencies have experimented to develop formulae or criteria that will result in a stable gravel-pack gradation. According to Anderson (1995), the following criteria have generally been found satisfactory in actual practice and thus will be used in current project. It should be mentioned that, actual selection of gravel pack will be reevaluated during the construction stage in accordance with existing soil profile at each well.

1. ***Aquifer material with uniformity coefficient less than 2.5:*** Use uniform gravel-pack material with a uniformity coefficient less than 2.5 and with the D_{50} of the gravel pack 4 to 6 times the D_{50} of the aquifer. If uniform gravel pack is not available, use a gravel pack with uniformity coefficient between 2.5 and 5 and with the D_{50} of the gravel pack not more than 9 times the D_{50} of the aquifer.
2. ***Aquifer material with uniformity coefficient between 2.5 and 5:*** Use uniform gravel-pack material with uniformity coefficient less than 2.5 and with the D_{50} of the gravel pack not more than 9 times the D_{50} of the aquifer. If uniform gravel pack is not available. Use a gravel pack with uniformity coefficient between 2.5 and 5 and with the D_{50} of the gravel pack not more than 12 times the D_{50} of the aquifer.
3. ***Aquifer material with uniformity coefficient greater than 5:*** Multiply D_{30} of the aquifer by 6 and 9 and locate these points on the sieve analysis graph. Draw two parallel lines

through these points having a uniformity coefficient of 2.5 or less, and specify gravel-pack material that will fall between these lines.

In the current project and in accordance with soil profiles in the area, case 3 above is expected to control the design as explained in the design section in this report.

Although smaller thicknesses already fulfill the objective of a gravel pack, the thickness of a gravel pack should at least be 76 mm to ensure that a continuous layer of filter material will surround the entire screen. Under most condition, the upper limit of gravel-pack thickness should be about 200 mm because the energy created by the development procedure must be able to penetrate the pack to repair the damage done by drilling, break down any residual drilling fluid on the borehole wall, and remove finer particles near the borehole (Delleur, 2007).

6.2.1.5 Well Development

The principal purpose of well development is as follows:

1. to remove the fine materials adjacent to the well bore,
2. to increase porosity and hydraulic conductivity of the aquifer and gravel pack,
3. to remove any mud cake or compacted zone that results from the actual drilling, and
4. to minimize or eliminate sand pumping.

Upon completion of drilling, most wells require development to reach maximum efficiency. This is particularly true of wells producing from unconsolidated aquifer material sand those in which an artificial gravel pack has been placed around the well screens. In addition, many wells may require periodic redevelopment to restore production capacity that has been lost as a result of such factors as encrustation of screens, clogging of screens by fine particles into a gravel pack. The following discussion summarizes some developments and procedures.

The method of removing finer material from water-bearing formations is by over pumping, that is, pumping at a higher rate than the well will be pumped during exploitation. Over pumping, by itself, seldom produces an efficient well because most of the development action takes place in the most permeable zones dose to the top of the screen. The same applies to a certain extent to surging/backwashing. It consists of pumping a well at a high rate for a short period, shutting down the pump to allow water in the column to fall and backwash the screen, and then repeating the process until the discharge is clear. Although over-pumping and backwashing techniques are widely used, and in certain situations may produce reasonable results.

6.2.1.6 Minimizing Maintenance

The performance of a well usually declines after some years of operation, resulting in higher drawdowns and higher pumping costs. The well is in need of rehabilitation when the specific capacity of the well becomes so small that the pumping costs increase or the discharge rate of the well can no longer be maintained. Before that time, the well needs to be rehabilitated. An effective well-maintenance program begins with good records being kept of the well's construction, including good records of the geological conditions, the position and types of

aquifers and aquicludes, water quality, and the specific capacity of the well, determined during well testing.

Every type of well requires its own maintenance program. Driscoll (1986) provides a checklist to evaluate the performance of well. The major causes of a reduction in well performance are:

1. A reduced well yield due to chemical encrustation or clogging of the screen due to bacteriological activity;
2. Plugging the formation around the well screen by fine particles of clay and sand in the pores;
3. Pumping of sand due to poor well design or corrosion of the well screen;
4. Collapse of the well screen due to chemical or electrolyte corrosion of metal well screens.
5. Deterioration of pump impellers due to for example the existence of high level percentage of sand in the pumped water.

The use of hypochlorite is a relatively safe and convenient alternative to chlorine gas. The occurrence of iron bacteria in wells can be prevented by disinfecting the well and the pump immediately after installation.

Physical plugging by clay and silt particles can best be prevented by proper well development after the well screen has been installed. The removal of fine particles from the formation immediately around the screen can best be achieved by washing and brushing the screen with dispersing compounds such as sodium tripolyphosphate (STP) and other types of polyphosphates.

Sand pumping causes the abrasion of pump bowls, which leads to failure of the pump. Sand pumping results from over-sized slots in screens, over-sized gravel pack, corrosion of the well screen, inadequate development of the well or too-high entrance velocities, causing the transport of sand from the formation toward the well. One of the above conditions, or a combination of them, results in sand from the formation entering the well. Remedying this problem may be uneconomical: it may be better to drill a new well. The best alternative, if possible, is to replace the screen or to place an inner screen inside the original well screen.

Corrosion of well screens can severely reduce the lifetime of a well. Chemical corrosion occurs especially when metal well screens are used in aggressive and saline water loaded with gases like hydrogen sulfide, carbon dioxide, and oxygen. Corrosion can be prevented by applying nonmetal screens or, when the water is not aggressive, only metal screens of stainless steel and low-carbon steel. As mentioned earlier screens of stainless steel will be used in this project.

Finally, to pump water from a well in the most economical way, proper maintenance of pumps and engines is a prerequisite. Pump and engine manufacturers prescribe periodical maintenance of their products. Maintenance procedures depend on the pump type. They include the adjustment and replacement of impellers, bearings, stuffing boxes, and bowl assemblies. A

complete analysis of pump and engine maintenance is beyond the scope of this chapter, so readers are referred to the maintenance procedures specified by manufacturers.

Complete well records can be kept at relatively minor expense, and these are indispensable in determining the causes of well failure and selecting the maintenance and rehabilitation program.

A comprehensive maintenance program for the current project will be developed after the design stage.

6.2.2 Collection Pipes from Wells to Water Tanks and Irrigation Network

This section expressed the design criteria for the two parts of the water network 1) the collection pipes from wells to the water tanks and 2) the irrigation pipes from the booster pumps to the farms. For both types the pipelines will be under pressure.

6.2.2.1 Type of the Distribution System

The pipe network of a distribution system includes facilities to shut off the flow in the pipes, to empty and ventilate the pipes and to regulate the pressure and flow direction. Reservoirs and booster plants may also be included which are considered in separate sections. In the design of a pipe network, consideration should be given to supply reliability and water through-put. The distribution system for the collection pipe from wells and water tanks and the pipes from the booster to the farms is selected as branching system. It is designed so that each point in the pipe network is fed from a single direction Fig. 6.5.

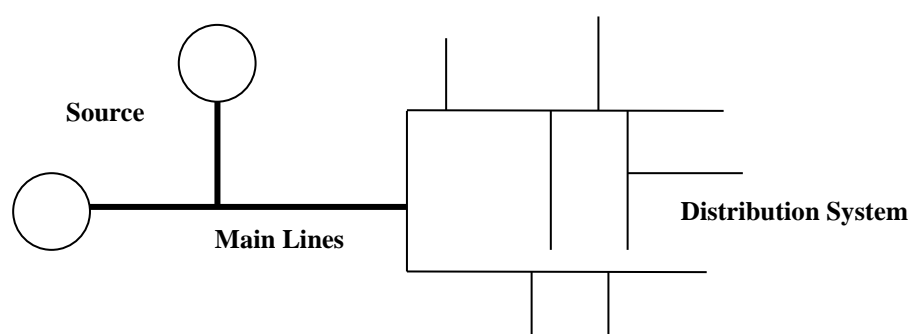


Fig. 6.5: Typical water distribution network designed as a branching system (Delleur, 2007).

6.2.2.2 Pipe Material

There are different types of pipes that can be used to construct a pipeline. The choice of which type to be used should be studied carefully. The following points may be used as guideline criteria for selecting the most suitable type of pipe material.

1. Cost.
2. The project area.
3. Type of soil.
4. Chemical characteristics of the conveyed fluid.
5. The available head.

6. The source of pipes.
7. Environment of the project area where the transmitted water is partially treated waste water.
8. Available experience.
9. Pressure of pipeline.

Many different pipe materials are used for water pipelines. Pipes can be classified into three major categories regarding the raw material used in their manufacturing which are:

1. Metallic pipes; e.g. black steel (ST), galvanized steel (GS), Cast Iron (CI) and ductile iron (DI).
2. Cementitious pipes; e.g. pre-cast concrete (C), pre-cast reinforced concrete (RC), glass reinforced concrete (GRC), pre-stressed and asbestos (AC).
3. Plastic pipes; e.g. Unplasticized Polyvinyl Chloride (UPVC), polyethylene (PE), and polypropylene (PP).

Table 6.7: Comparison between UPVC, Steel and Ductile Iron.

Criteria	UPVC	Steel	Ductile Iron
Capital Cost.	Low	High	Moderate
Operation and Maintenance cost	Low	High	Moderate
Corrosion Control	NA	Difficult	Easier
Chemical characteristics of the conveyed fluid	Not Influenced	Not Influenced	Not Influenced
The source of pipes.	Not Local	Not Local	Not Local
Environment of the project area where the transmitted water is partially treated waste water	Can be used	Can be used	Can Be used
Available experience	High	High	High
Pressure of pipeline	Moderate Resistant	High Resistant	High Resistant
Field Condition	Low Adapted	Moderate Adapted	High Adapted

Table 6.7 includes a summary of a comparison study for the use of the three piping materials, i.e. UPVC, steel and ductile Iron. Based on the aforementioned factors and Table 6.7, a plastic UPVC pipes are recommended in the both networks parts with size less than 600 mm and Ductile iron pipe will be for pipes have a diameter greater than 600 mm. A plastic pipes UPVC have been selected for attention because of a number of physical properties that make its use advantageous over other types. Some of the advantages of plastic pipes are summarized as follows:

1. Favorable initial and maintenance cost compared with other pipes of traditional materials for smaller sizes.
2. Longer length, depending on type and ease of joining reduce jointing costs. It is easy to bend.
3. Light weights resulting in lower handling and transporting costs and make it easier and faster to install.
4. Lower coefficient of friction permitting greater flows through a particular size.
5. Resistance to corrosion and built-up of deposits.
6. Good chemical resistance with non-absorbent walls.
7. Lower modulus of elasticity giving an advantage where there is soil movement or vibration.
8. Good tensile strength.
9. Thermal and electrical insulator.
10. No danger to health (non-toxic) and internationally approved for potable water use and also for stormwater and wastewater.

PVC is commonly used in Finnish water supply systems, and according to experience does not generally relate to any special problems in water quality. For larger sizes, the ductile iron has been selected since it has the same advantageous criteria of the UPVC over steel other steel pipes, such as:

1. It is easier and less expensive to control corrosion on ductile iron pipe than it is on steel pipe, where Ductile Iron Pipe Corrosion Control is accomplished with Polyethylene Encasement.
2. The largest practical advantage of Ductile Iron pipe compared with steel pipe is that Ductile Iron pipe is much easier to install properly. Handling, assembling, backfilling, and adapting to field conditions all are areas in which Ductile Iron pipe offers distinct benefits.
3. Ductile Iron Pipelines Adapt to Field Conditions in Installation more than steel pipes.
4. Since Ductile Iron pipe design results in a thicker wall for a given set of parameters, Ductile Iron pipe is a stiffer product than steel pipe
5. In all normally specified pipe sizes, cement-mortar lined Ductile Iron pipe has an inside diameter that is larger than the nominal pipe size
6. Pumping costs are lower for Ductile Iron pipelines, this reduction in pumping costs will save the system owner significantly over the life of the pipeline.
7. Another aspect of comparing Ductile Iron pipe with steel pipe are the costs associated with operating systems. Protection systems, often a requirement for steel pipelines, involve higher design and installation costs. They require monitoring and maintenance over the lifetime of the pipeline. There are also costs associated with pumping water through a pipeline and these costs are directly related to pipe inside diameters.

6.2.2.3 Pressure and Head Loss

The range of water pressure experienced at any location is a function of the hydraulic grade and the service elevation within a specific pressure zone. The hydraulic grade is affected by the pump setting or reservoir water level, pressure reducing valve setting, and friction losses in the distribution system.

The collection pipeline should be designed using a minimum pressure at the outlet of the pipe (in the water Tanks) equal to 1 Bar. The irrigation pipelines should be designed for a minimum pressure in the farm gate equal to 2.5 Bar.

The piping system is designed by considering the head loss or pressure drop that occurs when transporting flows from one point to another. Friction losses through pressure piping are based on the Hazen–Williams formula (Eq. 6.6):

$$V = 0.849 C R^{0.63} S^{0.54} \quad (\text{Eq. 6.6})$$

where

V = velocity (m/sec).

C = roughness coefficient

R = hydraulic radius (m).

S = friction head loss per unit length.

The size of the pipelines in the recovery network (pipes connecting the wells to the water tanks) was selected based on the pumping rate while the size of the pipelines in the irrigation network was selected based on the pumping rate of the booster pumping station. The pipelines were designed using Water Cad Software V.8.0. The pipe roughness coefficients that will be used in the hydraulic model build up are presented in Table 6.8.

Table 6.8: Pipe Roughness Coefficients

Pipe Material	New Pipe	Old Pipe
PVC, UPVC	150	130
PE	150	130
Steel (cement lined)	150	120
Asbestos, Cement	140	130

Source: Adopted from Heastad WaterCad manual (2003).

6.2.2.4 Velocity

Based on the pressure designed value the sizes of the pipelines are determined based on minimum velocity that water should flow at all times, with sufficient velocity to reach the target point with enough pressure head. During the peak flow period, the minimum velocity should not be less than 1.5 m/sec. Maximum velocities are usually limited to about 3 m/sec. In this case a special provision should be made to protect against displacement by erosion and shock.

6.2.2.5 Network Simulation

Hydraulic network simulation models are widely used by planners, water utility personnel, consultants, and others involved in the analysis, design, operation, and maintenance of closed-conduit hydraulic systems. The results of network models have been used to assist the design of the collection pipelines and the irrigation network.

The consultant has developed a computer model, using the WaterCAD V8 XM from Bentley Ltd software, for the water distribution pipelines in both parts. This internationally used modeling software is convenient program for steady-state as well as above mentioned dynamic approach in design of irrigation network system. The computer model developed for the hydraulic analysis of sizing the transmission mains and distribution mains of the irrigation networks. This hydraulic model was built in compliance with the planning bases and design criteria that were set in this design report. The developed model is considered as a key tool for analysis; design, planning, operation, and maintenance, therefore it will be used later by PWA staff and will play an important role in keeping the operation of the system. In our study, two hydraulic models were developed one for the collection pipes from the wells to the tanks and the second form the booster pumps to the farms. Figs. 6.6 and 6.7 show the schematization of the two models.

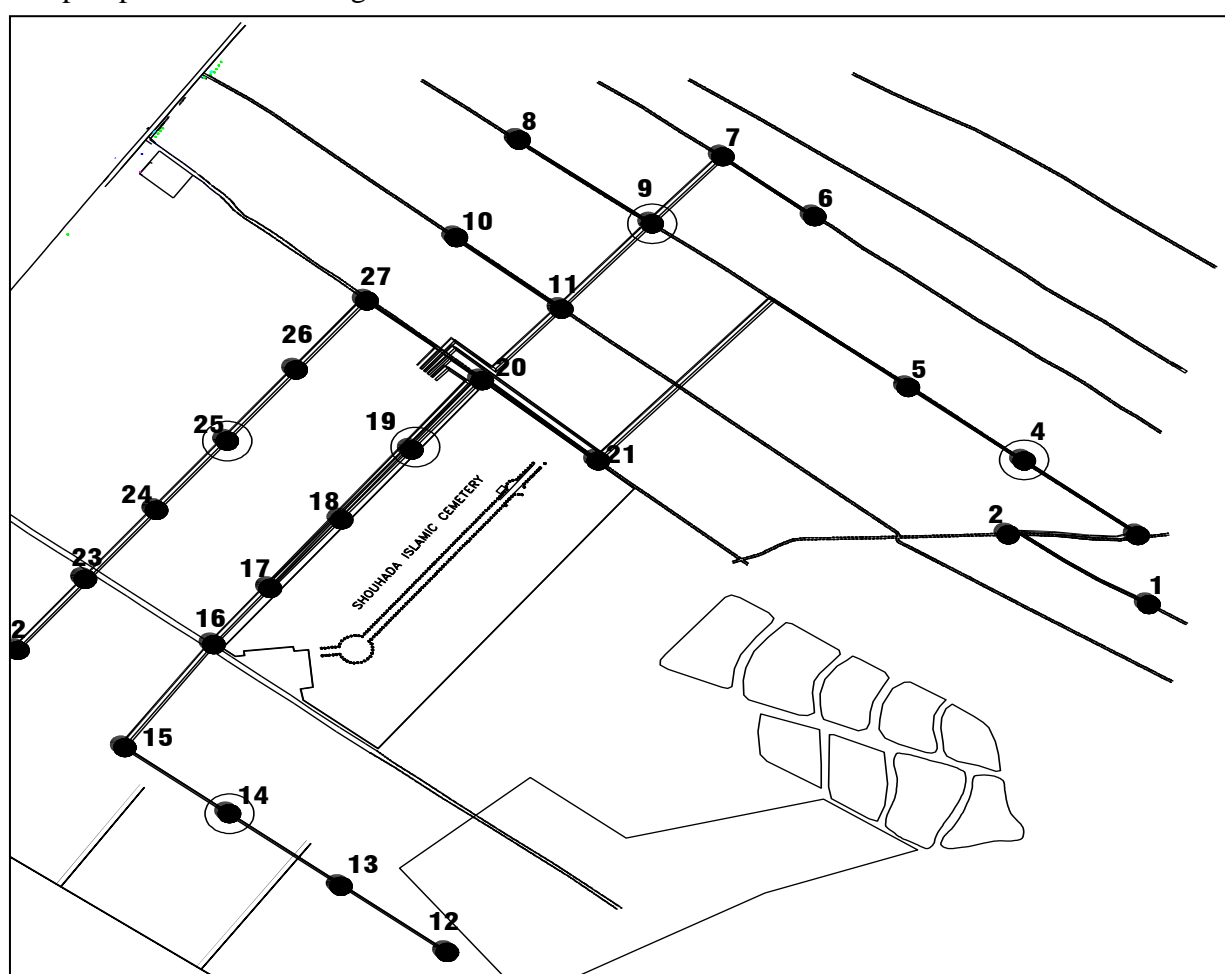


Fig. 6.6: Schematization of piping system in the first model (Recovery wells to the Tanks).

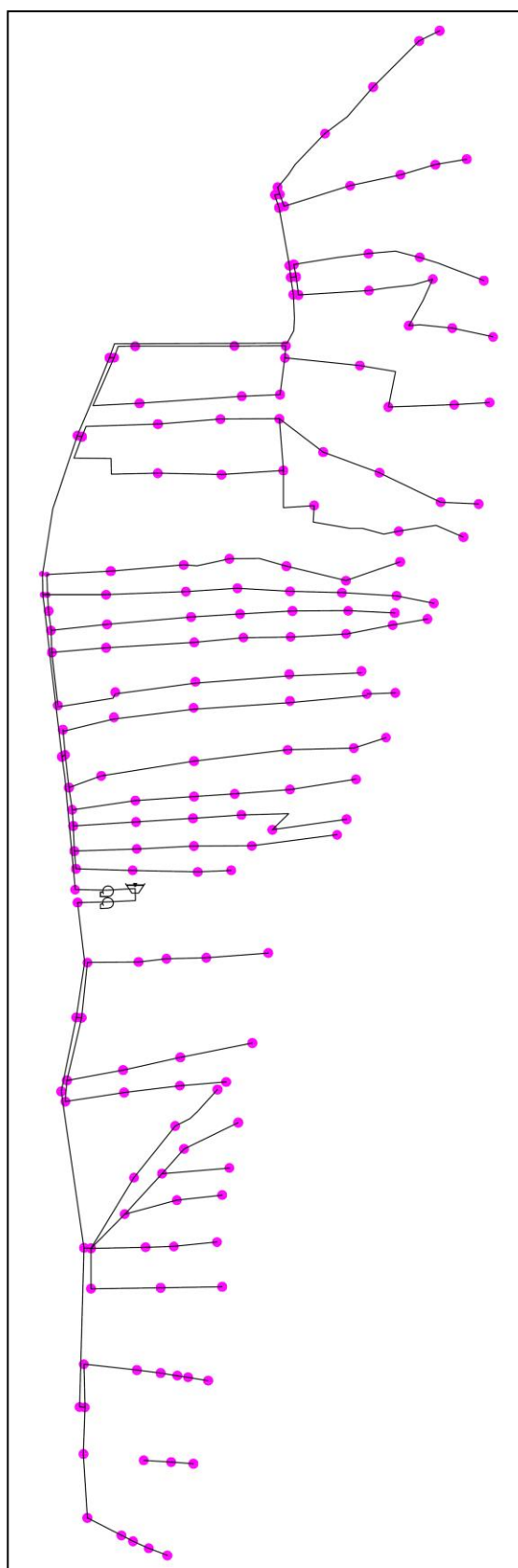


Fig. 6.7: Schematization of piping system in the second model (Irrigation Network).

The hydraulic modeling of irrigation system network is carried out for the analysis of variable operational situations such as full and partial use of pumps during the high and low water tables in new recovery and existing agricultural wells.

Dynamic approach of modeling is used to test normal and ad-hoc operation on designed irrigation system network in 24 hour period. It is possible to pay attention to time and distribution area related transient water flow to farm gates. Moreover it gives preliminary design parameters to new facilities such as recovery wells, collection water network, storage reservoir as well as booster pumping station required for appropriate operation and maintenance in designed network system.

Construction of irrigation network modeling consists of following five steps:

1. Identification of main components.
2. Skeletonization of collection and irrigation networks (length, diameters, materials and wall thickness of the pipes, roughness).
3. Characterization of well pumps and booster pumps (type, QH-curves) as well as storage reservoir (volume)
4. Operational data.
 - a) Control of well and booster pumps.
 - b) Volume curve of water storage reservoir.
 - c) Operation time of pumps.
 - d) Water distribution areas.
5. Running of dynamic model (eg. 24 hour run with interval of 15 min).

As a result of hydraulic modeling, optimal operation of new and old pumps as well as water storage reservoir can be calculated for irrigation system network. The maximum and minimum flow and pressure can be modeled in every pipe and junction of irrigation network. Moreover, the results will be used as initial data for different operational situations and 24 hours simulations in order to guarantee adequate amount of water for agricultural needs.

The process of developing the model is initiated with skeletonization process to distribution system from the recovery wells to the tank. Another model was made from the tanks to the farms. Piping system in the two models consists of trunk lines, main feeders. The skeletonization percentage according to pipe diameter has been preceded as shown in Tables 6.9 and 6.10 for the first and second models, respectively.

Table 6.9: First model skeletonization from the recovery wells to the tanks.

DN(mm)	%
225	33.8
280	11.3
315	5.0
355	10.7
400	30.8
450	8.4

Following the system skeletonization, 37 nodes have been assigned throughout the system of the first model in order to represent demand values and/or to fit with system configuration.

Table 6.10: Second model skeletonization from the tanks to the farms.

DN(mm)	%
110	0.4
140	0.7
160	3.5
225	12.4
280	7.7
315	5.5
355	3.8
400	8.4
450	6.3
500	6.0
630	16.9
710	10.0
800	0.3
900	18.2

Following the system skeletonization, 240 nodes have been assigned throughout the system in order to represent demand values and/or to fit with system configuration.

Starting from 2013 target year, the following assumptions have been considered in developing the hydraulic model:

1. The distribution system fixed points are the ground tanks.
2. The two systems are simulated separately as they will work separately.
3. In the second model, the area is divided into 6 areas and numbered as (A1,A2,B1, B2,C1,C2,D,E,F) where each area will receive 5500 m³/hr as the pumping capacity of the designed booster pumps.
4. The ability of the system to meet demands has been analyzed based on an extended period continuous flow (12 hours) simulation taking into consideration the effect of demand fluctuation during the day. Which means the average demand for each node is multiplied by demand factor corresponding to each hour of the day. The demand patterns for the irrigation areas are shown in section 6.1.4
5. The collection pipelines were tested for two scenarios where the pumping rates of the wells were 170 m³/hr and 200 m³/hr respectively.
6. The irrigation network was tested using three scenarios, the maximum pumping rate is 6000 m³/day, the minimum pumping rate is 2100 m³/day and the expected pumping rate which was 5600 m³/day

The hydraulic analyses characteristics are summarized as follow:

- Analyses : Steady State simulation
- Friction method: Hazen-William formula
- Accuracy: 0.001
- Trials: 40
- Starting time: 12.00 AM
- Duration: 24 hours

6.2.3 Monitoring Wells

6.2.3.1 Monitoring Strategy and Plans

Before preparing a groundwater monitoring plan, the overall strategy of the groundwater monitoring program should be defined to guide the development of the plan. In this sense, “strategy” refers to the manner in which a hypothetical release from a regulated unit will be detected or measured. Examples of issues that should be addressed when developing a monitoring strategy include:

1. The type of monitoring data needed;
2. The locations (both horizontal and vertical) from which the samples are to be collected (i.e., definition of “target monitoring zones”);
3. The manner in which the samples will be obtained; and
4. The ability of the monitoring features to rapidly detect a change in groundwater quality.
For detection monitoring programs,

The types of data needed are usually defined by regulation; for other types of monitoring programs, the types of data needed are typically based on site-specific considerations.

Development of a groundwater monitoring strategy is illustrated in Figs. 6.8 and 6.9. As shown in these figures, the potential sources of contamination and the aquifers of concern should be characterized before developing a groundwater monitoring strategy because selection of target monitoring zones cannot be made until the source and the aquifer of concern have been evaluated, usually through a detailed hydrogeologic evaluation of the site. When evaluating the ability of a monitoring system to rapidly detect a release from the potential source, the impact of preferential flow paths and vertical gradients should be carefully evaluated; a two-dimensional analysis of groundwater elevation may not reveal actual upgradient or down gradient locations of groundwater flow. The presence of vertical gradients may significantly affect the selection of monitoring locations which is the case of the current.

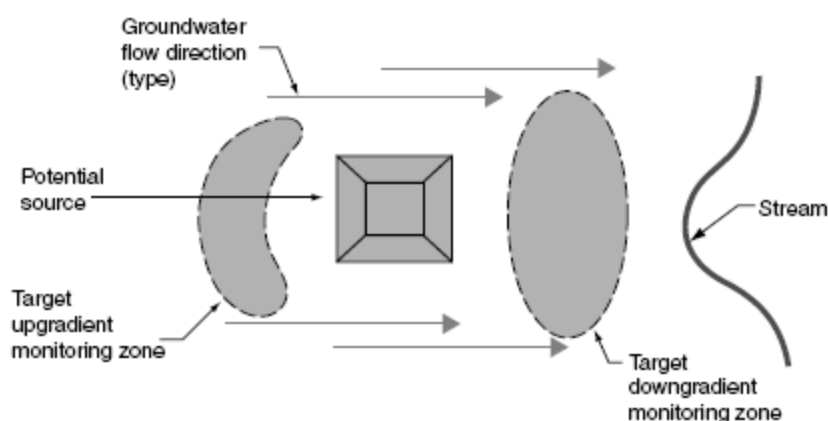


Fig. 6.8: Plan view of typical unconfined aquifer groundwater monitoring system.

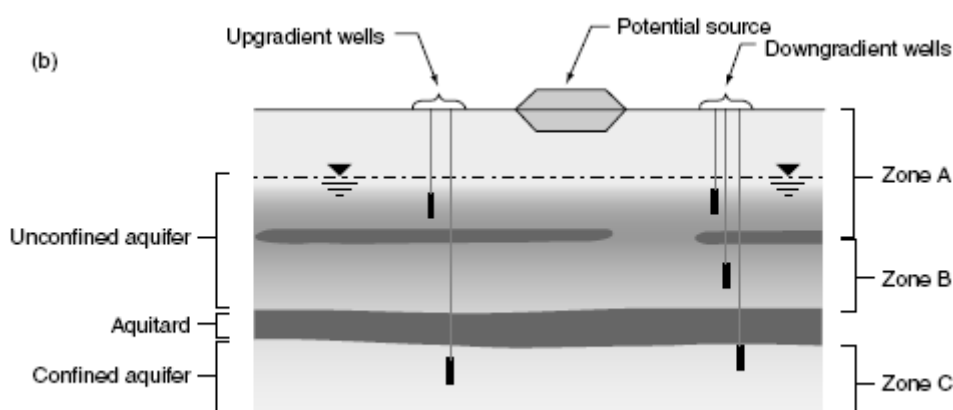


Fig. 6.9: Vertical cross section of target monitoring zones.

6.2.3.2 Design of Monitoring Wells

A three-phased procedure for designing a groundwater monitoring system is described as follows.

I: Select Monitoring Locations: Locating the appropriate monitoring point locations is essential in designing a monitoring network capable of providing data of adequate quality to achieve the program objectives. At times, monitoring well locations may be prescribed by the regulations under which the groundwater monitoring program is being developed. For example, some regulations require monitoring Locations be placed at a designated “point of compliance,” which is often at the property boundary or a groundwater discharge location. For other groundwater monitoring programs, the groundwater professional should select monitoring locations that provide the most reliable data needed to detect or assess a groundwater contaminant plume. To verify that the monitoring network can accomplish this goal, target monitoring zones must be selected based on the site hydrogeologic conditions and anticipated contaminant pathways, which have been discussed in Section 4 (Groundwater Modeling) in this report.

Examples of monitoring well location layouts for a detection monitoring program in both a typical unconfined and layered aquifer system are provided in Figs. 6.8 and 6.9. As shown in

these figures the locations and orientation of confining units have a significant effect on potential contaminant migration paths and therefore the vertical spacing of monitoring wells. Additionally, the physical and chemical characteristics of the contaminant must also be considered when identifying target monitoring zones and selecting monitoring locations. To facilitate the selection of monitoring locations, numerical groundwater flow models with particle tracking capabilities can be used to predict contaminant migration pathways and identify potential target monitoring zones which is followed in the current project as mentioned in section 4.7.

II: Select Monitoring Devices: Appropriate monitoring devices should be selected for obtaining the required samples or data from the target monitoring zones. Groundwater monitoring programs most often incorporate monitoring wells, piezometers, and groundwater discharge features as monitoring points as the case of the current project.

III: Design the Monitoring Features: Finally, after the monitoring features have been identified, they should be designed to meet the specific goals of the monitoring program and to provide accurate, representative samples of groundwater. The purpose of a groundwater monitoring well is to provide access to the target monitoring zone for collection of a representative sample of groundwater. The representativeness of the sample may be affected by installation of the well or by the materials used to construct the well; the design of the well must account for these factors. In this section, groundwater monitoring wells and their applicability are described. The discussion presented in this section should be considered to be a general guide; site-specific conditions and applicable regulatory requirements should be considered over these guidelines when designing a groundwater monitoring well.

Standard approaches for design of groundwater monitoring wells are presented by a number of agencies and organizations, including the USEPA (1991a, 1992a) and the American Society for Testing Materials (ASTM, 1995). Examples of typical groundwater monitoring well designs are presented in Fig. 6.10. The design shown in Fig. 6.10 incorporates several features that minimize the possibility of introducing contaminants into the well (e.g., the protective cover, the bentonite seal, and the well apron). These designs can be modified as needed to meet site-specific conditions or regulatory requirements. For example, the number of monitoring points can be increased by installing multiple, discrete sampling points within a well; also, uncased, open boreholes can be used to monitor bedrock aquifers where migration of soil particles into the well is not expected to occur.

Some of the key features of the groundwater monitoring well, shown in Fig. 6.10, are the well screen, filter pack, bentonite seal, cement grout backfill, concrete apron, and protective cover. The most important aspect of monitoring well design is the proper sizing and placement of the well screen or open-interval.

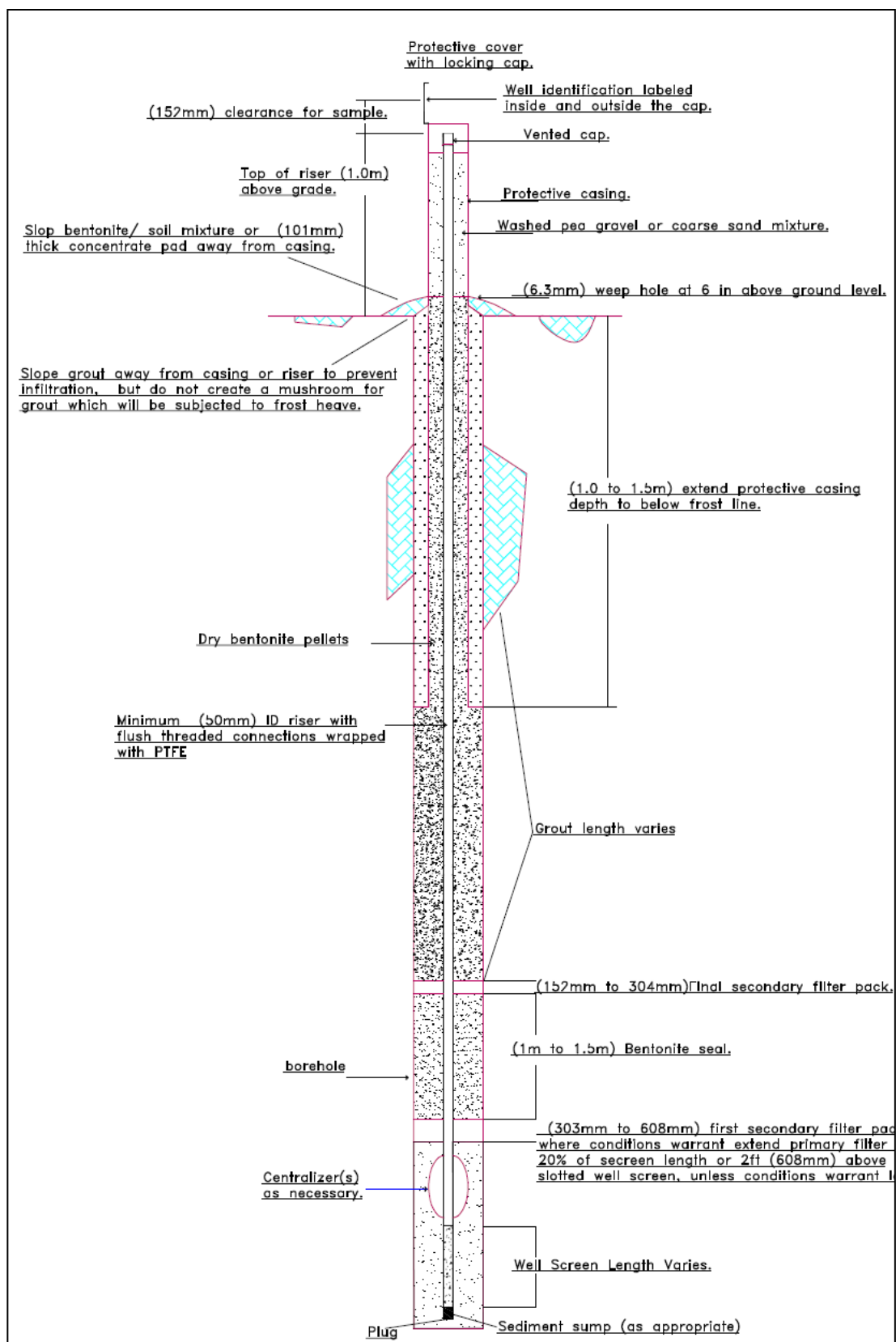


Fig. 6.10: Typical monitoring well design.

When sizing the well screen, both the screen-interval size (i.e., slot size and screen type) and screen length for the proposed monitoring well must be considered. The screen-interval of the monitoring well screen should be sized based on the geologic materials outside of the screened interval and the proposed filter materials. The well screen or open-interval length should be limited to the target monitoring zone. Monitoring well screen lengths typically range from 2 to 10 ft (EPA, 1991) and CAMP project screen length was 5 m. To the extent possible, the screen length should be minimized to avoid dilution in the screened zone and to minimize interactions with, and potential contaminant migration to other zones within the aquifer. Also, as previously discussed, some regulatory agencies prescribe well design requirements particularly screen or open interval dimensions. The filter pack is intended to promote formation of a graded filter outside of the well to prevent migration of fine-grained soils into the well. This is because soil particles are composed of minerals that may be constituents of concern, the presence of fine-grained soils in a well, which is the case of current project, can cause inaccurate groundwater monitoring results, as well as clog the well.

The filter pack material should also have a characteristic particle size (i.e., the diameter greater than 85%, by weight, of the soil particles) that is bigger than the well screen slot size to prevent clogging of the well screen by the filter pack material. Similarly, the filter pack material should be capable of retaining the coarsest 15% of materials in the adjacent geologic formation. The bentonite seal is intended to prevent the cement grout backfill from migrating into the filter pack; the presence of grout in the filter pack could permanently compromise the validity of groundwater samples from the well. The concrete apron is intended to route surface water away from the well and to prevent downward migration of surface water into the well screen. The protective cover is intended to prevent unauthorized access to the well and to protect the exposed portion of the riser pipe from damage due to incidental contact. When installing a groundwater monitoring well, the following potential problems should be anticipated and avoided to the extent possible (Nielsen 1991; USEPA, 1993c):

1. Use of well construction materials that are physically or chemically incompatible with either the surrounding natural earth materials or contaminants in the target monitoring zone and strong enough to prevent collapse under the stress applied by the soil.
2. Improper selection of well screen sizes (screen sizes that are too large may allow siltation of the well, and screen sizes that are too small may prevent proper development of a graded filter around then well).
3. Placement of the screened interval of the well across stratigraphic zones, if the intent of monitoring is to sample discrete zones of the aquifer (this problem could also limit the use of the well for hydraulic conductivity testing of the aquifer).
4. Improper selection or placement of filter packs material that could cause either siltation of the well or plugging of the well screen.
5. Improper selection or placement of annular seal materials that can allow plugging of the filter pack, cross-linking of discrete water-bearing units, or migration of grout into the filter pack.
6. Poor surface-protection measures that can allow damage to well casing materials or introduction of surface water into the well at the ground surface.

7. Poor well development and evacuation techniques that may alter the aquifer formation around the well screen, cause excessive siltation of the filter pack and groundwater samples, or compromise well yield.

6.2.4 Water Tanks

The main reason for construction water tank (reservoir) is to store fluid. The accepted rule for sizing a tank is the returned fluid theoretically will have two to three minutes in the tank before it circulates again. A baffle separates the return line from the pump inlet line, forcing the fluid to take the longest possible path through the reservoir before returning to the pump inlet. This arrangement also mixes the fluid well and provides more time to drop contaminants and de-aerate. In addition, the fluid spends more time in contact with the outer walls of the reservoir to dissipate heat.

The hydraulic design of the water tanks consists of determining the volume of the tank and the hydraulic dimensions of the piping system in the tank site. Based on the study concerning the demand for irrigation water for the system design, two water tanks of 4000 m³ each is used (controlled by Zone F during the peak summer month of June). The piping system may include the inlet manifold and the manifold connecting the tank with the booster pump stations.

6.2.5 Booster Pumping Station

Conceptual design of the pumping station was presented to supply recovered wastewater for the farmers to fulfill their irrigation demand via efficient variable speed pumps. The design has following features: a simple operation and reduced operation costs, simple modification of system parameters, automatically cycles lead pump position for even use, automatically starts and stops lead and lag pump to meet demand, automatically stops all pumps when system demand is zero, dry run overload protection (run out protection, and running and fault signals for motor temperature).

Criteria and Guidance referred below is provided by PWA for sizing and selection the booster pumps. This guidance shows the criteria for selection of pumps and pump drives, piping, control valve, procedures for determining pumping station location and future flow metering, pump station structures, operation demand requirements, and locating permanent pumps tonal features. So that there will be a positive head on pump.

In the current project the pumps will supply water to the irrigation zones; therefore, the booster pumps will generate a nominal discharge pressure. This pressure is required to overcome the head loss of the complex distribution system and to provide the required irrigation water supply pressure to the pumping stations.

The booster pumping system provides water to the farms (consumers) for irrigation process functions. The system is designed for irrigation service, with two intermediate tanks with capacity of about 4000 m³ each to supply the pumps that pressurize the water distribution system. When water is connected to the consumers, the storage tanks are kept full and the inflow from the recovery wells main lines to the tanks is regulated by a liquid level control system in the tanks.

6.2.5.1 Site Selection

Experience has shown that a pumping station should be located or sited with storage tank in such a manner as to produce the most direct possible inflow. Several analyses should be made to investigate alternative piping arrangements within the distribution system as well as for connecting proposed pumping stations to the distribution system.

Consideration of local soil conditions and characteristics may affect the proposed location of a pump station. The hydraulic designer should coordinate the site planning with a geotechnical engineer to ensure that the proposed location is not likely to encounter significant problems associated with water table levels, soil bearing capacities, plasticity, and seismic activity. The following consideration should be taken for site selection:

1. Land availability and relative property values.
2. Topography of service zone: For large distribution system design a pressure contour map is developed using known topography and the hydraulic network analysis.
3. Head provided by intake source.
4. Geology of proposed sites.
5. Site Access: Pump stations require frequent inspection and maintenance. Therefore, provisions should be made for easy access to the station and so that the station is compatible with the number and size of vehicles and hoisting equipment that will likely be required to construct and maintain the station. Such provisions should include:
 - i. Service road/driveway with suitable turning radii.
 - ii. off-street parking,
 - iii. station loading area,
 - iv. turn around area,
 - v. space for heavy lifting equipment, and
 - vi. Roadside warning signs.
6. Site Drainage: A primary consideration for the design of a pump station is the drainage. Therefore, the designer's goal should be to protect all facilities from damage. The primary means use gravity storm drains as deep as practicable to drain as much surface area as practicable, use retaining walls, where practicable, and prevent offsite runoff from flowing to pump station.
7. Security: A pump station is often an attraction for children and vandals. The site should be protected both during and after construction. The primary security measures are:
 - i. perimeter fencing,
 - ii. intruder alarms,
 - iii. concrete or masonry housing, and
 - iv. locked louvered windows
8. Safety: Safety must be a primary consideration for all pump station design and should include provisions for:
 - i. Construction personnel,
 - ii. Inspection and maintenance personnel,
 - iii. Motorists, and
 - iv. General public.

Provision for adequate access is a primary safety measure for inspection and maintenance personnel. Other considerations include meeting Occupational Safety and Health Administration (OSHA) requirements for station access holes, hoisting, steps, ventilation etc. construction, operation and maintenance for additional discussion of inspection and maintenance considerations.

Primary means of ensuring public safety include:

- i. minimizing traffic hazard by suitable site location
- ii. providing warning signs,
- iii. meeting clear zone requirements for the highway or providing appropriate protection,
- iv. providing adequate security and
- v. Providing failure and high water level alarms.

6.2.5.2 Station Capacity and Flow Rates

The sizing of each component in the distribution system will depend upon the effective combination of the major system elements: supply source, storage, pumping, and distribution piping. As presented in irrigation report in *Appendix I*, the irrigation consumption estimates are the basis for determining the irrigation demand of a design of the distribution system. Flow and pressure demands at any point of the system are determined by hydraulic network analysis. The pump discharge head will be derived from the system pressures at the pump station location plus the pump station piping head loss should be performed based on the peak demand of irrigation through the year.

6.2.5.3 Pump Selection

The booster pump pressurizes the lateral system to provide uniform distribution to the irrigation networks. Variable speed pumps appropriate for demand are designed to operate in the corrosive environment of the system. The pump size is selected based on the system flow rate in m³ per hour and the total dynamic head (TDH). The total dynamic head is determined by adding together:

- i. The elevation difference between the pump outlet and the laterals;
- ii. The head losses in the pipe and fitting; and
- iii. The desired head at the end of the laterals should be at least 2.5 bar.

Using pump performance curves, select the pump that best matches the required flow rate at the operating head. Using the pump performance curve, determine if the pump will produce the flow rate at the required head. When selecting specific manufacturer's pumps and piping the following should be considered:

- i. The pump selection is dependent on the system head curve and power requirements,
- ii. The power requirements are dependent on the total dynamic head requirements,
- iii. The system head curve is dependent on total dynamic head,
- iv. The total dynamic head is dependent on the pump and pipe head losses, and
- v. The head losses are dependent on the selected pumps and piping.

The designer must choose which way to proceed. The method presented here begins by estimating the system curves before selecting manufacturer's products. The assumptions are then checked for validity after selection. A design analysis is prepared in the following section to show head loss and friction calculations for present and future demands.

6.2.5.4 Pump Type

There are generally two types of pumps used for water pumping applications; the vertical turbine pump, line shaft and submersible type, and the centrifugal horizontal or vertical split case pump designed for water-works service.

If the pump station and intake structure are to be located within a surface or underground reservoir, vertical turbine pumps with the column extending down into the reservoir or its suction well will be a logical choice. If the pump station is located at an above ground storage facility, split case centrifugal pumps will be the preferred selection.

For standard waterworks design for wastewater systems, pump casing will be cast iron and impellers will be bronze with special coating to protect to corrosion and erosion. Base for pump and driver will be cast iron or fabricated coated steel. Pump impeller and casing may have wearing rings depending upon manufacturers' recommendations and consideration of the cost of replacing the rings. Pumps will have mechanical seals or packing seals, ball or roller bearings, and lubrication system.

Pumps which may operate under extreme conditions such as at the ends of pump curves or under frequent on-off operation will have packing seals in lieu of mechanical seals. Mechanical seals will be considered for pumps likely to stand idle for long periods of time. Where scale or abrasive water conditions exist, pump linings and other material options for impeller, shaft, wear rings, and seals are available.

6.2.5.5 Pumping Units

The design and selection of variable speed pumps obtain the desired operations at the lowest possible cost. The cost used to determine the pumping station design should be based on an annualized cost which should consider: the lowest cost is obtained with a minimum number of pumps. However, a minimum of two pumps is recommended. Base flow for combined-flow stations should have sufficient capacity for peak irrigation flows.

The greater the number of pumps, the smaller the reduction of the total station capacity if one pump malfunctions. This increased protection, however, results in higher equipment, facility, and more than 30 percent of the total required capacity of the pumping station. The number and capacity of the pumps shall be such that a 100 percent standby pumping capacity is available with failure of any installed pump.

For such installations, the maximum increment in pumping rates may be made equal to the smallest unit, making it possible to pump at a rate approaching that of the inflow. Experience has indicated that variable speed motors Pump Station Requirements

6.2.5.6 Pump Station Requirements

The decision as to the type of control to specify for a pumping station should be based on providing maximum reliability consistent with economic design. In the pump station, piping system will include: gate valves, globe and angle valves, butterfly valves, ball valves, check valves, and relief valves. Globe, ball, and butterfly valves will be best suited as control valves for modulating the flow to provide desired pressure or valves used rate.

In discharge piping valves a check valve and a gate or butterfly valve will be installed with the check valve between the pump and the gate valve. The check valve will protect the pump from excessive back pressure the gate valve will be used to isolate the pump and check valve for maintenance purposes.

Pressure relief valves, commonly diaphragm activated globe or angle type, will be installed in discharge piping system for flow control and/or pressure regulation, and to protect pump equipment and piping system from excessive surge pressures which could exceed the ratings of system components.

Air release and vacuum relief air release and vacuum relief valves will be used on discharge piping for vertical turbine pumps.

6.2.5.7 Head Capacity Curves

The pump station including suction and discharge piping systems will be designed. To make an accurate determination of the head requirements, a system head curve must be derived depicting calculated losses through the system for various pumping rates. From schematic showing configuration and size of all piping including valves and fittings, information on system head loss calculations can be found in calculation analyzing. Pumps at the pump stations will be sized to handle individually and in combination the maximum projected daily consumption, the peak hourly rate, and the estimated minimum hourly rate.

6.2.5.8 Operating Limits - NPSH Restrictions

Net positive suction head available (NPSHA) is the head available above vapor pressure head to move a liquid into the impeller unit of the pump. It is necessary to ensure that the NPSHA exceeds the net positive suction head required NPSHR to prevent cavitations.

6.2.5.9 Pumps Efficiency

Pump performance can be shown either as a single line curve for one impeller diameter or as multiple curves for the performance of several impeller diameters in one casing. Within the limit of pump efficiency from 60% and 120% the pump should be selected.

6.2.5.10 Key Design Requirements for Pumps

The following general design factors should be considered for booster pumping stations:

1. Pump efficiency at the operating points [at the intersection of the pump curves with the system head curves];
2. Pump start-up and performance testing requirements;
3. Pressure rating of pump casing and end connections;

4. Pump type: variable speed horizontal split case - centrifugal pumps.
5. Maximum pump speed: 2900 rpm.
6. Horsepower requirements 315 kW at full load for each pump; identify operating efficiency at full load, and specify service factor;
7. Electric motor thermal overload protection;
8. Availability of electric power supply at the voltage, amperage, and in the phase configuration desired;
9. Maximum suction velocity: 1.5 m/s.
10. Maximum discharge velocity: 2.5 m/s.
11. Efficiency minimum: 81.8%.
12. Maximum noise level in pump buildings 85 db measured 1 m from building wall at any point.
13. Pump duty selection:
 - a. Select pumps to operate between 65% and 125% of best efficiency point (BEP) flow under all conditions of operation.
 - b. Select pumps to operate at a constant flow rate under varying head conditions.
14. Provision of back-up power facilities
15. Potential for surge or transients (water hammer)
16. Need for treatment of pump station discharge (chlorination for example);
17. Pipe and equipment support requirements (thrust block or other restraint);
18. Maintenance requirements for access and equipment removal and replacement; and
19. Benefit of installing a piping by-pass around the pumping equipment.

***Note:** Valves outside the pumps room shall be buried and installed in concrete service boxes or in valve chambers. Valve chambers shall be provided for all valves installed below grade in unpaved areas. Valves installed below grade in paved areas may be direct buried with a riser provided to grade to access the valve operator.*

6.2.5.11 Regulation

Regulation facilities shall be used where necessary to control the flow or direction or limit the pressure in a section of the pipeline. Field automatic control valves shall have backup electric power supply or pneumatic control. This may include the followings:

1. Pipe
2. Stop/check valves
3. Flap gates/valves
4. Elbows
5. Manifolds
6. Tees
7. Reducers
8. Expanders

9. Brackets, bolts and other fixtures.

6.2.5.12 Basic Line

The discharge line should be kept as short and simple as possible. The simplest configuration is where each pump has its own discharge line, entirely independent of the other pumps. Each discharge line conveys pumped water from the pump to a channel or conduit outside the pump station. The elbow of the vertical riser from the pump should be set higher than the discharge line, with a slope down to the discharge end to minimize the volume of back flow when the pumps switch off. Where it is practicable, the centerline of each discharge pipe should be placed higher than the design backwater elevation in the receiving channel or conduit. A flap gate is generally preferred and should be placed at the terminus of each discharge line to prevent back flow if the centerline elevation at the end of the discharge pipe is below the design backwater elevation in the receiving structure. Consideration should also be given to the potential for back flow resulting from storms in excess of the design storm. A check valve may be desirable to prevent such back flow.

6.2.5.13 Manifold System

When excessive length and cost makes individual discharge lines impracticable, it is usual to connect the individual pump discharges into a common discharge line large enough to direct the combined discharge at an acceptable velocity. The connection element is called a manifold. Each pump discharge line must include a check valve to prevent recalculation of flow. It is rarely necessary to use a manifold system in highway pump stations.

6.2.5.14 Design Size:

The size of the discharge pipe should be designed to satisfy the following requirements:

1. At least as large as the pump discharge diameter.
2. Maximum discharge velocity of 3 m/s.
3. Be determined using Eq. 6.7.

$$D = 1.128 \sqrt{\frac{Q}{V}} \quad (\text{Eq. 6.7})$$

where:

- D: pipe diameter, m (ft)
 Q: discharge in pipe, m³/s (cfs)
 V: maximum velocity, m/s (fps).

6.2.5.15 Friction Loss Equations

There are differences in the way in which friction losses through pipes are calculated. These are:

1. Darcy-Weisbach.
2. Hazen-Williams.
3. Manning's Equation.

The choice is up to the designer and the available manufacturer's data. Generally, Hazen-Williams equation is used for the losses throughout the pumping station. The Hazen-Williams equation for friction loss (Eq. 6.8), is the most widely used.

$$h_f = \frac{C_u V^{1.85} L}{C^{1.85} D^{1.165}} \quad (\text{Eq. 6.8})$$

where:

- H_f: Friction loss, m (ft)
- L: Length of pipe, m (ft)
- C_u: Unit conversion coefficient = 6.83 SI (3.022 English)
- V: Discharge velocity, m/s (cfs)
- C:= Friction factor
- D:= Pipe diameter, m (ft)

The Hazen-Williams equation should only be used for turbulent flow and is most applicable to water at a temperature of about 23 °C. The friction factor, C, for the Hazen-Williams varies with pipe material and is typically in the range of 60 to 160. A design value of 100 is typical for smooth steel pipe and smooth concrete pipe and 120 for plastic pipe.

6.2.5.16 Appurtenance Energy Losses

The most common approach to computing energy losses through appurtenances such as valves and elbows is by use of a dimensionless loss factor, K, applied to the velocity head as given in Eq. 6.9.

$$h_l = K \frac{V^2}{2g} \quad \text{Eq. 6.9}$$

where:

- h_l: friction loss through appurtenance, m (ft)
- K: loss factor based on standard data or manufacturer's specified data
- V: velocity through appurtenance, m/s (fps)
- g: acceleration due to gravity, m/s² (ft/s²)

Where an appurtenance incurs a velocity change, such as a reducer or expansion, the head loss calculation as given in Eq. 6.9.

$$h_l = K \frac{(V_2^2 - V_1^2)}{2g} = K \frac{\delta(V^2)}{2g} \quad (\text{Eq. 6.9})$$

where:

- h_l: friction loss through appurtenance, m (ft)
- K: loss factor based on standard data or manufacturer's specified data

- V1: entrance velocity to appurtenance, m/s (fps)
 V2: exit velocity from appurtenance, m/s (fps)
 g: acceleration due to gravity, m/s² (ft/s²)

6.2.6 *Water hammer and surge tank design:*

Tripping of pumps can lead to overpressures, which may either require excessive pipe wall thickness or some form of water hammer protection. The most appropriate type of water hammer protection depends on the pipeline profile as well as the flow characteristics of the pipeline. , the most effective way of preventing negative pressures and also for reducing overpressures is the use of surge control valves at the nodes and pressurized surge tanks and even non return valves if negative pressures are tolerable. Water hammer following pump trip is usually most severe in the case of lines of low frictional resistance. Pump trip is practically instantaneous, especially for lines where the pump rotational.

6.2.6.1 *Celerity of the pressure wave*

The wave celerity (another word for velocity) in pipelines will, but it will always be lower than the wave celerity of sound in the fluid (in water the celerity of sound is close to 1450 m/s). The wave celerity (C) of a circular pipe is

$$C = \sqrt{1/p \left((1/K) + (D/(eE)\phi) \right)} \quad (\text{m/s}) \quad (\text{Eq. 6.10})$$

where:

P = water density 1000 kg/m³

K = compression modulus for water 2.19 x 10⁹ Pa

D = average dia of the pipe (m)

e = is wall thickness [m],

E = elasticity modulus for HDPE pipe 0.8 10⁶ MPa

Φ = Poisson's ratio = 1 - μ² , μ = 0.4 for HDPE pipe = 1 - 0.4 = 0.6

From Newton's second law we understand that force (pressure time's area) is the result of a mass being accelerated. In this connection the wave celerity c stands for the mass per unit time which is accelerated. The acceleration is caused by pumps, valves, etc. Therefore it is likely that stiff systems with high wave celerity will give higher force and pressure. The Joukowsky equation expresses the rise in pressure Δp caused by a change in velocity ΔV:

$$\Delta p = \pm c \rho \Delta V \quad [\text{Pa}] \quad (\text{Eq 6.11})$$

If we use hydraulic head h instead of pressure, we obtain:

$$\Delta h = \pm c/g \Delta V \quad [\text{mWc}] \quad (\text{Eq 6.12})$$

where ρ is the density of the fluid (water: 1000 kg/m^3) and g is the gravity constant (9.805 m/s^2).

The sign of Δp or Δh depends on the direction. If we close a valve we get a pressure rise at the upstream side of the valve and a pressure drop on the downstream side.

6.2.6.2 Pressurized surge tank design:

The incompressible flow differential equations of motion were analyzed for a number of cases in order to obtain a generalized air vessel volume as a function of the minimum relative head at the pumping station. Figs. 6.11 and 6.12 show the nomenclature and minimum and maximum head envelope for a generalized pipeline. Using the results of the analyses, summarized in Fig.6.12, the minimum head can be calculated as a function of the initial pumping head.

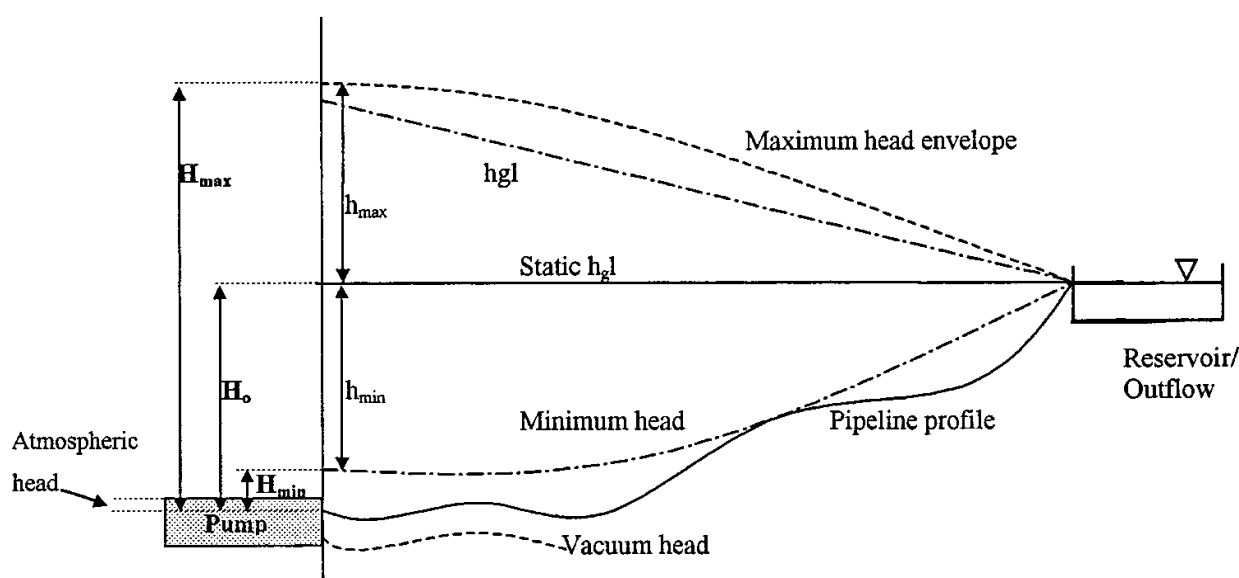


Fig. 6.11 Typical hydraulic heads of the system

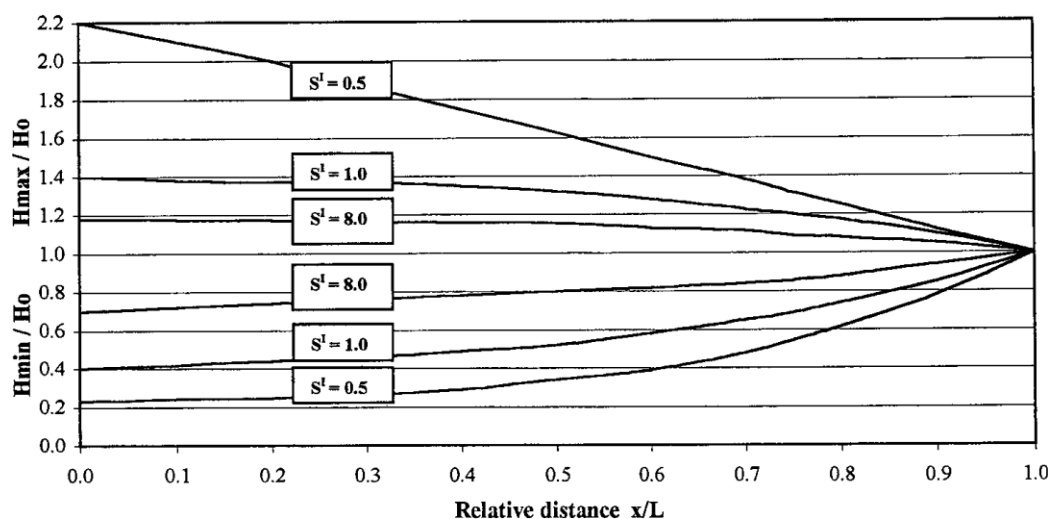


Fig. 6.12: Maximum and minimum head envelopes using incompressible flow theory.

The symbols are as follows:

S' = dimensionless gas volume = $S_0 g H_0 / A L V_0$;

S_0 = gas volume at steady state operating pressure;

Flow theory suggests the following relationship between decelerating head on a water column and the rate of deceleration:

$$h = - L / g \, dV / dt \quad (\text{Eq 6.13})$$

which may be integrated to obtain the maximum cavity volume remaining upstream before the water column reverses, i.e., total vessel volume;

$$S = A L V_0^2 / 2gh \quad (\text{m}^3) \quad (\text{Eq 6.14})$$

where:

S = volume of vessel would force into the pipeline behind the water column,

A = cross-sectional area of the pipe,

L = its length,

V_0 = initial water velocity,

g = gravitational acceleration, and

h = average decelerating head ($h_{\min} / 2$).

From Boyle's law for gas expansion in the air vessel air volume

$$S' = S_0 g H_0 / A L V_0^2 \quad (\text{m}^3) \quad (\text{Eq 6.15})$$

pipe diameter is as follows:

$$D_e / D_p = (2 V_0^2 / (2gh_{\min}))^{0.25} \quad (\text{Eq 6.16})$$

6.3 Bases and Parameters for Electrical Design

6.3.1 Wells

6.3.1.1 Electrical Power Subscriptions

As wells are spread in a relatively wide area, it is not practical to have a centralized Low Voltage (LV) power substation for all wells. Electrical cable diameters, losses, and voltage drops would be unacceptable. On the other hand, having a large number of wells makes allocating individual LV subscription impractical neither. An individual stand-by generator along with its Automatic Transfer Switch (ATS) along with individual electrical metering makes the system more costly and difficult to manage and operate. Therefore, our strategy is to use a combination between the two extremes. In other words, we will split the wells to groups so that the power consumption of each group lies in the mid-range electrical loads used in Gaza which is from 600–1000 A. Meanwhile we will keep the distance between each well in the group and the power station of the

group less than 400 m. This yields in feasible cable diameters for the pumps which are about 75HP.

We will allocate a service building for each pump group which will contain the 22kV/380V transformer along with the power panel. A standby generator will be specified as an alternative power supply in case of main power failure.

6.3.1.2 Transformers

1. Indoor transformers will be utilized to provide a higher degree of protection for the transformers from possible gun shots.
2. The load profile will be reviewed and transformers will be selected to obtain peak loading between 60–80%.
3. A fused-disconnect or circuit breaker is required on the secondary of a transformer when the secondary conductor length is more than 8 m to the panel board.
4. Special consideration shall be given to locate transformers in a location where normal vibration would not be detected by the occupants. Also, avoid locating transformers where the magnetic fields generated could interfere with control equipment.
5. Consideration shall be given to structural issues.
6. Adequate ventilation/cooling shall be provided for transformers enclosed in closets.

6.3.1.3 Panel Boards

1. For all panel boards, all pertinent information including the voltage, amperage, and minimum system (i.e. individual component) short circuit rating shall be specified on a one-line diagram (or in a separate panel schedule).
2. Panel boards shall typically be located in dedicated electrical rooms; rooms shall not be shared with tele/data equipment.
3. Panel boards shall be checked for capacity before adding new equipment. If future circuit breakers will be needed in the panel boards, it shall be noted on the drawings.
4. Feeder routings to panel locations will be determined. Panels should not be located under or over beams.
5. Panel boards shall contain 20% spare circuit breakers and choose the standard size manufactured panel board.
6. Main circuit breakers shall be provided for all panel boards which are not located in the same room as their feeder disconnect.

6.3.1.4 Circuit Breakers and Fuses

1. Interrupting capacity of circuit breakers in switchgear or panel boards shall be suitable for the power system feeding them.
2. When specifying circuit breakers and fuses, consider the existing electrical system as well as all the changes and additions to the system, so that the proper coordination of the over current protection is developed throughout the entire electrical distribution.
3. All electronic trip circuit breakers and circuit breakers with ground fault protection shall be identified on the drawings (in one-line diagram).

4. When electronic trip circuit breakers or molded-case circuit breakers with field adjustable trip settings are installed, the set points are addressed and specified.

6.3.1.5 Emergency Systems/Generator Sets

1. Typically, an engine driven generator, with transfer switching, shall provide backup power for the emergency systems.
2. A 4-pole transfer switch shall be used on systems.
3. A diesel driven generator with independent cooling system shall be used for generator units.
4. An engine-driven generator shall be located in a room designed for the purpose. The generator sets shall be isolated from other areas as required in the code for the isolation of hazards. The generator set should be installed close to the normal electric service. Allow a minimum of 1 m around the generator set for service and to ensure free flow of cooling air.
5. An adequate supply of combustion air and cooling air shall be provided for the emergency generator room. Manufacturer's recommendations for air supply and exhaust shall be determined and facilities designed according to these recommendations. Supply air shall be taken from outdoors or from indoor areas having normal ambient.
6. Exhaust generator into an upright stack well above ground level, not into an area well or underground pit. Location of exhaust outlet shall not be located where it would affect building occupants.

6.3.1.6 Cables

1. Generally, all wire and cable shall be installed in conduit. Low voltage control or signal cables may be installed without conduit above accessible ceilings if the cable meets standard listing requirements for the application. If certain low voltage or signal cabling is to be run in conduit, the appropriate drawings, riser diagrams, and specification sections will indicate this.
2. A power cable and a control cable will be installed underground for each pump. These cables facilitate transition of power and control signals between a substation and its belonging wells.
3. In areas where low voltage or signal cables are to be run without conduit, air return plenum locations shall be identified on the drawings.
4. The use of multi-wire branch circuits with common neutral feeding loads is not permitted.
5. Wiring methods under raised floors shall be specified.

6.3.1.7 Soft starters and recovery process control

1. Limiting the motor inrush current shall be investigated. Generally, 460 volt motors 50 HP and over need reduced voltage starting. Solid-state reduced voltage starters (soft starters) will be specified.

2. Soft starter along with its accessories and control devices will be installed in a small electrical panel nearby the well.
3. Manual override will be provided for each automatic control.
4. Highly sophisticated automation may put proper maintenance beyond the capability of the plant operator, leading to equipment breakdowns or expensive servicing. Therefore, the automation scheme of wells will be kept simple and compatible with similar wells widely present in Gaza.
5. Control signals will be transferred from the substations to a centralized control room via a data network.
6. At the centralized control room a PLC will be adopted to control the recovery process.
7. The PLC is connected to a SCADA system for efficient monitoring and management of the whole recovery and irrigation scheme.

6.3.1.8 Design Calculations

1. Basic electrical system design calculations and information shall be performed prior to the completion of design. Copies of this information shall be submitted as a part of the overall project design documentation to the client for review.
2. The secondary distribution system shall be examined for voltage drop from the service transformer downstream to the branch level panel board, and on to the branch circuits. Calculations shall be sufficient to encompass the application range of the project. Secondary distribution and branch circuit system design shall be based on a maximum of 5% voltage drop from the transformer to the utilization equipment.
3. The designer will analyze the distribution system and perform short circuit calculations to ensure that equipment is adequately protected against the effects of short circuits. System components shall be specified with adequate short circuit ratings and/or protective devices or components shall be specified that will reduce fault current levels or durations. It is preferred that higher rated equipment be specified if data on available fault current is questionable, if utility substation or line capacity is projected to increase, or if calculated fault values fall near a standard equipment rating. Minimum equipment standard interrupting ratings shall be identified on the plans preferably on a one-line diagram, or alternately in schedules.

6.3.2 Tanks

1. High level and low level switches along with a level meter will be installed for each tank to signal the water level information to the control system.
2. The water level in tanks will determine the number of operating wells.
3. The high level switch will be used as a back up device of the level meter (in case of not detecting the upper limit threshold) and signal an overflow alarm.
4. The low level switch will be used as a back up device of the level meter (in case of not detecting the lower limit threshold) and signal a low water level alarm.

6.3.3 Booster Pumping Station

6.3.3.1 Electrical Power Subscriptions

Ten booster pumps with a total of 3150 KW will be installed. For this huge amount of power relative to the power network of Gaza, it is not practical to have all these loads fed from a single transformer or a single stand by generator. Therefore, we will split the load to three groups so that we have three electrical subscriptions fed by three transformers and have three independent stand-by generators.

6.3.3.2 Frequency Converters

Scientist desired to have a smooth control for the water flow of in the irrigation network. In current project frequency converters will be utilized for the booster pumps rather than classic soft starters.

6.3.3.3 Control System

1. The purpose of the control system is to operate the pumping station and transmit the information about the operational status of the pump station utilities to the Control Center. The control system will be operated manually and automatically by using PLC system.
2. PLC will be programmed to manage the operation of the pumping station and it could be reprogrammed via connection with computer software. For regular calibration, the hand held programmer could be used.
3. PLC will monitor any fault caused by the internal or the external protection. PLC also shows the location of all levels of float switches and the high- or low-pressure on the main header. It also identifies the generator condition, the no flow caused by any reason throughout the check valves micro switches, and will check the fuel tank levels.
4. The number of operating boosters along with their speed will be determined according to the irrigation schedule.
5. The low water level in tanks and the high pressure at the pump outlets will interlock the booster pumps.

6.3.4 SCADA system

1. The SCADA system will be designed such that the integrity and function of each process (the recovery process and the distribution process) is maintained irrespective of the state of any other system. Nevertheless, on operator work place level, the two processes have to appear as one integrated system.
2. There will be one SCADA server and another redundant server.
3. The operator interface has to include two operator workplaces (one for each process) and one printer. The installed system will be set to allow at least five workplaces placed remotely at the group substations and two workplaces placed locally to be added.
4. The data network should be designed with a high degree of reliability.
5. Computer based control technologies such as SCADA must be secured from unauthorized physical access and potential cyber-attacks. Wireless and network based communications should be encrypted as deterrence to hijacking by unauthorized personnel. Vigorous computer access and virus protection protocols should be built into computer control

systems. Effective data recovery hardware and operating protocols should be employed and exercised on a regular basis. All automated control systems shall be equipped with manual overrides to provide the option to operate manually. The procedures for manual operation including a regular schedule for exercising and insuring operator's competence with the manual override systems shall be included in facility operation plans.

6.4 Bases and Parameters for Structural Design

6.4.1 Structures in this Project

This project includes the design of a variety of structures including *two 4000 m³ water tanks, booster pump station* and *associated facilities* such as *well control rooms* and administration building.

6.4.2 Basis for Structural Design

The structural design basis, criteria and parameters for the various buildings and structures in this project are given in this section.

6.4.2.1 Units

In general International Standard (SI) units are used.

6.4.2.2 Building Codes

Currently, Palestinian building codes for design and construction of structures do not exist. Therefore, the structural design in this project is carried out using the following relevant building codes.

1. ACI Standard, "Building Code Requirements for Structural Concrete (ACI 318M-08) and commentary, American Concrete Institute, Detroit, 479 pp.
2. Specification for the Design Fabrication and Erection of Structural Steel for Buildings, American Institute of Steel Construction (AISC).
3. The 1997 Uniform Building Code (UBC, 1997) is used for calculating the seismic loads at ultimate load level.
4. Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-05) have been used for calculating other loads including dead, live, and wind loads.
5. Specifications for materials shall follow the soil, rock, and other standard specifications of the American Society for Testing and Materials (ASTM).

The design of the reinforced concrete structures is based on limit states methods. The Ultimate Limit state is used to ensure adequate strength, rigidity, and stability and the serviceability limit state is used to control cracking, deflection, and vibration.

6.4.2.3 Software

A number of software specialized in structural analysis and design are used in this project including **SAFE8, STAAD PRO, SAP2000, PROKON, Etabs, etc.** The varieties of the software are used to suit the type of the structural element and for comparison purposes. Excel spread sheets are also used for calculating the loads on some structural elements such as columns

and walls. These sheets mainly facilitate faster calculations of analysis and design of structural elements. It should be mentioned that regardless of the software and Excel spread sheets used, hand calculations are carried out to verify the computer outputs. Also, regardless of the computer program used, all members are designed based on one single code, i.e. ACI 318M-(08).

6.4.2.4 References

Recognized relevant references have been used to assist consultant competent structural engineers in the analysis and design of the structures in this project. These include the followings:

1. Reinforced Concrete: Mechanics and design, by James G. MacGregor, 4th Edition, Published by PRENTICE-HALL International, Inc. 2004, 950 pp.
2. Design of Reinforced Concrete, by Jack C. McCormac, James K. Nelson, Published by Published by John Wiley & Sons, Inc. 2005.
3. Design of Concrete Structures, by A. H. Nilson and C. W. Dolan, 13th ed. Published by McGraw-Hill, Inc. 2004.
4. Reinforced Concrete: A fundamental Approach, by Edward G. Nawy, Published by PRENTICE-HALL International, Inc. 2002.
5. Reinforced Concrete Design, by Chu-Kia Wang and Charles G. Salmon, Published by HARPER & ROW.
6. Steel Structures Design and Behavior, by Charles Salmon and John Johnson, Harper & Row, Publishers.
7. Manual of Steel Construction, American Institute of Steel Construction (AISC).
8. Design of reinforced Concrete Water Tanks, by Khalil Waked, Scientific Book House for Publishing and Distribution, Cairo, Egypt. 2003.
9. Theory and Design of Reinforced Concrete Tanks, by M. HILAL, Faculty of Engineering University, Egypt.
10. Reinforced Concrete Design Handbook, by Shaker El-Behairy, Ain Shams University, Egypt.
11. Foundation Analysis and Design, by Joseph E. Bowles, Published by McGraw-Hill Book Company.
12. Wastewater Engineering Treatment Disposal Reuse, by Metcalf & Eddy, Inc. published by McGraw-Hill Book Company.
13. Other available references, design manuals and aids, papers, reports, studies, case studies, etc.

6.4.3 Design Criteria and Parameters

6.4.3.1 Building Shape

The water tanks in this project have been selected to be ground circular tanks. A circular tank is structurally more efficient than rectangular one both in terms of wall area per unit volume and economy of materials. Circular shaped tank is geometrically the most economic shape giving the least amount of walling for a given volume and depth. Circular tanks are also efficient from a

structural point of view. This is because the straining action along the circumference is mainly hoop tension. Theoretically, bending moment does not develop along this direction; although it develops along the vertical direction. Circular tanks have the attraction of constructing a thin dome shaped roof, free of supporting columns. From technical and economical point of view, neither elevated tanks nor underground tanks are suitable in this project, especially when considering the large volumes of the tanks and complexity of construction.

The booster pump station in this project is of normal shape which is influenced by the general layouts of the site which will accommodate also the water tanks, administration buildings, and other facilities. The size of the pump station is however large considering the number and size of booster pumps.

Other buildings in this project such as administration buildings and well control rooms have normal shape that is also influenced by design requirements and general site layout. Considerations are, however, given to allow for installation and maintenance of pumps, fittings and other electro-mechanical parts.

6.4.3.2 Concrete

1. Ordinary Portland cement concrete of different strengths and characteristics is used to satisfy the requirements of various structural elements in this project as follows:
 - i. **B400** ($f_{cu} = 40 \text{ MPa}$) is used in the circular ground tanks. These elements are subjected to direct contact with the water. The concrete used in such elements must be impermeable, dense of low water to cement (w/c) ratio and workable to allow good compaction during casting. The required minimum slump is 100 mm and the minimum cement content is 350 kg/m^3 .
 - ii. **B300** (concrete cube strength $f_{cu} = 30 \text{ MPa}$) is used in all structural elements of the booster pumping station, service building, the foundations of the generator house and fuel tank and in the structural elements of secondary importance. The required slump is 100 mm and the minimum cement content is 300 kg/m^3 .
2. The use of super-plasticizers by the concrete batching plant is permissible to provide the required slump and to allow the use of low w/c ratio in all concrete types. The Consultant recommends that concrete for tanks should have a water cement ratio (w/c) no higher than 0.53 for thin sections and 0.44 for thick sections.
3. Continuous wet curing will minimize shrinkage during the time of strength built-up and should be considered preferable to the use of curing compounds during this initial period. For walls, forms may be loosened and left in place so that a continuous flow of water may pass over the fresh concrete surface. For horizontal surfaces, water- retaining coverings may be used continuously moistened. After the initial 7-day curing period, the use of membrane curing may be used for the subsequent curing.
4. For design, the required average concrete strength is assumed to be equal to the specified concrete strength plus 50 MPa to account for variation in concrete results.
5. The structural design based on the ACI code considers the concrete compression strength of the standard cylinder (f'_c). For the design purpose the value f'_c will be taken equal to $0.8 f_{cu}$.

6.4.3.3 Reinforcement Steel Bars

1. Reinforcement steel bars of diameters of not less than 10 mm are used in the various structural concrete elements including the stirrups of columns and beams. Bars of diameter less than 10 mm may be used as secondary reinforcement only, e.g., shrinkage reinforcements that are placed in the solid part of the ribbed slabs. The following two steel grades are used in accordance with the standard specification for deformed and plan billet-steel bars for concrete reinforcement ASTM A615:
 - i. **Grade 60** deformed reinforcing steel bars of yield strength $f_y = 400 \text{ MPa}$ (specified 420 MPa) are used in the reinforcement of all structural elements.
 - ii. **Grade 40** smooth reinforcing steel bars of yield strength $f_y = 276 \text{ MPa}$ may be used as secondary reinforcement.
2. In concrete wall sections of thickness 225 mm or greater, two layers of reinforcing bars shall be placed, i.e. one at each face of the section.
3. Minimum reinforcement ratios will be maintained in all sections as specified in the relevant ACI section.

6.4.3.4 Structural Steel

Structural steel ASTM - A36 is used in the steel structures and members such as the crane girder in the booster pump station.

6.4.3.5 Concrete Cover

1. For faces away from the liquid and for parts of the structure neither in contact with the liquid on any face, nor enclosing the space above the liquid, the cover should be the same as provided in Chapter 7 in the ACI code.
2. For liquid faces of parts of members either in contact with the liquid or enclosing the space above the liquid (such as inner faces of water tank roofs), the absolute minimum cover to all reinforcement shall be 25 mm or the diameter of the main bar whichever is greater. In the presence of soils, and water of corrosive character the cover should be increased by 12 mm, but the additional cover shall not be taken into account for design calculations. In this project minimum covers for such surfaces will be kept at 40 mm to ensure against corrosion of reinforcement.

6.4.3.6 Loads

Dead Load (D) is calculated based on the volume and unit weight (γ) of the materials and soil, e.g.:

$\gamma = 25 \text{ kN/m}^3$ for reinforced concrete;

$\gamma = 18 \text{ kN/m}^3$ for soil;

$\gamma = 20 \text{ kN/m}^3$ for tiles, plaster and mortar;

$\gamma = 12.5 \text{ kN/m}^3$ for typical hollow blocks of $400\text{mm} \times 200\text{mm} \times 200\text{mm}$.

Live Load (L) is assumed equal to 2.5 kN/m^2 .

Moving Loads such as lifting cranes are calculated according to actual weight of these parts. In addition, impact, shaking, and braking (or accelerating/decelerating) loads are

calculated as percentages of the vertical load equal to 25%, and 10% respectively. Traffic loads are assumed equal to 5 kN/m² for private cars and 20 kN/m² for trucks.

Earth Pressure (H) is calculated in accordance with Rankin theory for lateral earth pressure. The earth pressure is considered active (K_a) for cantilever retaining walls and at rest (K_o) for retaining walls and other members that are laterally restrained.

Fluid Pressure is calculated equal to fluid own weight and acts perpendicular on the surfaces, i.e. vertical and horizontal fluid pressures that act on horizontal and vertical surfaces at the same point are equals.

Wind Load (W) is expected not to control the design since the majority of the structures are either low rise, e.g. water tanks, booster pump stations and administration buildings. Thus, wind load is not considered in the structural design. In general, lateral stability of buildings against lateral loads is provided by the external and partitioning walls and confinement provided by surrounding earth when exists. In dome design, however, the wind load is accounted for in determining maximum straining actions.

Seismic Loads (E) are expected not to produce a critical loading case since Gaza is not subjected to severe earthquakes and thus its classification based on the UBC-97 can be reasonably assumed to lie between Zone 1 and 2. In addition, the structures are low rise and thus an earthquake will not subjected them to severe seismic forces. It should be noticed that soil liquefaction cannot occur considering the soil type and location of ground water table in the project area. Therefore, the loading combinations in the ACI code that include the influence of seismic loads will not be considered critical in the design. However, detailing of reinforcement will be made as required for low to moderate earthquake areas in accordance with Chapter 21 in the ACI code.

6.4.3.7 General Design Requirements

The basic requirements that have been considered for the design of water structures are:

- i. Adequate strength, i.e. the structure should have enough resistance to applied straining actions including bending moments, shear and axial forces, and torsion.
- ii. Free from excessive cracking, i.e. crack control to minimize both size and number of cracks.
- iii. Limited deflections.

6.4.3.8 Required Strength (Factored Loads)

The load factors which are given in the ACI Code Section 9.2 are used for the structural design to calculate the required strength (U) corresponding to the following load cases:

$$U = 1.4(D + F) \quad (\text{ACI 9-1})$$

$$U = 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R) \quad (\text{ACI 9-2})$$

$$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (1.0L \text{ or } 0.8W) \quad (\text{ACI 9-3})$$

$$U = 1.2D + 1.6W + 1.0L + 0.5(L_r \text{ or } S \text{ or } R) \quad (\text{ACI 9-4})$$

$$U = 1.2D + 1.0E + 1.0L + 0.2S \quad (\text{ACI 9-5})$$

$$U = 0.9D + 1.6W + 1.6H \quad (\text{ACI 9-6})$$

$$U = 0.9D + 1.0E + 1.6H \quad (\text{ACI 9-7})$$

where:

U:	Required strength
D:	Dead load
L:	Live load
W:	Wind load
E:	Seismic Load
H:	Earth pressure load
F:	Fluid pressure load.

6.4.3.9 Design Strength (Reduced Strength)

The design strength is calculated by multiplying the nominal strength with strength reduction factors (ϕ) corresponding to the type of the straining actions as follows:

Design Strength = ϕ Nominal Strength

Strength reduction factors “ ϕ ” (ACI 9.3):

$\phi = 0.9$	for tension-controlled sections.
$\phi = 0.75$	for compression-controlled sections with spiral reinforcement
$\phi = 0.65$	for other compression-controlled sections
$\phi = 0.75$	for shear and torsion
$\phi = 0.65$	for bearing on concrete
$\phi = 0.75$	for strut-and-tie models

6.4.4 Design Considerations for Special Structures in this Project

The project includes normal structures such as administration buildings and structures of special use such as water tanks. The design of special structures required in this project special considerations and measures including the followings:

1. Water tanks are designed to provide stability and durability in addition to maintaining the quality of stored water in accordance with acceptable engineering standards. The most important requirement in the construction of water tanks has been water tightness, i.e. no water should be allowed to leak. Corrosion of reinforcing steel bars is to be prevented.
2. Water tanks are in direct contact with water and/or earth, which will expose these structures to severe environmental conditions. Concrete strength and cement type, structural system, applied concrete technology, and construction techniques are selected to suit the existing conditions. The considerations that are taken in their design include the followings:
 - i. All materials including additives, coatings compounds, etc. used in contact with water must have certification that it is safe for use in contact with water. These materials should be carefully used and according to their manufacturer’s recommendations. To avoid unnecessary public health concerns and consumer complaints, the following should be addressed:

- a. For water concrete tanks, use appropriate form oils, concrete surface sealants, and curing compounds and plasticizers.
- b. Temperature, time and ventilation conditions as well as thickness of the applied layers specified for proper curing of coatings are critical elements to assure protection against the leaching of undesirable level of substances into the water. In any case, water quality should be monitored and tests should be done before and during service.
- ii. Crack width is controlled in order to protect the reinforcing steel bars from corrosion. Crack is controlled by following ACI code requirements regarding concrete cover, spaces between reinforcements, reinforcement size and stress. The limit on crack width will be assumed to range from 0.3 mm to 0.1 mm for structures that are subjected to normal (e.g. service building) to severe (e.g. circular tank) environmental conditions, respectively. Based on the ACI code crack control is handled indirectly by defining specific roles of the distribution of reinforcement. The cracks in this project are also controlled by keeping the reinforcement stresses low, i.e. the steel and concrete strains will be low which minimize cracking.
- iii. Use of adequate concrete cover for each member type and environmental conditions as detailed in Chapter 7 in the ACI code. For example concrete cover for concrete members cast and permanently exposed to earth will be 75 mm. If concrete is protected against soil and environmental factors the concrete cover will be 50 mm.
- iv. Construction joints other than those which are specified in the design drawing will not be allowed. Planned construction joints will include water stopper if necessary and will be selected in suitable locations that are both convenient for construction and structural soundness.
- v. All concrete surfaces will be protected against harmful environmental factors. For example all concrete surfaces subjected to earth will be protected by bitumen layers. Concrete internal surfaces that are subject to water will be protected against environmental factors and water leakage using appropriate sealant agent.
- vi. In order to obtain dense concrete of low permeability, concrete will be mixed, transported, cast, compacted and cured in accordance with high standards as will be detailed in the specifications.

6.4.5 Joint Details and Placement

Given good quality concrete with a minimum of drying shrinkage, some shrinkage stresses will still exist. To control cracking, reinforcement and joints are used. These two must be used together; an increase in spacing between control joints will require an increase in steel reinforcement percentage. Four types of joints may be used as specified on drawings.

1- Construction Joint: This joint defines the end of concrete placement. It is a rigid joint where reinforcement is continuous and either a water stop is used, or the new concrete is bonded to the old. Properly bonded horizontal construction joints can be made watertight. Bonding of vertical construction joints is difficult, and water stops are used instead. The possibility of forming cold joints during construction in this project must be minimized. Proper treatment, e.g. cleaning of old services and using bonding agent must be applied in cases where concrete casting had to stop. The details of construction joints in this project will be clearly shown on the relevant final structural drawings.

2- Contraction Joint: This is a typical movement joint which accommodates the contraction of concrete. It may be either a complete contraction joint in which there is discontinuity of both concrete and reinforcement, or it may be partial contraction joint in which there is discontinuity of concrete but reinforcement runs through the joint. Water stops are used across the joint. Properly spaced contraction joints will presumably interrupt restraining forces such that other random cracking is eliminated. Unbonded dowels or keys are used to transfer shear forces. The details of contraction joints, if used in this project, will be clearly shown on relevant structural drawings.

3- Expansion Joint: This is a movement joint with complete discontinuity in both reinforcement and concrete. Its purpose is to accommodate either expansion or contraction of the structure, and it eliminates both tension and compression forces. This joint should be used to separate structures or portions of structures of different masses. This joint may be used in the base of the water tank. The details of expansion joints will be clearly shown on the relevant structural drawings.

4- Sliding Joints: This is also a movement joint with complete discontinuity in both reinforcement and concrete at which special provision is made to facilitate relative movement in place of the joint. A typical application of such a joint is between the wall and floor in some cylindrical tank designs. In this project rigid joint was assumed between the wall and floor. However, the joint between the dome roof and tank walls is selected to be simple where friction is to provide lateral constraint.

7 RECOVERY AND REUSE SYSTEM DESIGN

This section includes the design input and results for the recovery (recovery wells, collection pipes, observation wells, and associated facilities), and the reuse schemes (water tanks, booster pumping station, irrigation water network, and associated facilities). The design covers all concerned fields, i.e. hydraulic, structural, mechanical, electrical, etc.

7.1 Hydraulic and Mechanical Design

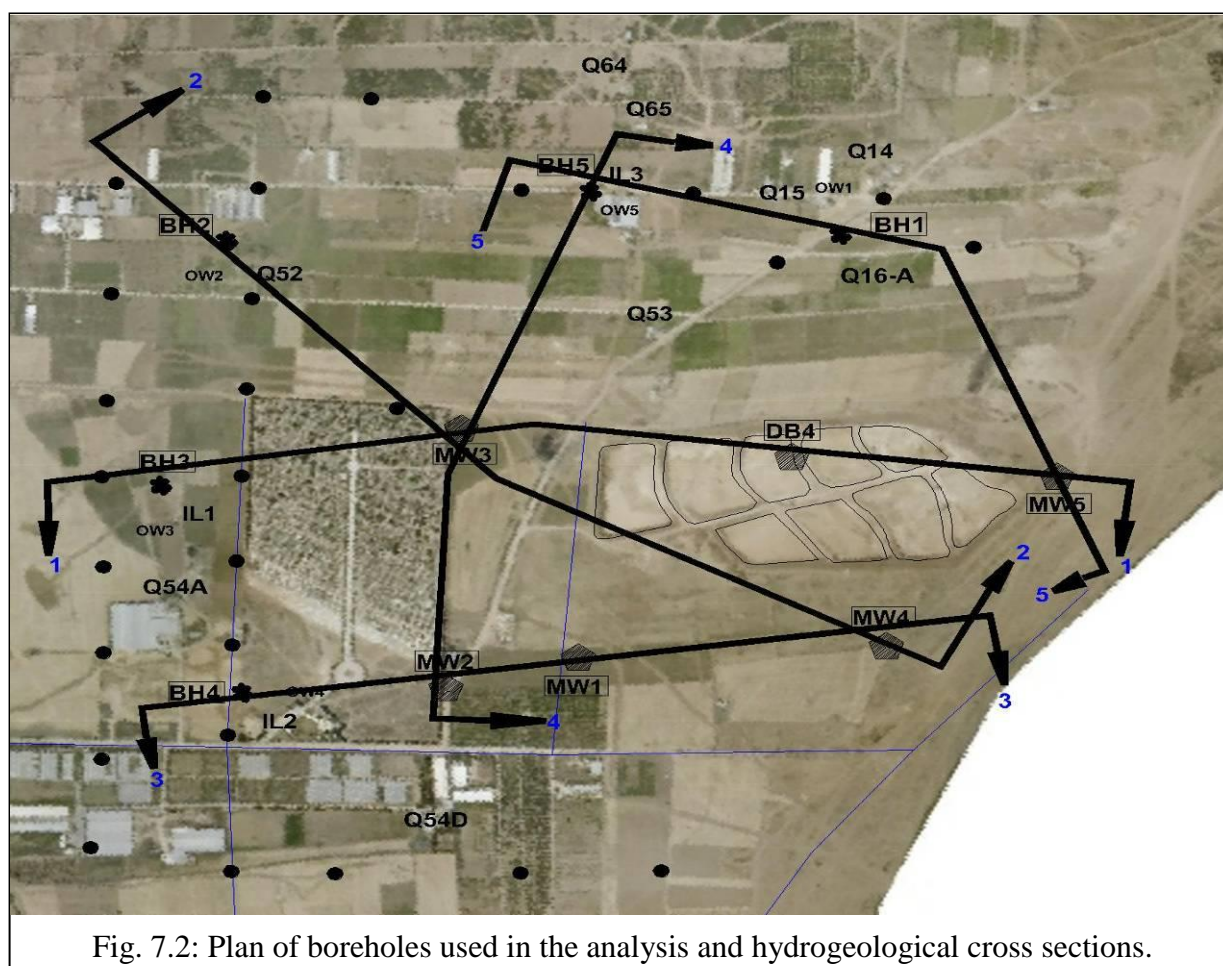
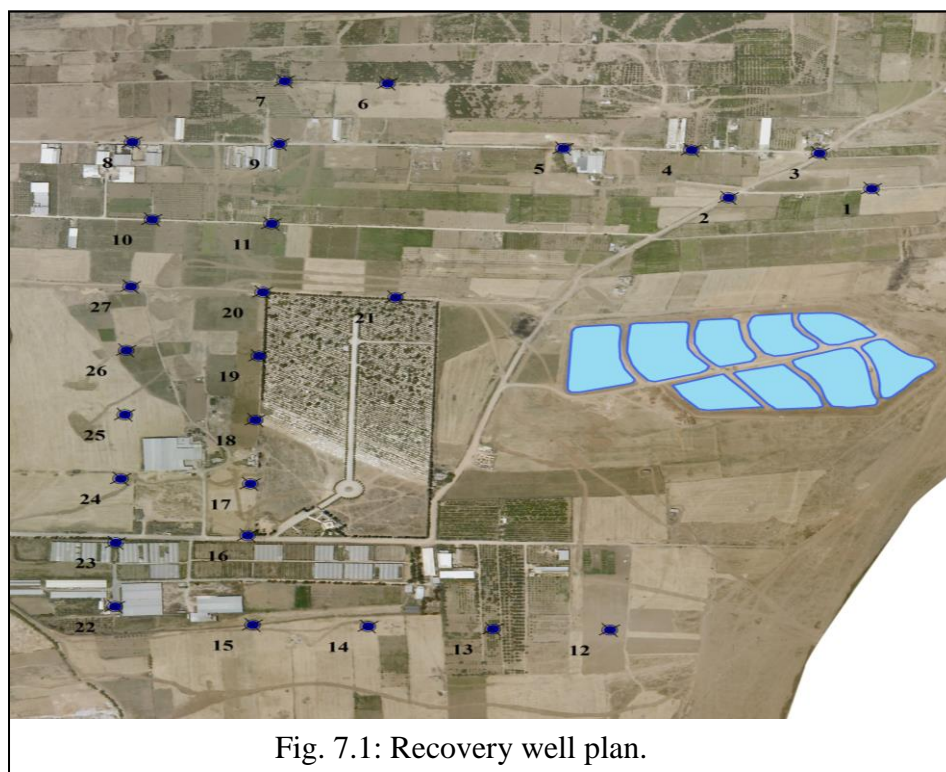
7.1.1 Wells

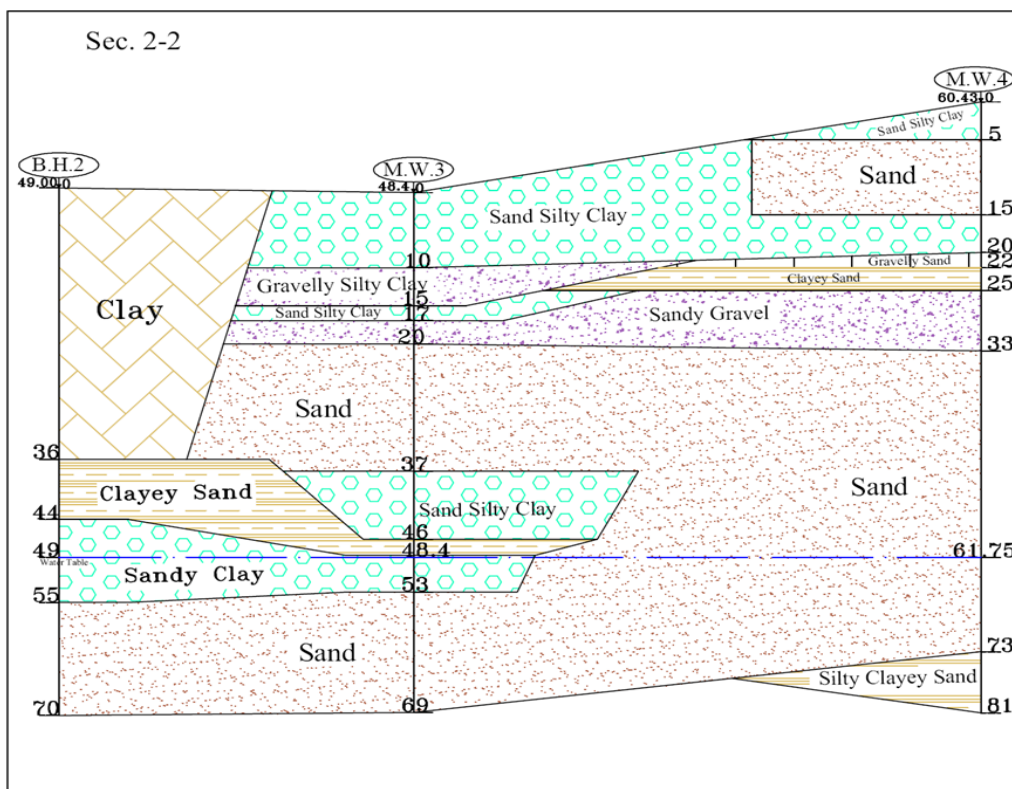
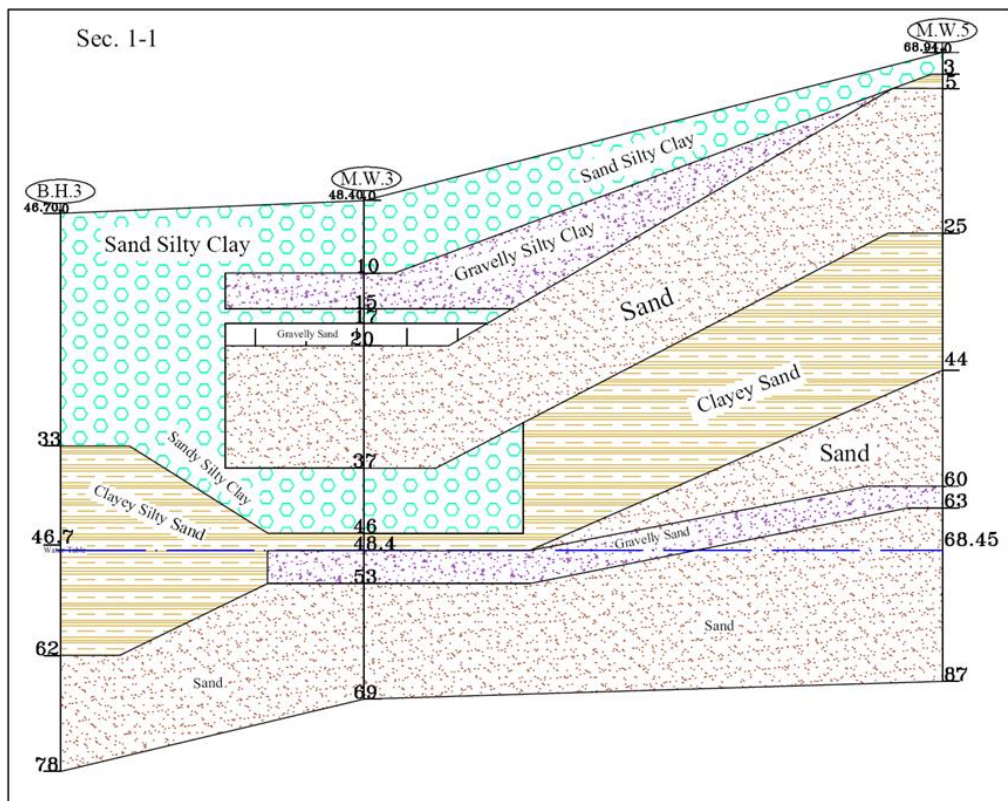
7.1.1.1 Location

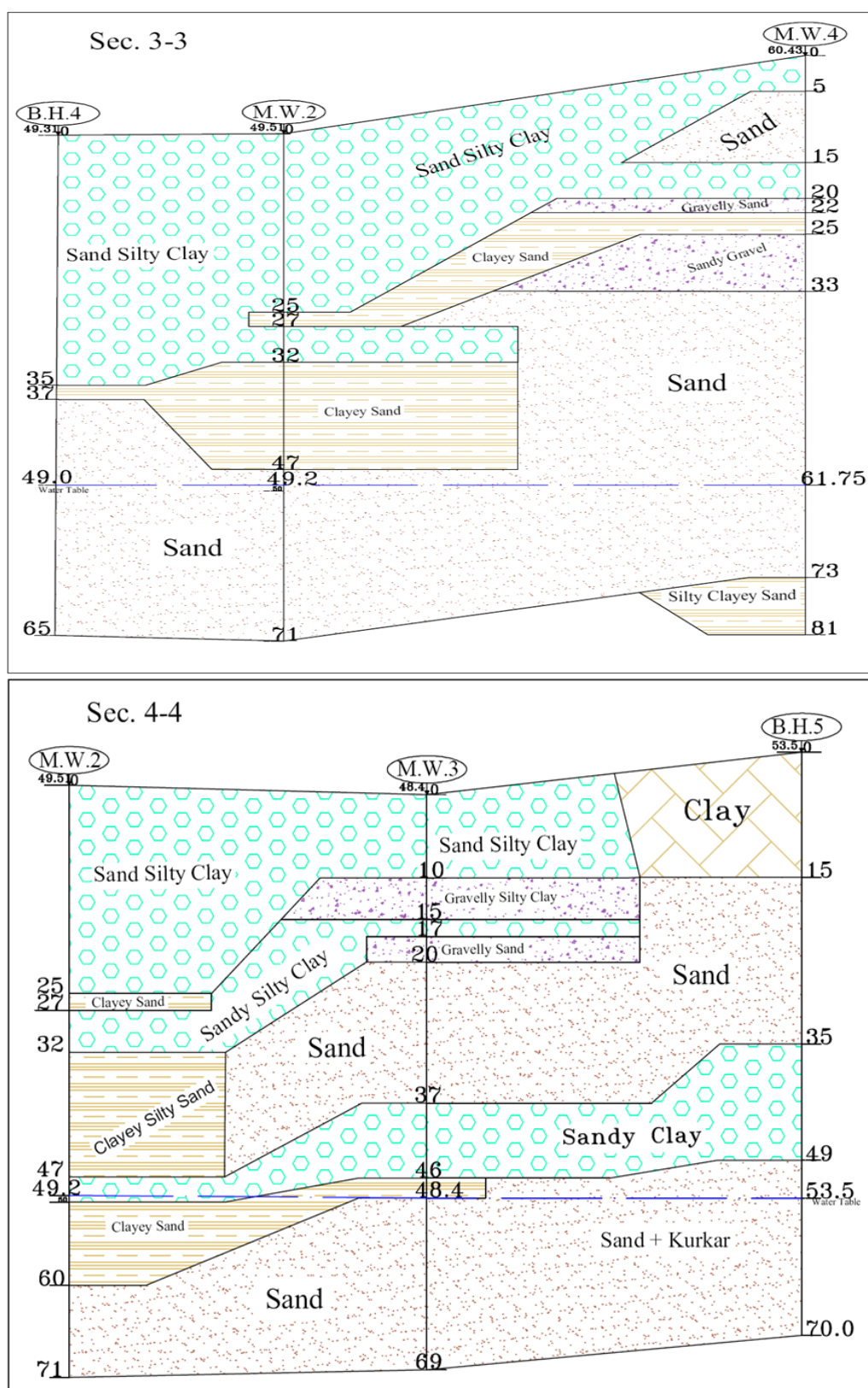
The number of recovery wells was calculated based on the maximum quantity of water that should be recovered during the peak month of October which is equal to 50,885 m³/day. The total number of wells is 27 where each should have a capacity of pumping between 150 m³/hr to 200 m³/hr. The number of operation wells is 25 wells with a capacity of 170 m³/hr. Two wells are allocated to provide flexibility in operation and to compensate any shortage in water supply in case of emergency if for example some wells are failed.

Based on the hydrogeological approach and groundwater model in Section 5, the wells were carefully allocated around the infiltration basin with a distance of 550 m to 750 m from the basin. The minimum distance allows of a retention time equal to 1000 days which ensures the operation of the sand aquifer treatment process. The wells are concentrated in the water flow direction which allows to capture the plume and prevent exceeding the 750 m distance from the basin (modeling approach). Fig. 7.1 shows the planned locations of recovery wells.

In addition to groundwater modeling, hydrogeological approach was used to determine exactly the location of the recovery wells. Based on the hydrogeological investigation carried out under the current project and previous hydrogeological data, several cross sections were drawn to determine the exact location of the wells and the depth of the well screen. Fig. 7.2 shows the plan of boreholes that were used to draw the hydrological cross sections in different directions. Fig. 7.3 shows the various cross sections that pass through the recovery wells areas.







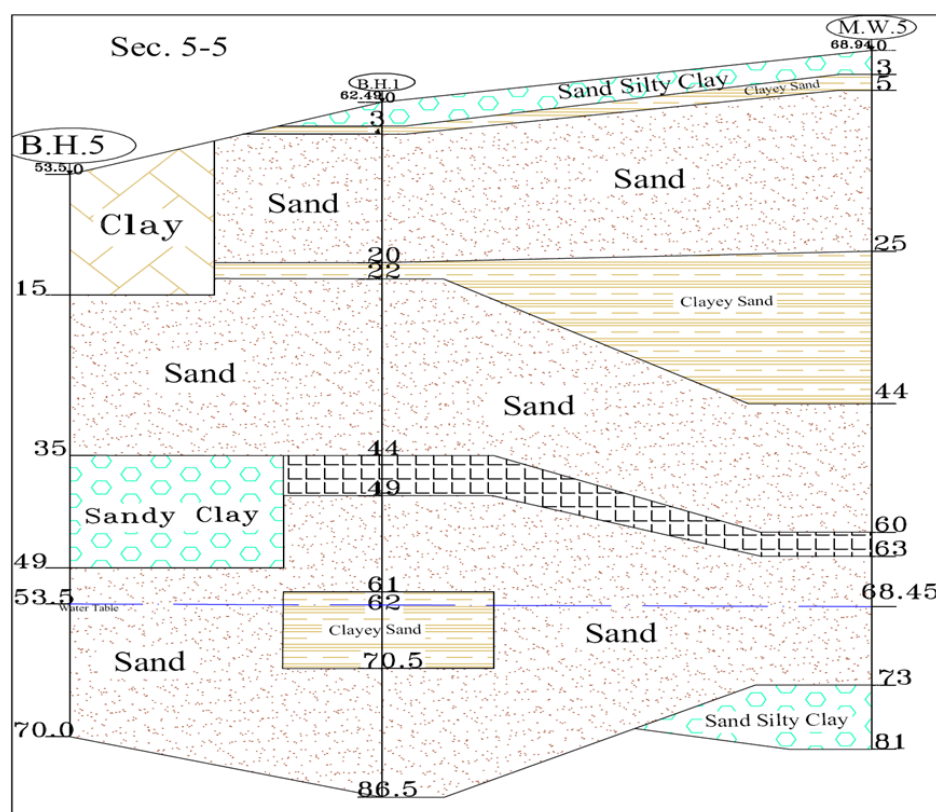


Fig. 7.3: Hydrological cross sections.

7.1.1.2 Well Components

Based on the design criteria mentioned in Section 6, well components such as the screen diameter and length, gravel pack, the location of screen, the location of pump, etc. were determined. Fig. 7.4 shows an example of the design results of a recovery well (well No RW1 in Fig. 7.1) that has the following characteristics:

1. The external diameter of borehole is 20 inch.
2. The diameter of screen is 12 inch, opening size is computed to retain 90% of gravel pack, therefore, the opening size will range between 0.6 mm to 0.8 mm and the opening slot percentage is 30%.
3. The length of screen is 13 m located in sand or coarse sand layer below the water table. Stainless steel screens are used. The screen is located below the water table with a distance equal to double the expected drawdown of the water table after pumping of 200 m³/hr. Based on the pumping test report, the drawdown of the water table will be about 6 m, therefore, the shaft of the pump should be about between 10 m to 12 m below the groundwater table. The design calculations for this well are included in *Appendix 2*.
4. The total length of pump housing depends on the depth of the water table, the depth of permeable layers (sand aquifer) and the drawdown of the water table. Table 7.1 shows the total length of pump housing for each well. The gravel pack was designed based on the safe

analysis of the permeable layer. Based on the design procedures presented in section 6.1, the recommended range of D_{50} of gravel pack size is ranging from 2 mm to 4 mm. In addition, a sieve analysis curve is made for each gravel pack of each well as presented in *Appendix 2*.

5. The distance between the recovery wells is estimated based on the water table drawdown records from observation wells during the pumping tests. It was found that in the case of pumping $200 \text{ m}^3/\text{hr}$, the drawdown in the well is about 6 m and at 50 m the drawdown is 34 cm. With extrapolation of the drawdown curve, the zero drawdown is expected to be at 70 m from the well. Therefore, the distance between the wells should not be less than 140 m. Fig. 7.5 shows the drawdown and the recommended radius of influence between the recovery wells.

Table 7.1: Total length of pump housing for each well

Well No	Length of Pump Housing (m)
1	81.64
2	81
3	81.17
4	75.5
5	67.75
6	63
7	62.5
8	61
9	63
10	63
11	65.98
12	68
13	68
14	68.5
15	68.5
16	67
17	66
18	72.77
19	71.91
20	71.55
21	72.36
22	67
23	65.5
24	72.08
25	71.71
26	71.48
27	71.38

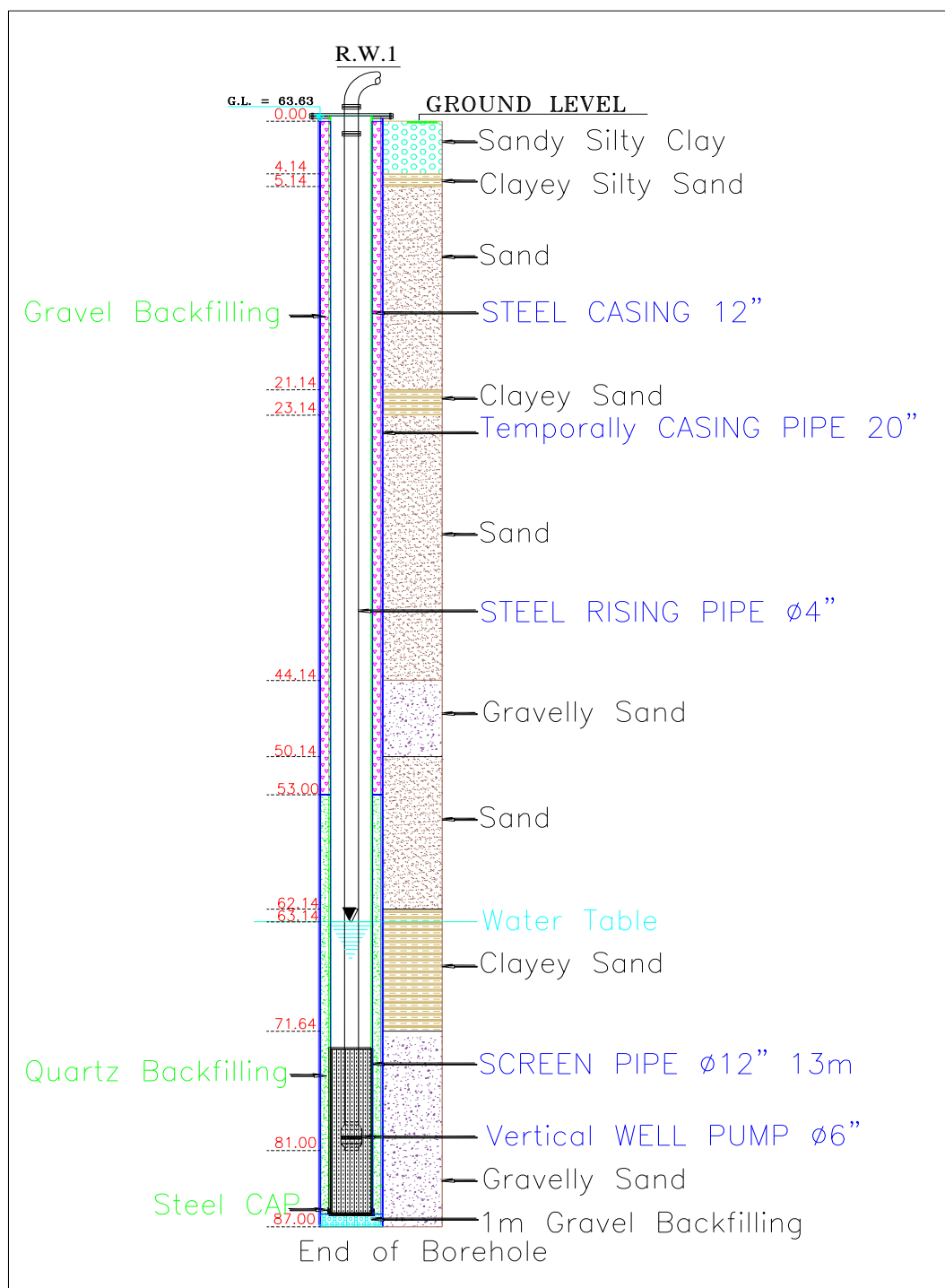


Fig. 7.4: Prototype design example of a recovery well (well No. 1)

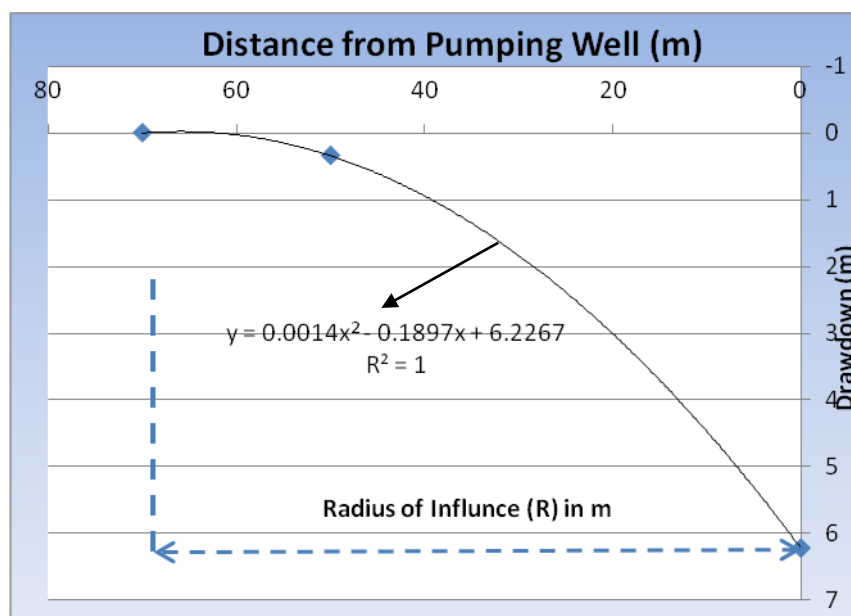


Fig. 7.5: Distance-drawdown curve of 200 m³/hr pumping rate.

7.1.1.3 Well Pump

Recovery well pump delivers the water to the intermediate tank. There are five individual pumping groups (G1....G5), containing five wells with a single pump in each well. The pressure pipeline of each pump in the group is connected to a common pipeline, which takes the total water flow of a group to the reservoir tank, where the water is discharged to the free water level of the tank. The principle of the pumping arrangement is shown in the Process flow diagram.

The pumps in a group form thus a hydraulic system where several pumps are pumping parallel in a common discharge pipeline. The pressure loss for each individual pump varies somewhat, due to the different length of the pump's pressure line prior to the common line. Two of the groups have also one additional well with the pump as a stand-by well unit.

The pump size is selected based on the maximum flow rate of a pumping group in m³ per hour and the total dynamic head (TDH) in this hydraulic situation. The total dynamic head is determined by adding together:

1. Distance between GW level and pump discharge pipe outlet level in the tank (geodetic head)
2. Sum of the head losses in the pressure pipeline of the individual pump and in the common pipeline, containing all fittings.

It's assumed that the geodetic head for each pump is approximately the same.

The pump type shall be a vertical turbine pump, installed in the bottom of the recovery well, at the level of the screen pipe of the well. The pump unit shall be supported at the rising main by the discharge flange of the pump.

The pump design shall be vertical single stage or multistage pumps with mixed or axial flow impeller design; broad hydraulic coverage provides best selection to meet specific operating conditions. Fabricated or cast iron underground discharge head, shaft and bearing combinations promote long life with options of open or enclosed line shaft construction. The inside diameter of the well and the screen pipe is 12 “.The outside diameter of the discharge head and shaft shall be 10”.

Hydraulic data for the pumps in different groups:

Group G1

- Max capacity/pump 200 m³/h
- Resp. total head approx. 105 mwc (meter water column)

Groups G3, G4 and G5

- Max capacity/pump 200 m³/h
- Resp. total head approx. 95 mwc

Group G2

- Max capacity/pump 200 m³/h
- Resp. total head approx. 90 mwc

The pump unit shall be equipped with at least following:

- Heavy duty, self-aligning axial thrust bearing system, capable of taking the negative axial thrust also
- Radial plain bearings with high wear resistance for trouble-free long term operation; pump bearings lubricated by the fluid handled, motor bearings by the motor's filling fluid.
- Replaceable, robust wear ring assembly at the pump stage.
- Wear-resistant mechanical seal for motor shaft

Motor features:

- water- or special fluid which can be driven by VHS motor VSS motor or diesel engine through right angle gear box.
- enclosure class at least IP68
- frequency 50 Hz
- rated voltage 400 V
- max. frequency of starts 15 / hour

Materials

- Bowl: Cast iron or stainless steel
- Impeller: Cast iron, Bronze or stainless steel
- Shaft: Stainless steel
- Discharge head: Cast iron or carbon steel

- Motor shroud: cast iron
- Scrwes, bolts and nuts : CrNiMo-steel

7.1.1.4 Well Fittings and Details

In addition to the main components of the wells, other fittings should be added to the manifold of each well which includes cyclone, bypasses, gates, sand monitoring, etc. Fig. 7.6 shows a typical manifold of the recovery well. The cyclone is designed based on a maximum pumping rate of 200 m³/hr. The design of the cyclone is shown in *Appendix 2*.

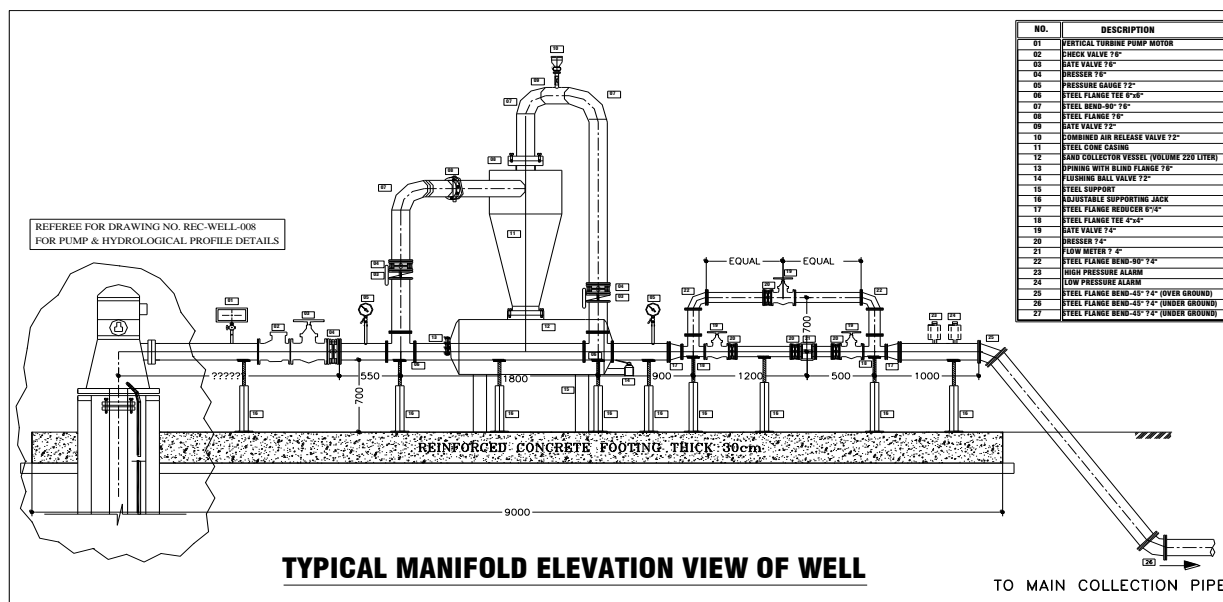


Fig. 7.6: Typical cross section of the well manifold.

7.1.2 Monitoring Wells

The monitoring wells are distributed in two rows: around 400 to 500 m from the infiltration basin and the second row around 1100 to 1200 m from the basin. The first monitoring well row should be located before the first row of the recovery wells in the direction of infiltration basin. The second row of the monitoring wells should be located after the second row of the recovery wells to check the quality of groundwater outside the recovery well area. Fig. 7.7 shows the location of the monitoring wells.

According to the distribution of the recovery wells, adequate number of observation wells is proposed to give accurate data about groundwater status. Ten new observation wells will be used for monitoring groundwater quality; in addition, 27 recovery wells and 5 existing monitoring wells will be used. The total number of monitoring wells will be 42. The water pumped to the irrigation network should also be monitored through samples of water from random farms taken to check the quality at the end use of water. Trunk lines, water tanks, and irrigation networks should also be monitored by taking random samples from each component.

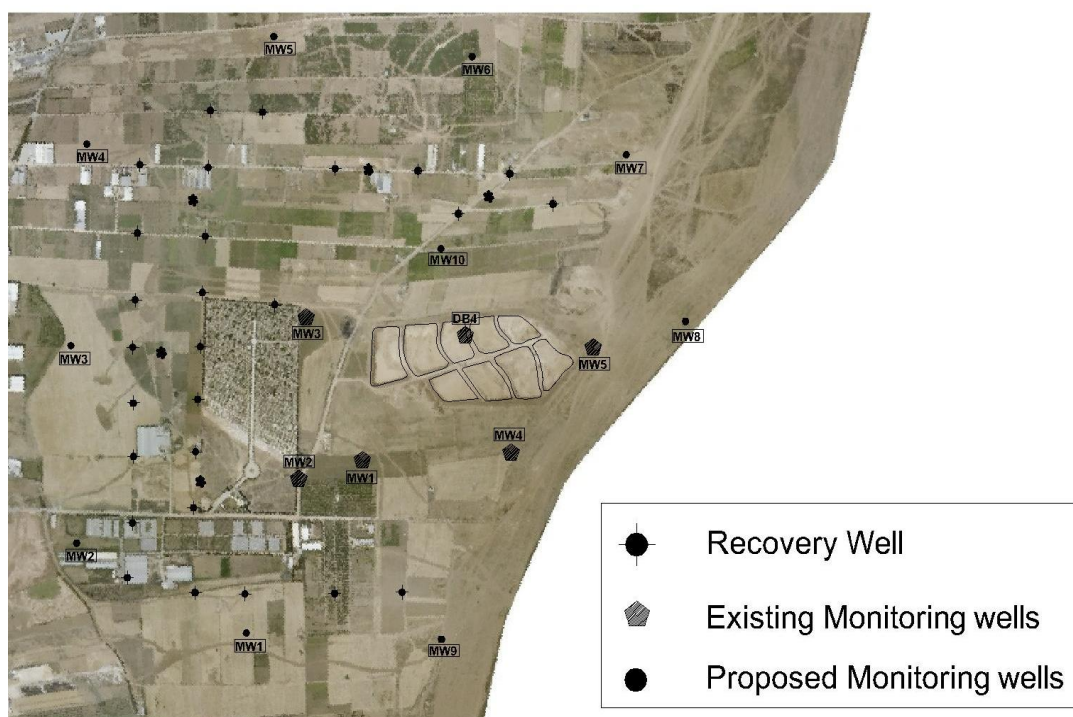


Fig. 7.7: Location of monitoring (observation) wells.

There are several parameters that could reflect the chemical and biological characteristics of the wastewater in the groundwater. The parameters shown in Table 7.2 are proposed to be measured and could be analyzed in Gaza Strip laboratories.

Table 7.2: Monitored parameters and frequency of monitoring.

Parameters	Frequency of Monitoring
Water Level	Monthly
pH	Four Times a year
TDS	Four Times a year
BOD	Four Times a year
COD	Four Times a year
DOC	Four Times a year
TC	Four Times a year
Ammonia as N	Four Times a year
NO ₃	Four Times a year
NO ₂	Four Times a year
T.N	Four Times a year
Cl	Four Times a year
Detergents	Four Times a year
F.C.	Four Times a year
Phosphorus	Four Times a year
Heavy Metals	Four Times a year
O ₂	Four Times a year
Oxygen Isotopes	Four Times a year
Mg	Four Times a year

Table 7.2 also shows the frequency of monitoring which indicates when the monitoring should take place. It is recommended that the groundwater level should be monitored monthly whereas the rest of parameters should be monitored quarterly. The frequency of monitoring seems to be extensive since the project is categorized as high risk project where extensive monitoring should take place, especially at the start of the project. These frequencies may be relaxed after 3 years from starting the operation.

A monitoring well is designed according to the design criteria section where the well consist of 12 in. casing and the inner pipe will be 4 in. and ends with a screen of 4.5 m length, which is very close to CAMP project recommendations, located under the groundwater table in the sand or gravel layer. The depth of monitoring well depends on the hydrogeological profile of the area. Hydrogeological cross sections used in the design of the recovery wells are used to locate the screen of the monitoring well. Fig. 7.8 shows typical design of a monitoring well and a hydrological profile.

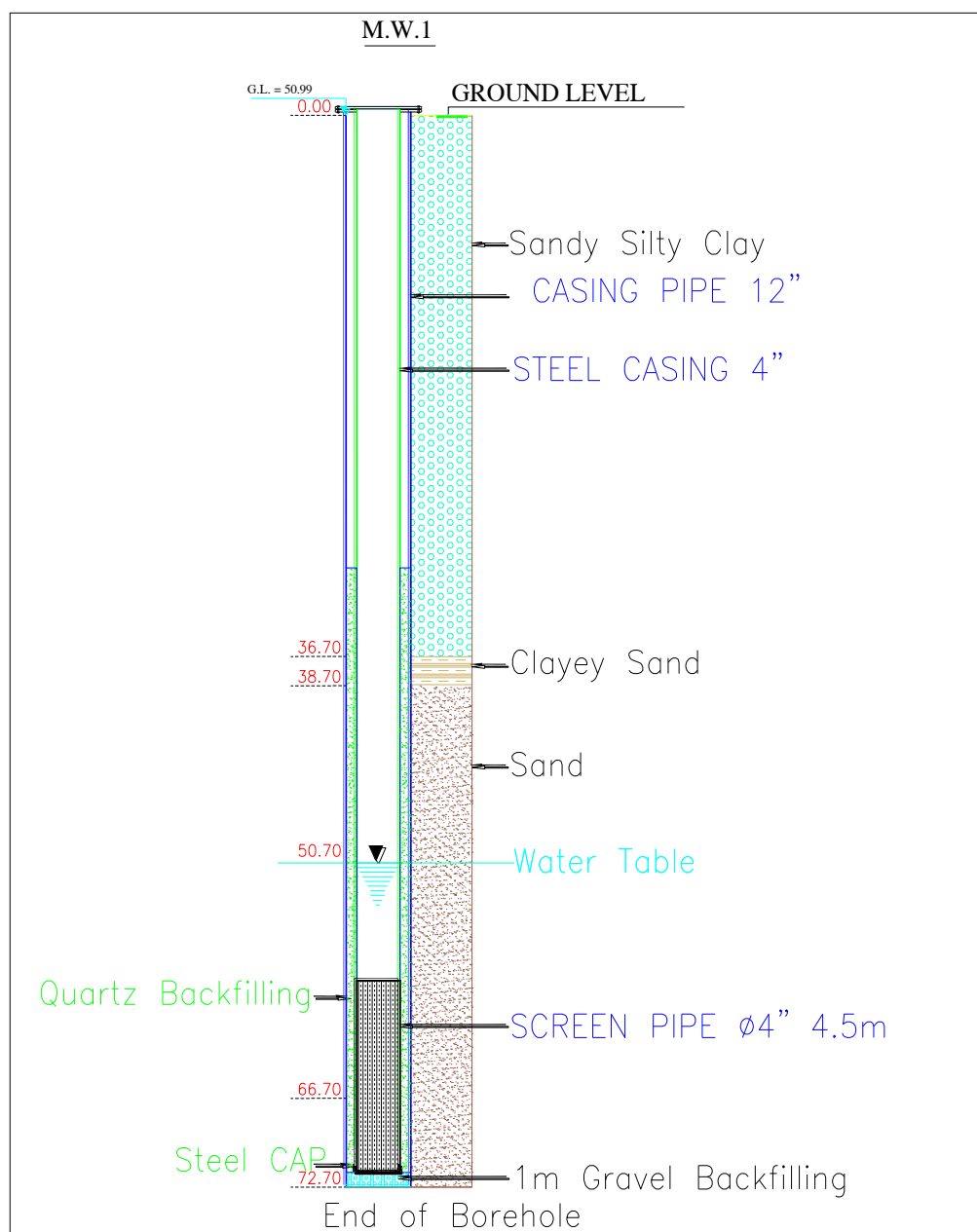


Fig. 7.8: Typical design output of a monitoring well.

7.1.3 Water Networks

The design of the water network follows the water flow system mentioned in design criteria section. The components of the flow system which are considered in the design stage consist of two parts:

1. Collection pipelines from recovery wells to water tanks.
2. Irrigation network including the trunk lines from the booster pumps to the farms.

7.1.3.1 Collection Pipelines from Recovery Wells to Tanks

The design output of the collection water network based on the adopted hydraulic model is shown in Fig. 7.9 and summarized in Appendix 2. The design output presents the material, length, and diameter of the pipe lines. The design output is based on modeling approach of which the calculation and other results such as velocity and pressure in each pipe are included in Appendix 2.

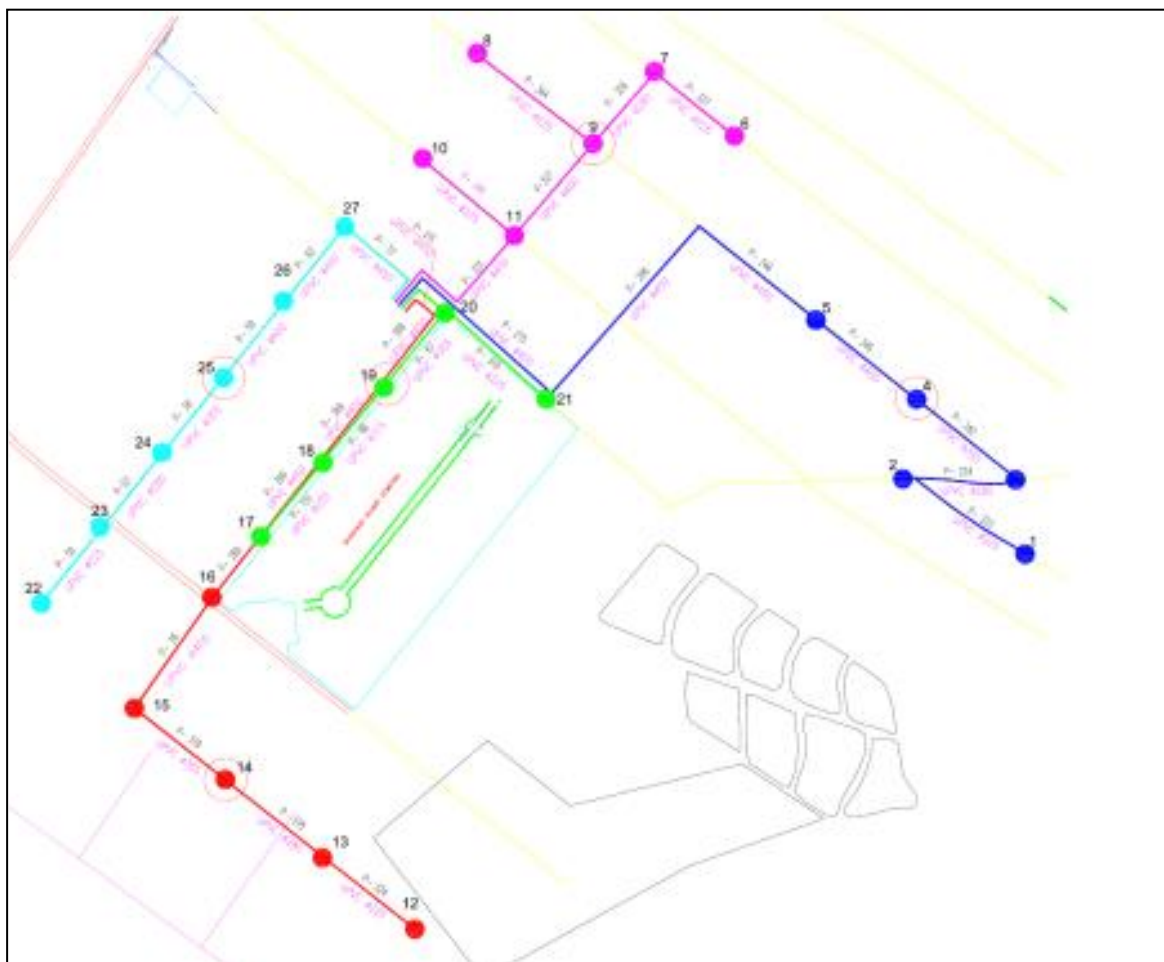


Fig. 7.9: Collection pipeline design outputs.

7.1.3.2 Irrigation Network

The design output of the irrigation network based on the adopted hydraulic model is shown in Fig. 7.10 and summarized in *Appendix 2*. The design output presents the material, length, and diameter of the pipe lines. The design output is based on modeling approach of which the calculations and other results such as velocity and pressure in each pipe are included in *Appendix 2*.

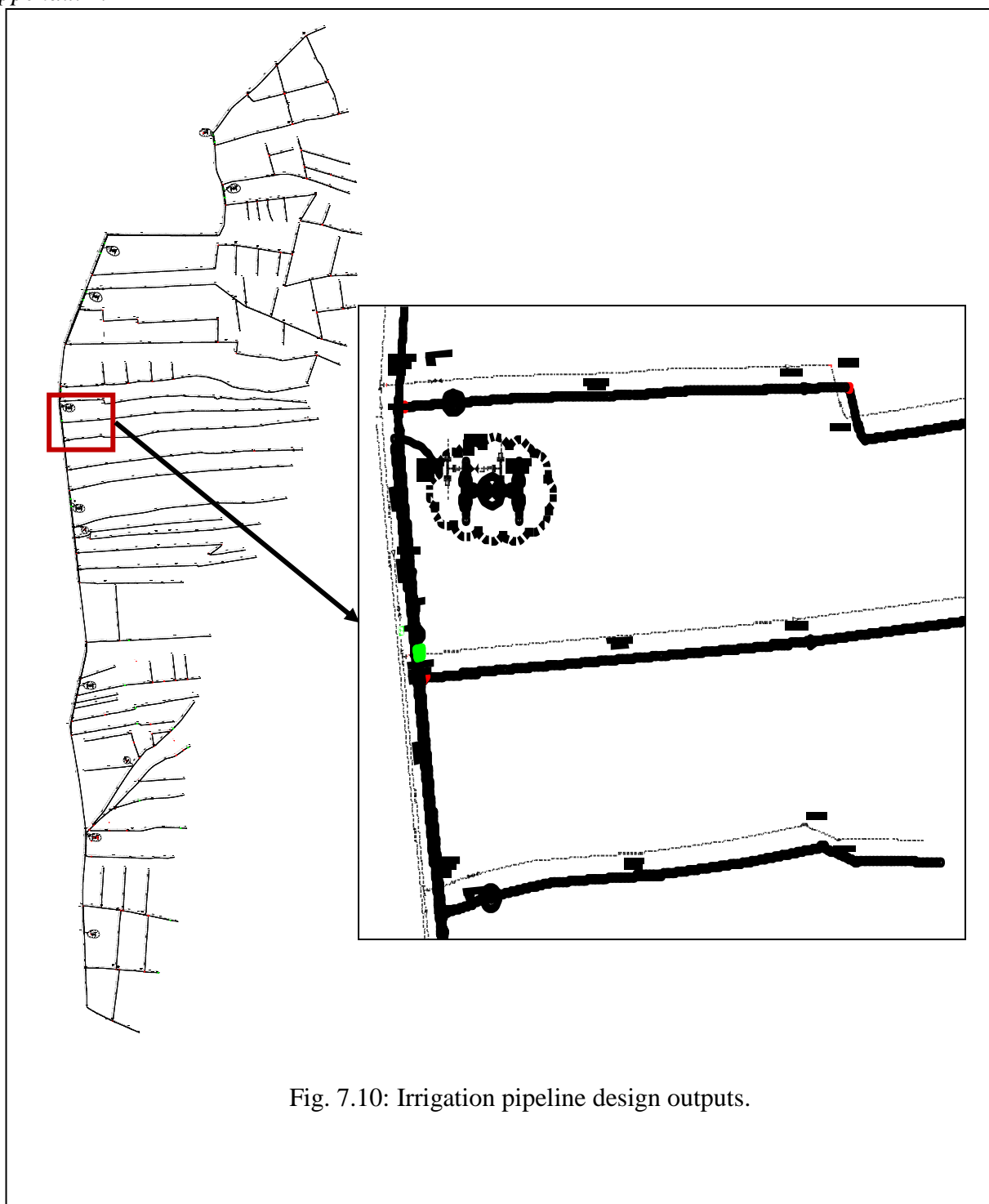


Fig. 7.10: Irrigation pipeline design outputs.

7.1.4 Water Tanks

The hydraulic design of the water tanks consists of determining the volume of the tank and the hydraulic dimensions of the piping system in the tank site. The piping system includes the inlet manifold and the manifold connecting the tank with the booster pump stations.

The storage capacities of the two tanks were determined in the design criteria section in this report. The two tanks of 4000 m³ each are shown in Fig. 7.11. There are two inlet pipelines from well groups C and D with a diameter of 450 mm to Tank 1 and three inlet pipes with diameter equal to 450 mm from well groups A, B, and E to Tank 2. The two tanks are connected by a balancing pipe of 900 mm diameter. Washout pipes of 200 mm diameter are located in two places in the bottom of each tank. Overflow of 200 mm is to be connected with washout pipes out of the tank with a gate valve on the washout pipe. The overflow and washout pipes from the two tanks are connected to each other with a pipe of 300 mm diameter. The feeder from each tank to the booster pump stations is 800 mm diameter with main gate valve as explained in the following booster pumping station design section.

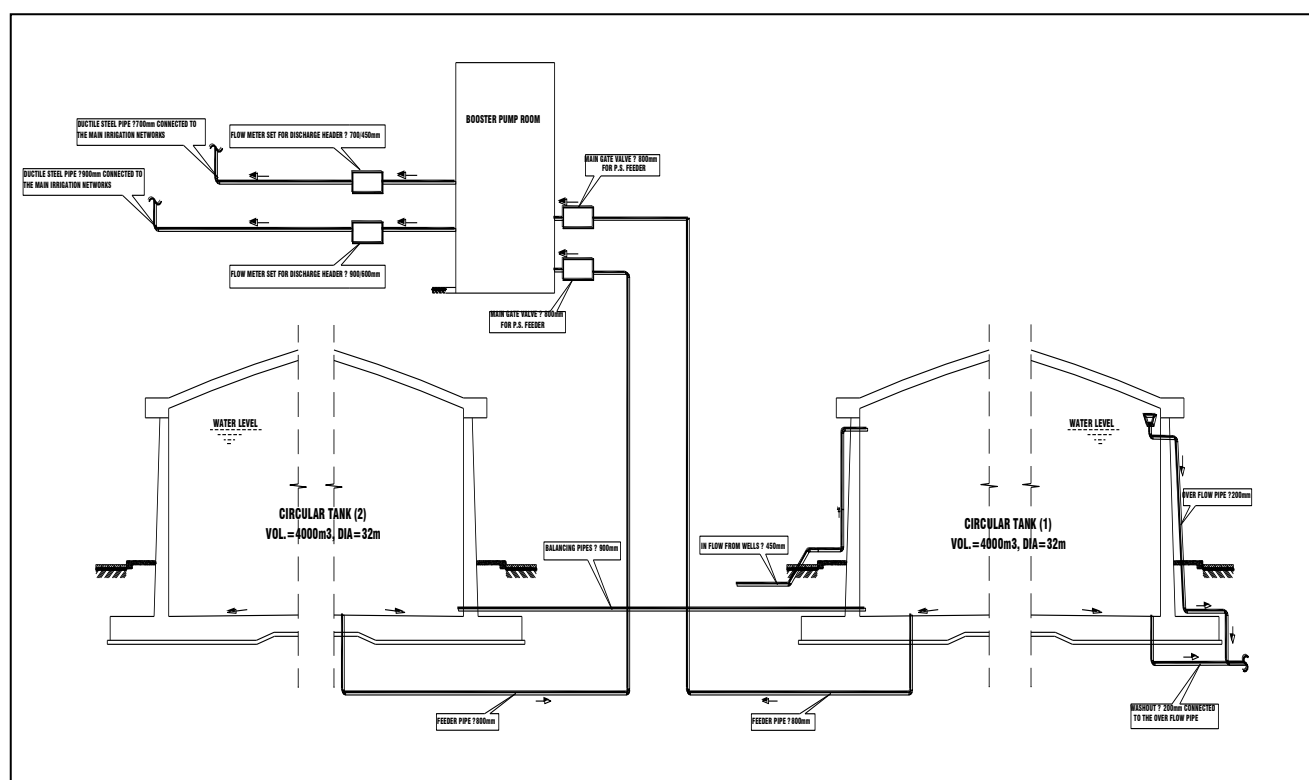


Fig. 7.11: Water tanks piping system.

7.1.5 Booster Pumping Station

The number and resulting size of pumps has been determined based on technical and economical factors. The greater the number of pumps, the smaller is the reduction of the total station capacity if one pump malfunctioned. This increases protection; however, it results in larger number of equipment and increased facility size. Flow and pressure demands at any point of the system are determined by hydraulic network analysis of the supply, storage, pumping, and distribution system as a whole. Supply point locations such as wells and storage reservoirs are known.

Selecting HPE pipe DN 900 mm will result in a velocity of 2.4 m/s which is less than the Vmax (3.0 m/s).

The booster pumps are located in a pumping hall together with the suction and pressure manifolds and with all necessary pipe works. The pumping station will serve both irrigation network, the south area with three irrigation zones and north area with six irrigation zones.

There are all together 8 of duty pumps and 2 of stand-by units, all similar pumps, installed parallel and pumping from a common suction manifold into a common pressure manifold. The pumps will serve the irrigation zones according to Table 7.3.

Table 7.3: The number of operating Pumps and Irrigation Zones

Irrigation zone	Number of pumps	
North A1.	5	Simultaneous pumping
South A2	3	
North B1.	5	Simultaneous pumping
South B2	3	
North C1.	5	Simultaneous pumping
South C2	3	
North D	8	
North E	8	
North F	7	

The pump size is selected based on the max. system flow rate 6000 m³/hr with the total dynamic head (TDH) 101 m wc. The number of duty pumps for each pumping mode is selected based on Table 7.4 determined by the consultant with pumping model software, and showing the pump discharge pressure for irrigation zones with different flows.

Table 7.4: The pumping flowrate and the pressure for each irrigation zone

Irrigation zone	Output pressure in booster station (bar) when output flow is (m ³ /h):											Max flow of the zone
	1000	1500	2000	2500	3000	3500	4000	4500	5000	5500	6000	
North A1	5,90	6,30	6,80	7,50								2382
South A2	4,60	4,70	4,90	5,00								2539
North B1	7,30	7,70	8,30	8,90								2571
South B2	4,80	4,90	5,10	5,30								2482
North C1	6,70	7,10	7,60	8,30								2269
South C2	5,20	5,50	6,00	6,40								2301
North D	6,90	7,00	7,20	7,40	7,60	7,80	8,10	8,50	8,90	9,20	9,70	5444
North E	6,40	6,50	6,70	6,90	7,20	7,50	7,90	8,40	8,90	9,40	10,10	5175
North F	5,90	6,00	6,20	6,30	6,50	6,70	6,80	7,10	7,40	7,60	7,90	5159

Booster pumps

Pos./marking	BP1, BP2, BP3, BP4, BP5, BP6, BP7, BP8, BP9, BP10
Number	8 nos + 2 nos as stand-by
Location	Booster pumping hall Hall floor level + 44.30
Type	Dry-installed, single volute end suction pump, horizontal assembly with separate pump, coupling and motor, installed on a common steel frame. Horizontal axially suction end. Horizontal discharge end directing 90° towards suction end. Acc. to ISO 5199. Frequency controlled.
Flow media	Soil-aquifer treated effluent.
<ul style="list-style-type: none"> SS-content Temperature 	max. 150 mg/l +10....+25 °C
Ambient temperature	max. +40 °C
Available NPSH	NPSH _a = 9,00 m
Installation	All pumps to be installed horizontally and parallel to each other on same floor level acc. to equipment layout drawings. Suction from a common suction manifold. Discharge to a common pressure manifold, divided in two parts.
Duty point	
<ul style="list-style-type: none"> Capacity Total head 	750 m ³ /h 101 m wc

Note: The pump performance curve (capacity vs. head) should be as slightly

curved as possible.

Design revolution speed ≤ 1500 rpm.

Max. revolution speed Acc. to manufacturer. **Note:** Must be given in the tender.

Initial power demand
(shaft power) approx. 250 kW

Flange pressure class PN16

The pump must withstand water hammer, where the pressure can rise to 16 bar.

Materials of the pump

- Casing High grade cast iron or CrNiMo-steel
- Impeller Cast iron, cast steel or CrNiMo-steel
- Shaft High tensile steel; parts which are in contact with water: acid proof steel or Duplex. steel (CrNiMo-steel)

Coupling Flexible spacer-type coupling. Bearing unit should be able to be separated from the pump without removing the electric motor.

Electric motor

- Type Air cooled cast iron squirrel cage motor. Minimum efficiency class IE2(EFF1)
- Voltage 400 V
- Frequency 50 Hz
- Initial recommended minimum power rating 315 kW
- Protection class IP55
- Temperature class B
- Insulated N-side bearings
- EMC cable gland
- Temperature control for pump bearings 1 pc PT-100 or equivalent temperature detector / each bearing
- Temperature control for stator windings 6 pcs PT-100 or equivalent temperature detector (2/phase) embedded in stator windings

Pump

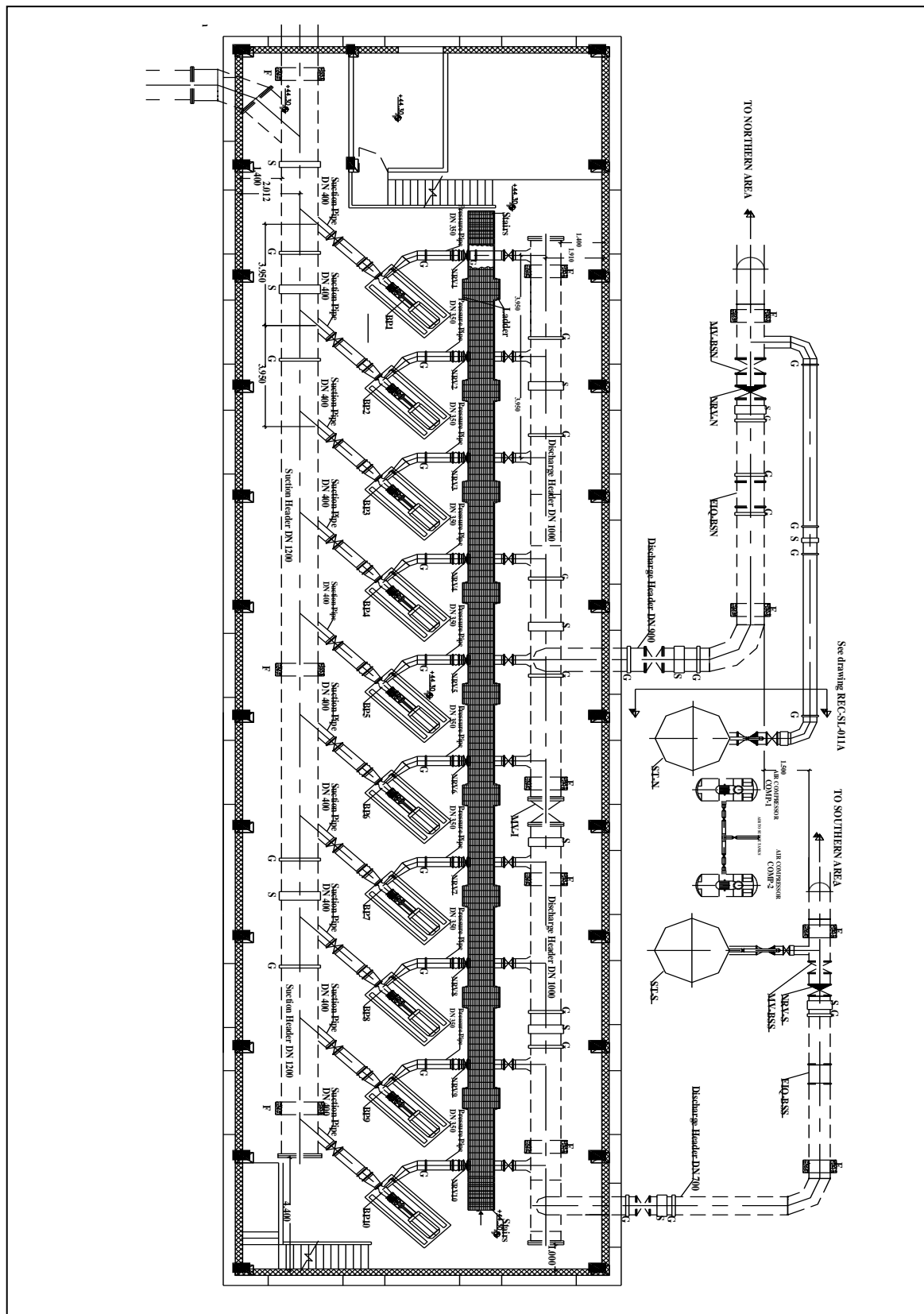
- Temperature control for bearings 1 pc PT-100 or equivalent temperature detector / each bearing
- Connections for pulsating control

Other features and requirements

In the extent of flow variation, pump's performance curve should be as slightly curved as possible and as near horizontal shape as possible.

Main dimensional drawing of the whole assembly, including the pump, coupling and motor installed on a common steel frame, must be submitted with the tender. Example of pump manufacturer and type: Sulzer Ahlstar A53-150 SO. The performance curves of the booster pumps are shown in figure

The layout and cross section of the pump station are shown in Figs. 7.12 and 7.13. Fig. 7.14 shows the pump curves which satisfy the required design capacities. The shown operation characteristics are for the selected pumps with max variable speed motor at speed equals to 2900 rpm. Table 7.3 summarizes the design input and output of booster pump stations. Fig. 7.15 shows the performance system curve of the pumps. Water hammer effect were considered in the design by adding air release valve and tow surge tanks. One of surge tank has a volume of 28 m³ added to the line 900 mm and another surge tank of 15 m³ was added to the line 600 mm. The calculation of the surge tanks sizes is shown in *Appendix 2*.



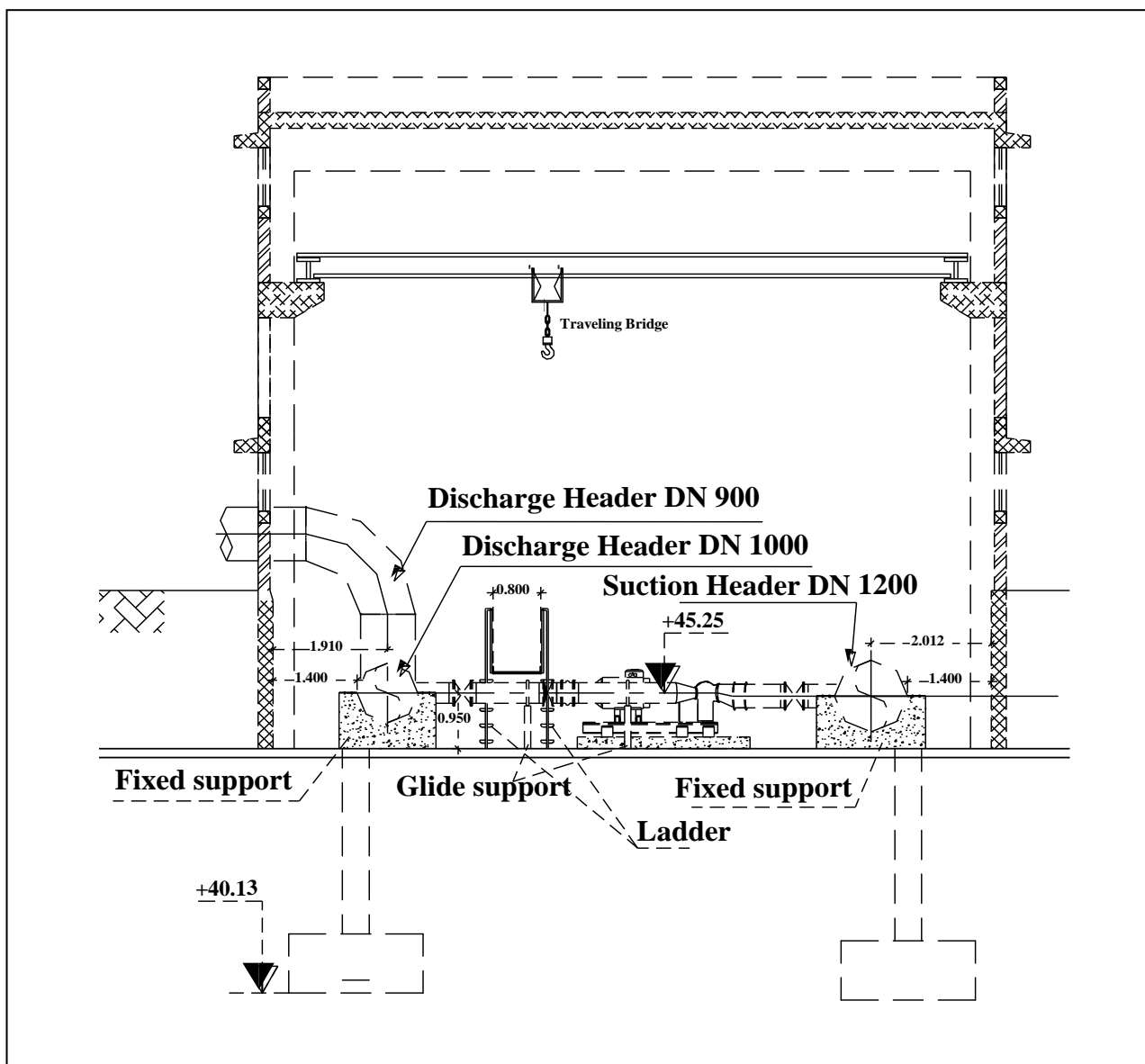


Fig. 7.13: Cross section in the boost pumping station.

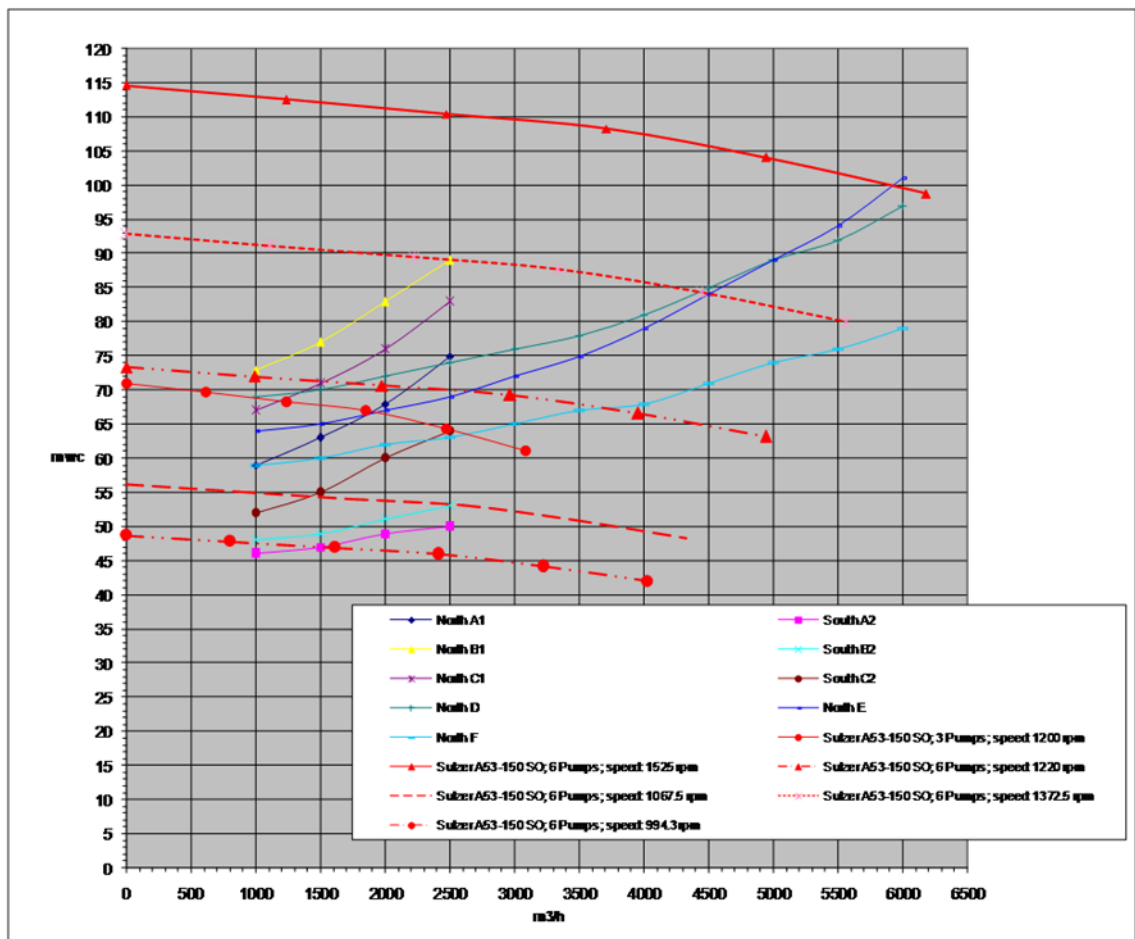


Fig. 7.14: Performance system curves for the booster pumps

7.1.6 Hydraulic Gradient Lines of the System

7.1.7 From Wells to the Tanks

Two hydraulic gradient lines (HGL) were drawn to check the adequacy of the design of the project components. The first one gathers the well pump and the pipeline from the well to the tanks. Fig. 7.15 shows HGL for the recovery scheme.

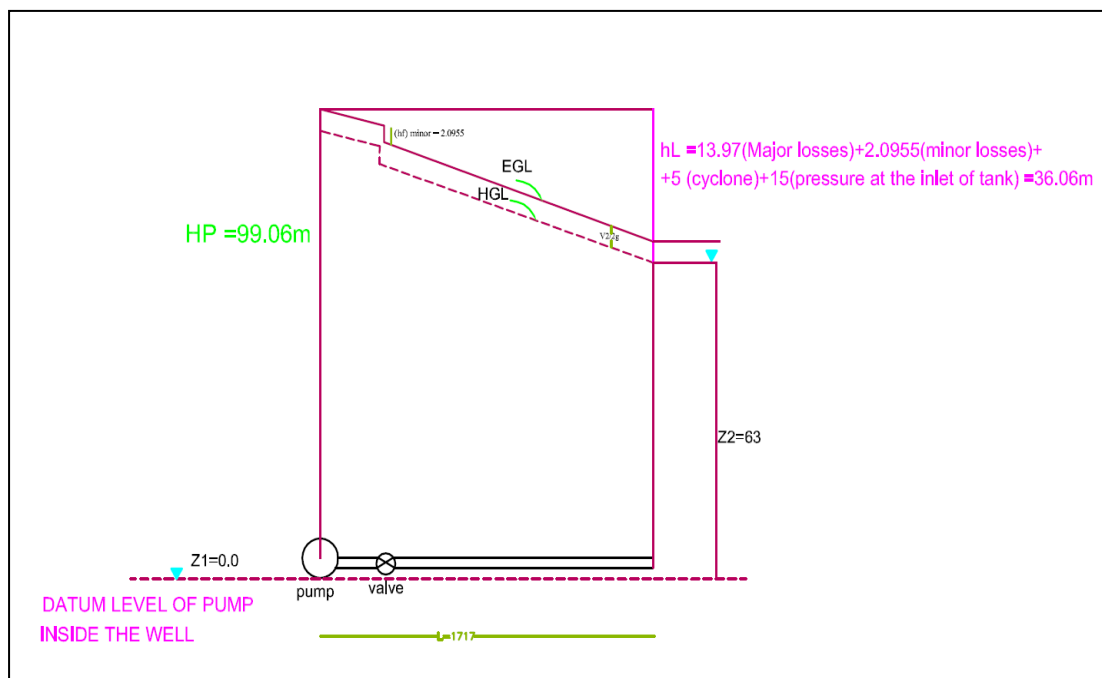


Fig. 7.15: HGL for Recovery Scheme

7.1.8 From Tanks to Farms

The second HGL was drawn for the reuse scheme which gathers the tanks, booster pump and irrigation pipelines. Fig. 7.16 shows typical HGL for the reuse scheme for the farms in Zone F.

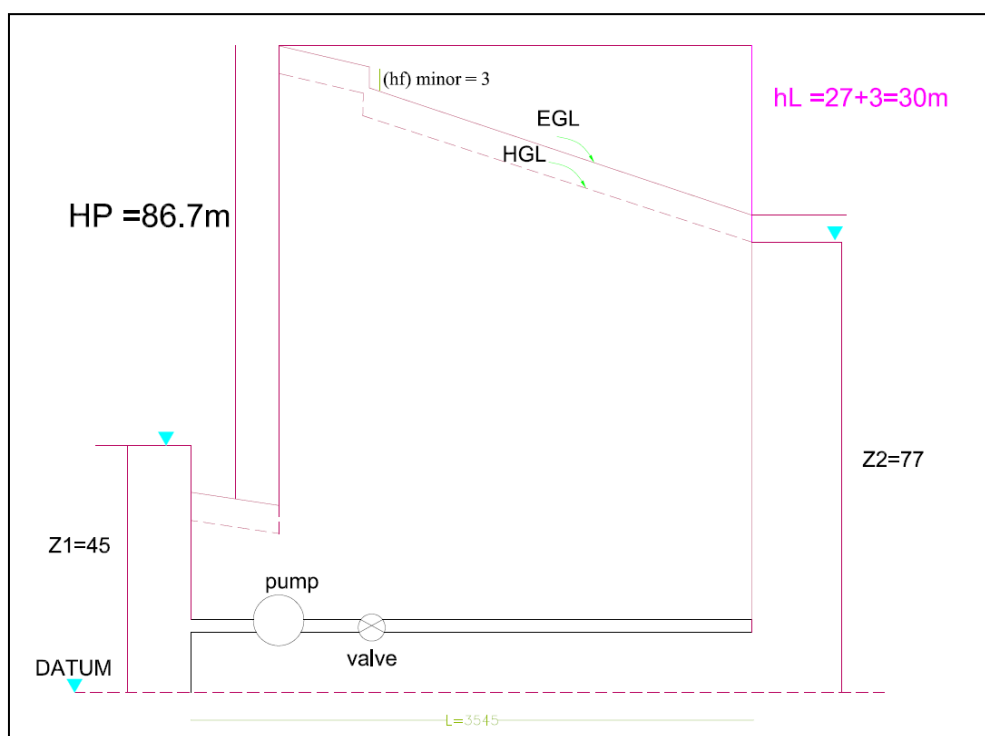


Fig. 7.16: HGL for Reuse Scheme

7.2 Electrical Design

7.2.1 Wells

There are 27 recovery wells to be constructed in an approximately 1.3 x 1.3 km² area. These wells are split into 5 zones (groups) according to their geographical distribution. These zones are named Zone A, B, C, D, E, and F as shown in Fig. 7.17 and in the corresponding drawing in *Appendix 3*. For each one zone there is a High-Voltage (22kV) node and an electrical service building.

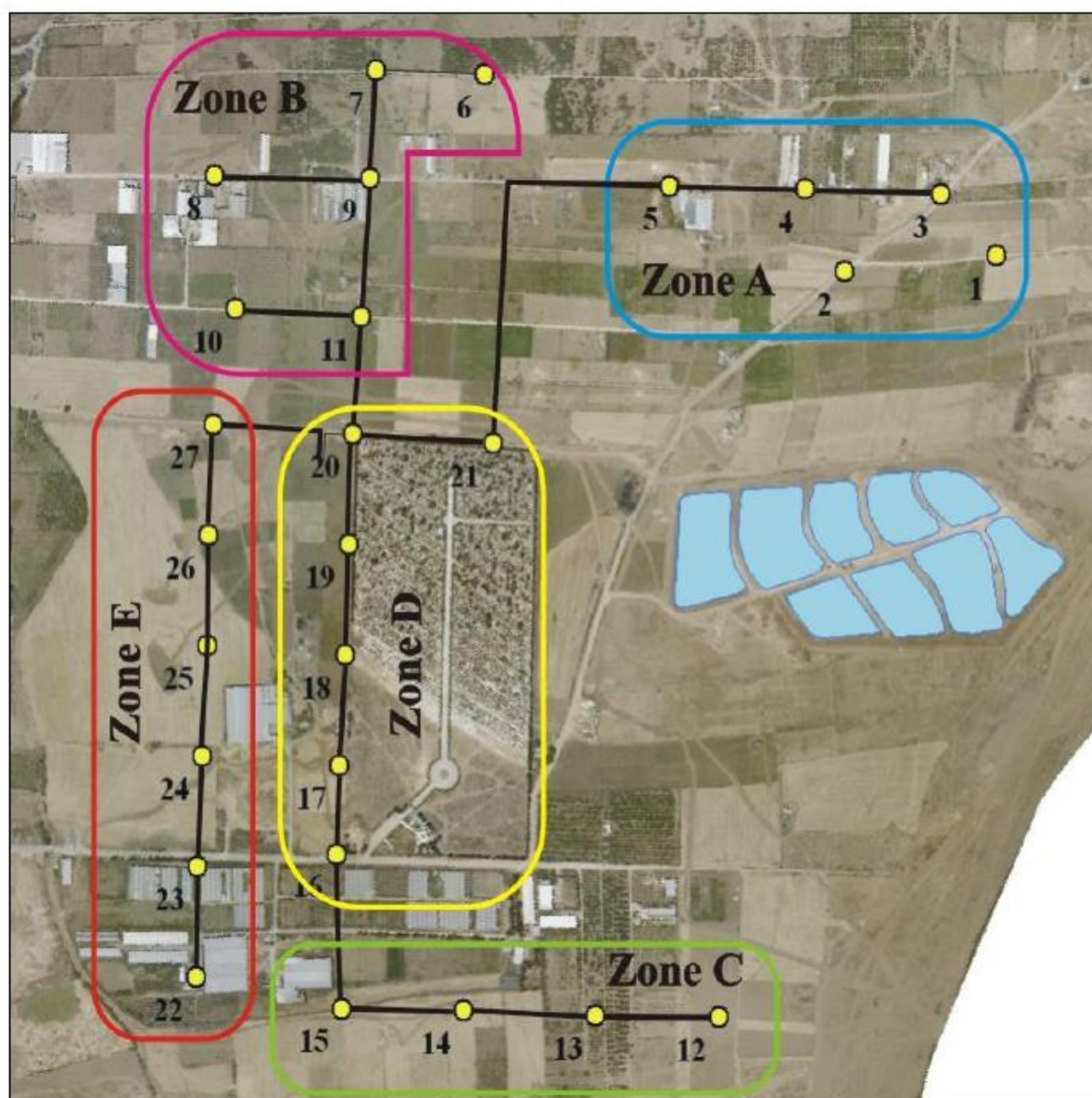


Fig. 7.17: The five well groups.

7.2.1.1 Transformer Compartment

Electrical loads of the wells are summarized in Table 7.5. A transformer station with a capacity of 630KVA should be installed to cover the power demand of each group. Each transformer station is connected to Main Distribution Board (MDB) which will be prepared to feed the control panels of the wells and pumps.

Table 7.5: load groups and subscriptions for recovery wells.

Group	load	Power Hp	Power kw	Group Current pf=0.9, (A)	Group Power kw	Transformer KVA	Subscription A	Generator KVA
A	R1	75	56	473	280	630	1000	500
	R2	75	56					
	R3	75	56					
	R4	75	56					
	R5	75	56					
B	R6	75	56	567	336	630	1000	500
	R7	75	56					
	R8	75	56					
	R9	75	56					
	R10	75	56					
C	R11	75	56	473	280	630	1000	500
	R12	75	56					
	R13	75	56					
	R14	75	56					
	R15	75	56					
D	R16	75	56	473	280	630	1000	500
	R17	75	56					
	R18	75	56					
	R19	75	56					
	R20	75	56					
E	R21	75	56	567	336	630	1000	500
	R22	75	56					
	R23	75	56					
	R24	75	56					
	R25	75	56					
	R26	75	56	567	336	630	1000	500
	R27	75	56					

7.2.1.2 Standby Generator

To meet the power demand of each group in case of failure of the main supply, a standby generator set with a capacity of 650 KVA is used. The generator sets shall be weather protective and sound attenuated housed under steel shed. Furthermore, the generator shall be complete with radiators, automatic transfer switch, batteries, fuel system, fuel storage tank and fuel tank containment.

7.2.1.3 Locations of Electrical Equipment

The location of most electrical parts for each group is placed inside a service building, which contains transformer room containing the transformer, H.V. switch gear, and panel room containing the main distribution board (MDB).

Cable trenches are used to make a connection between the several parts installed in these rooms (transformer, H.V. switchgear, MDB). The generator set will be located outside the service building. This requires that the generator set should be installed inside a sound and weather proof enclosure.

The suggested HV and LV networks and the single line diagram of the group distribution boards are shown in the relevant drawings in *Appendix 3*.

7.2.1.4 Motor Circuits

For the wells the starting method for the motors will be soft starting, so every motor circuit shall be provided with:

1. Solid state soft starter, which includes overload protection.
2. By-pass contactors, to give pumps transfer from the soft starter to the full voltage when the machine reaches the full load.

7.2.1.5 Power Factor Correction

An automatic system including step regulator, capacitors, contactors and control devices will be erected for each group to improve the system operation according to PEA recommendations.

7.2.1.6 Cables

All cables are planned to be underground and are dimensioned for 45°C ambient temperature and parallel cable installation. All cables to be calculated so they will not exceed 5% voltage drop of the nominal voltage at the switchboard when passing the full-load current. In all cables for 380/220V a PE-conductor of the same cross section as the leading conductors is provided. Cables for control equipment will be multi-core up to 37 x 1.5 mm² ended in centrally placed junction boxes with terminal racks for distribution to few-core cables to each electrical component. Where it is not possible to run the electrical cables in cable channels they will be placed in cable ladders.

7.2.1.7 Recovery Process Control

The water recovery process controller (PLC1) will communicate with the control boards of pumps of a certain zone via a remote terminal units allocated at the zone service building as shown in the relevant design drawing in *Appendix 3*.

For each recovery well, a control cable is connected to its associated zone RTU/Controller. This cable shall have the signals described in Table 7.6.

Table 7.6: Control signals of recovery wells.

Sn	Signal	Type
1	start/stop	Digital output
2	pump running	Digital input
3	pump alarm	Digital input
4	dry run protection interlock	Digital input

PLC1 controls the number of operating wells according to the percentage water level in the reservoir. It orders the pumps which is ready for operation in a waiting queue according to their running hours. A pump is added when the water level in the tank drops below a preset level. On the other hand a pump is removed (switched of) once the water level reaches another preset level. The pumps will enter and leave service line in first enter first out (FEFO) rule. This helps limiting excessive pump restarts.

The deference between the levels of adding a pump and removing a pump from the group of operating pumps must be reasonable so that it keeps an acceptable hysteresis for this operation. The level value thresholds will be set through the SCADA system. For example if the hysteresis is 10% of the reservoir capacity, then the start and stop levels as well as high and low alarm levels could be as shown in Table 7.7.

Table 7.7: Water tank preset levels.

Level	Default (%)	Reset pump	Set pump
L0	20	Low Level Alarm	
L1	63		27
L2	64		26
L3	65		25
L4	66		24
L5	67		23
L6	68		22
L7	69		21
L8	70		20
L9	71		19
L10	72		18
L11	73	27	17
L12	74	26	16
L13	75	25	15
L14	76	24	14
L15	77	23	13
L16	78	22	12
L17	79	21	11
L18	80	20	10
L19	81	19	9
L20	82	18	8
L21	83	17	7
L22	84	16	6
L23	85	15	5
L24	86	14	4
L25	87	13	3
L26	88	12	2
L27	89	11	1
L28	90	10	
L29	91	9	
L30	92	8	
L31	93	7	
L32	94	6	
L33	95	5	
L34	96	4	
L35	97	3	
L36	98	2	
L37	99	1	
L38	100	High Level Alarm	

In order to improve the process control, it is suggested to formulate the control task so that the number of pumps is also function of the rate of change of the water level along with the level itself. This results in some sort of PI control. However, the rate of change of the water level in the reservoir may be practically difficult to relay on due to the waves on the water surface. Therefore, comparable improvement may be achieved by forcing the number of operating pumps to be at least sufficient to substitute a preset factor (say 0.8) of the reservoir discharge rate. That additional control signal is easily acquired via the flow meter installed at the distribution network entry.

7.2.2 Booster Pumping Station

7.2.2.1 Subscription, Transformer, and Standby Generator Ratings

There are 10 booster pumps and they will be split into 3 groups as shown in Table 7.8. Ratings of transformers and standby generators are also indicated in the table.

Table 7.8: load groups and subscriptions for Booster pumps

Zone	Group	load	Power (hp) Hp	Power (KW) Kw	Group Current pf=0.9, (A)	Group Power (KW) kw	Transformer KVA	Subscription A	Generator KVA
F	6	P1	425	315	1595	945	1600	2000	1500
		P2	425	315					
		P3	425	315					
	7	P4	425	315	1595	945	1600	2000	1500
		P5	425	315					
		P6	425	315					
	8	P7	425	315	2127	1260	2000	2500	2000
		P8	425	315					
		P9	425	315					
		P10	425	315					

7.2.2.2 Water Distribution Process

The water booster pumping station will be controlled by PLC2. The rate of quantity of water which is preferred to be pumped and delivered to farmers has been already specified on daily basis along the year. This may suggest using the water flow rate as the control variable, i.e., adjust the pumping capacity to meet the planned demand. The pumping capacity is set by number of operating posters along with their speed. Frequency converters will be used to control the speed of the posters. Usually speed of one poster pump increases as demand increases. If speed reaches 100% and still not sufficient, the controller automatically starts the next pump.

Unfortunately, farmers may not precisely obey the recommended and planed irrigation schedule. This may result in undesirable large water pressure values. For example, if it is planned to start pumping at 7 o'clock and half of the farmers who are expected to start irrigation at the same time did not open their water taps, the water pressure will be almost doubled.

Alternatively, one may suggest using the water pressure at the output process as the control variable. This helps solving the problem of excessive pressure values, however, due to the finite capacity of the reservoir another problem will occur when harmers consumes water quantities

larger than planned. The 27 recharge wells even operated concurrently will not be able to substitute the discharged water from the reservoir. The water level in the reservoir will drop down to the low level threshold and all posters will be blocked.

According the previous discussion the operation will be based on fuzzy control rules in which the number of operating boosters along with their speed is dependent on the following control variables:

1. Planned irrigation schedule.
2. Water pressure at the distribution pipe.
3. Water flow at the distribution pipe.
4. Water level in the reservoir.

The controller will be responsible to automatically change the order of the pumps after certain amount of the running hours.

7.2.3 SCADA System

7.2.3.1 General Requirements and Concept

The requirements and specifications of the SCADA will include the followings:

- Hardware equipment definitions including programmable logic control system (PLC), control device (PC with peripherals) system, data transfer system
- Software program definitions shall include PLC program, control PC process-control program and reporting.
- Functional description of the PLC, control and reporting programs applications.

7.2.3.2 Detail Design of SCADA System

The detail design of the hardware system contains the lists and charts descriptions of the required hardware system.

The functional description contains the description of the whole system operations. The basic definitions shall be defined as user authority levels, event classes and priorities (alarm events, process events and operator events), historian and reports, display structure, basic parameters set points.

The reporting shall report all the information (flows, levels, etc.) in 1 hour cycle. Running hours of the pumps shall be recorded separately.

7.2.3.3 General functional descriptions of the plant

The whole irrigation system can be considered to cover two parts as recovery well pumping / water storing to water tanks and booster pumping to irrigation areas. The pumping time / day is limited from 8 to 12 hours.

The automation system shall control all the 27 recovery well pumps according to the levels of the water tank and according to output flow of the booster pumping to the reservoirs.

The booster pumping shall be controlled so that first the operators select the areas to be irrigated and start the pumps on the concerned pressure level to the selected area.

The information shall be transferred by fiber cables inside the booster and recovery well area and by gprs system to irrigation area targets.

Fig. 7.18 shows the general automation system. The detailed design of the SCADA system is attached in a spate report in ***Appendix 6***

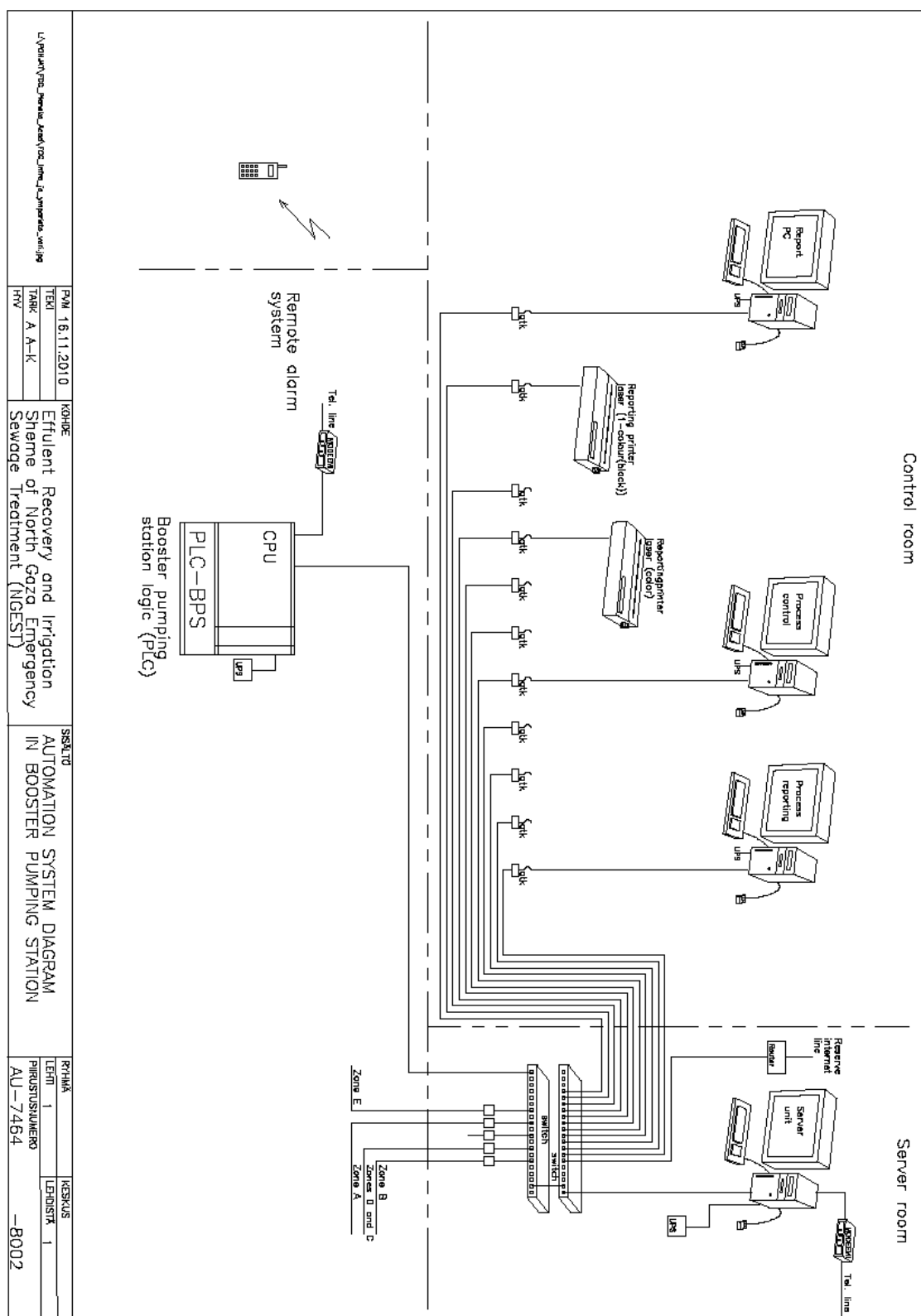


Fig. 7.18: General Automation System

7.3 Structural Design

This section includes the design of the various structures in the project. It describes the design approach, input data, and design results for each structure type, i.e. water tanks, booster pumping stations, and other buildings. The design calculations and results are included in *Appendix 2* and in the relevant structural drawings in *Appendix 3*.

7.3.1 Structural Design of Water Tanks

The following are the design considerations, inputs and results for the 4000m³ two identical water tanks.

7.3.1.1 Selection of Structural Elements

Foundations: The foundation type was selected such as to minimize differential settlement as this could lead to excessive cracking and loss of water tightness. Uniform settlement, if not excessive, was considered not to be harmful. It should be mentioned that the soil in the project area is clay.

Walls: In reinforced concrete walls, cantilevering from a board base or raft, which is also reinforced, have been used. The projection inside the reservoir of the wall base is called the 'heel' and the projection outside is called the 'toe'. Both the wall and the heel have been tapered, but tapering was not too large to avoid difficulties in construction. The length and thickness of the toe and heel have been adjusted to obtain the most economic design in regards to stresses on wall and foundation. The toe has been considered essential either for the reduction of maximum bearing pressure on the foundation under the tank when it is full of water, or for the development of adequate shear strength against sliding.

The minimum thickness for reinforced concrete walls should not be less than 225 mm. In this project the minimum thickness was equal to 400 mm. This is partly because with less thickness, there is some danger of leakage, but mostly because of the need to place two layers of steel reinforcement in the wall, and to maintain a minimum cover to the outer layer.

Roofing: The roofing system was selected to give fully enclosed reservoir structure that will not permit any entry of pollution. Air vents are needed to provide ventilation above water surface and displacement of air during emptying or filling operations.

Due to its large size the roof was not fixed to the tank wall. This is to limit shrinkage stresses in addition to the use of shrinkage reinforcement. The roof was simply supported to the walls. This consideration has also influenced the design of walls. Ring beam was used to resist the hoop tensile stress that develops at the bottom of dome.

7.3.1.2 Design Lifespan

The water tanks are designed to serve the needs for the planned number of years. The design life for properly maintained concrete tanks is typically assumed to be about fifty years. This span of life influences the type and level of loads in addition to applied factors of safety as included in the design codes and standards.

7.3.1.3 Loading Cases

In the structural design of the tank different types of loads and load combinations have been considered. Dead loads have been determined from known densities or unit weights of building materials. Live loads have been determined by rational consideration of expected live loads. Earth pressure has been calculated based on the considered soil unit weight.

The following three loading cases are considered in the design of water tanks when applicable.

- 1) Water pressure (inside the tank);
- 2) Lateral earth pressure (outside the tank) for underground tanks;
- 3) Lateral water pressure (outside the tank) for submerged underground tanks.

However, only the first case is relevant to the water tanks in this project.

7.3.1.4 Design Method

From a structural point of view the tanks have been analyzed and designed as ground circular tanks. The roof is designed as shallow dome that is subjected to gravity and lateral loads. The walls are subjected to fluid pressure. These pressures induce stresses in the foundation. The elements of the tank are analyzed and designed for a loading case resulting from water pressure inside the tank. **SAP2000** Version 14 was used in the design. Moreover, the results were checked against hand calculations using conventional methods as explained in *Appendix 2*:

7.3.1.5 Results of Structural Design of the Water Tanks

Design Data:

- Material Strengths: $f_y = 400$ MPa, $f'_c = 30$ MPa.
- Geometry: The design dimensions of the tank are as shown in *Appendix 2*. The wall height inside the tank = 5.5 m and the inside diameter = 32 m. The rise of the dome is 2.5 m measured from the top face of a 0.5 m depth ring beam. The inside radius of the dome = 52.41 m.
- Structural System: The roof slab of the tank is of dome shape ends with ring beam that is supported on the tank walls. The walls of the tank retain the water pressure in addition to carrying the roof load. The walls are supported by a continuous strip footing which is connected to the remaining footing as shown in *Appendix 2*.
- Loads: $L = 2.5$ kN/m² (for dome), Soil unit weight = 18 kN/m³, Water = 10 kN/m³, Wind velocity = 120 km/hr.
- Bearing capacity: $q_{all(net)} = 100$ kN/m².

Design Results for the Dome:

- The dome is of variable thickness (t) equals to 250 mm at the ring on top of wall and 200 mm at crown.
- $\theta = 0^\circ$ at crown and $= 17.76^\circ$ at ring.
- The design results of the dome showed that the hoop reinforcement requirements are constant for the whole dome and are equal to minimum reinforcement. Two steel layers

are used one at the top and the other at the bottom of dome slab. For each layer the reinforcement in each direction is $5\Phi 10/\text{m}$. As for the meridian reinforcement, $5\Phi 10/\text{m}$ are required as bottom reinforcement, $5\Phi 10/\text{m}$ are required at top in the middle and $5\Phi 12/\text{m}$ at the edges of the dome.

- The design results of the ring beam indicated a ring cross section of 800 mm (width) \times 500 mm (depth). The reinforcement is equal to $30\Phi 22$. The stirrups are of rectangular closed shape and equal to $2\Phi 10@200\text{mm}$ (two rectangles).
- Main design calculations and results are shown in *Appendix 2*. Design details are shown in the relevant structural design drawings in *Appendix 3*.

Design Results for the Wall

The design results showed that the concrete shear strength determined based on wall thickness was adequate to resist applied shear forces at critical sections.

The thicknesses have also been found adequate to result in sections in which the stress level in reinforcement is relatively low to control crack width. The maximum crack width has satisfied the serviceability limit state method in the ACI. Nevertheless, the surfaces of the tank shall be adequately protected against adverse environmental factors using waterproofing agent.

- Wall thickness at bottom = 0.5 m and at top = 0.4 m.
- Inside vertical reinforcement = $10\Phi 14/\text{m}$ at bottom and = $5\Phi 12/\text{m}$ at top.
- Outside vertical reinforcement = $5\Phi 12/\text{m}$ at bottom and = $10\Phi 12/\text{m}$ at top.
- Transverse reinforcement at inside and outside surfaces = $5\Phi 14/\text{m}$ at each face.
- Crack width ranged from 0.078 to 0.105 mm which is within the acceptable limits.

Design Results for the Foundation

- The design results showed that the thickness underneath the wall and under the middle floor were equal to 600 mm and 400 mm (without the 1% slope), respectively.
- The main flexural reinforcements for the whole foundation were equal to $10\Phi 12/\text{m}$. It should be mentioned that the extra reinforcement was needed to control crack width rather than resisting moments.
- The crack width ranges from 0.09 to 0.1 mm.

7.3.2 Structural Design of Booster Pump Station and Associated Facilities

The booster pump station building is a normal building from a structural design point of view. It is subjected to typical loading conditions and thus has been designed following the ultimate limit state method in the ACI code. Special attention has been given to connection between the elements that carry the moving parts. The building includes also a steel crane girder to carry and move pumps when necessary. The various structural concrete members have been designed as follows:

7.3.2.1 Design Data

- **Geometry:** The dimensions of the rectangular-shaped building are 48 m \times 12.5 m.

- **Structural System:** The structural system used is moment resisting reinforced concrete frames that run in the short direction of the building. This system is suitable for relatively long spans and provides flexibility in operating the facility since it does not include interior columns. The slab is one-way ribbed that is supported on the dropped beams of the frame. The foundations of frame columns are spread and thus assumed simply supported, since soil is not rigid and cannot provide necessary rigidity for fixation. In order to provide stability of the building in the other direction, i.e. transverse to the frame plan, drop beams have been provided at slab and corbel levels. The steel crane is supported on the crane rails which in turn are supported on the frame corbels. This arrangement allows reaching any point in the booster pumping station.
- **Loads:** $L = 2.5 \text{ kN/m}^2$ in addition to weight of equipments.
- **Material Strengths:** $f_y = 400 \text{ MPa}$, $f'_c = 20 \text{ MPa}$.
- Normal flexural theory is used for the flexural design of the slabs, beams, and other flexural members.
- The ACI shear design method is used for shear design.

7.3.2.2 Design Results of Slabs

The slabs are ribbed continuous slab with adequate thickness to control deflection based on section 9.5 of the ACI code equal to 270 mm. The thickness has been determined based the geometry and configuration of the building. The continuous slabs are subjected to gravity dead and live loads. The slabs are designed for the flexure and shear using traditional procedures. The widths of the ribs are taken equal to 120 mm. Typical flexural reinforcement were $2\Phi 12/\text{rib}$.

Design results are shown in *Appendix 2* and design drawings are shown in the relevant structural drawings in *Appendix 3*.

7.3.2.3 Design Results of Frame

The frame columns have a 400 mm (width) \times 600 mm (depth) cross section with maximum reinforcement equals to $(7\Phi 16 + 5\Phi 20)$. The frame beams have cross section of a 400 mm (width) \times 1000 mm (depth) with maximum reinforcement equals to $12\Phi 20$. $\Phi 10$ stirrups were used to resist shear in both the columns and beams.

Design calculations and results are shown in *Appendix 2* and design drawings are shown in the relevant structural drawings in *Appendix 3*.

7.3.2.4 Design Results of Crain Girder

Crane girder design in the booster pump station was carried out using equivalent static load that accounts for the dynamic effects for the moving load. A steel crane girder equivalent to W-shaped (W18 \times 106) was used to carry the applied moving loads of pumps in the booster pump building. The crane girder is moving on the side rail of equivalent W-shaped (W10 \times 45).

The details of design are shown in the relevant structural drawings in *Appendix 3*.

7.3.2.5 Footings

The sizes of footings have been calculated based on the allowable bearing capacity of the soil as determined from the soil characteristics. The size of footing has been determined such as to ensure against shear failure in soil and excessive settlement. The depth of footings has been determined based on the wide and two-way shear strengths of concrete. The reinforcement is determined based on the applied loads on the footings. Footings have dimensions of 3 m × 2.3 m × 0.5 m. Typical flexural reinforcements are equal 10Φ14/m.

7.3.2.6 Ground beams

Ground beams of 400 mm×500 mm and of typical top and bottom flexural reinforcement equal to 3Φ14 were used to connect footings with each other in the two directions and to carry walls on top of them.

7.3.3 Design Approach for Service Building and Other Structures in the Project

The project includes other structures such as electrical building, service buildings, guard room, well rooms and buildings, manholes, chambers, etc. From structural point of view, these are normal buildings and thus their structural design was carried out using normal design methods under applied loading cases which were discussed in the Design Criteria Section of this Report. Simple structural system was used for these buildings. This system consisted of continuous slab resting on continuous beams that in turn rest on columns. Concrete of normal strength (**B300**) and Grade 420 reinforcing steel bars was used in the design.

The plan and design drawings of these buildings are shown in relevant drawings in *Appendix 3*.

7.4 Main Design Drawings

The following design drawings are for the main project components. It should be mentioned that full drawings are included in *Appendix 3*. Table 7.8 shows main design drawings.

Table 7.8: Design main drawings

Item	Description	No. of drawings
1.	Booster site layout	7
2.	Circular tank	5
3.	Mechanical building	3
4.	Recovery wells	7
5.	Electrical building	3
6.	Guard room	1
7.	Irrigation net work	4
Total		30

8 IMPLEMENTATION STAGES AND COST PREDICTION

8.1 Investment Cost

Table 8.1 includes a summary of the capital cost for the main items. The bill of quantities (BOQ) of the work has been prepared and submitted as a part of the bidding documents of the project. BOQ includes the breakdown for this cost.

Table 8.1: Summary of the capital cost for the main items.

Item No.	Description	Total Rate (USD)
1	General Items	262,400
2	Circular Tank 4000 M3 (2 Tanks)	1,012,010
3	Booster Site (Civil)	281,022
4	Mechanical Building (Mech)	2,285,150
5	Electrical Building	225,690
6	Guard Room	10,622
7	Recovery Wells (27 Well)	2,833,917
9	Monitoring Wells (5 Wells)	222,600
10	Well Networks (around 6.7 Km)	674,190
11	Instrumentation & Automation Scada System	1,961,250
12	Electrical Works	2,885,897
13	Irrigation Network (around 128 Km)	15,649,730
Grand Total		28,304,478

8.2 Operation and Maintenance Cost

The operation and maintenance cost has been calculated based on manpower, power consumption, maintenance and repair works, consumables, etc. the cost of operation and maintenance is calculated based on the percentage of generator use. The cost of operation and maintenance will range between 1,3 to 2,35 Millions USD/ year as shown in Table 8.2. *Appendix 7* presents the calculation sheets of operation and maintenance cost where the operation manual of the system which includes the recovery wells and booster pumps scheme is presented in *Appendix 6*.

Table 8.2: Operation and Maintenance Cost

O&M Costs breakdown	Different assumptions for generator use										
	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
Manpower	180,000	180,000	180,000	180,000	180,000	180,000	180,000	180,000	180,000	180,000	180,000
Power consumption	956,987	1,062,760	1,168,532	1,274,304	1,380,077	1,485,849	1,591,621	1,697,394	1,803,166	1,908,938	2,014,711
Maintenance & repair works	83,345	83,345	83,345	83,345	83,345	83,345	83,345	83,345	83,345	83,345	83,345
Consumables & miscellaneous	76,960	76,960	76,960	76,960	76,960	76,960	76,960	76,960	76,960	76,960	76,960
Total O&M cost USD/year	1,297,292	1,403,065	1,508,837	1,614,609	1,720,382	1,826,154	1,931,926	2,037,699	2,143,471	2,249,243	2,355,016

8.3 Proposed Stages and Contracting Packages

Tentatively two implantation stages are proposed for carrying out the project for the 2015 design year. The first stage will include 15 recovery wells and concerned connection pipes, the civil works within the booster pumping station; however only one water tank will be constructed, 5 booster pumps, irrigation network for 5000 donums and 5 monitoring wells. The remaining works are to be implemented during the second stage. At year 2013 of the first stage the number of wells should be 21 wells the 15 wells will not be able to recover the 28,000 m³/day that will be pumped by year 2013. If the recovery system (6 wells, 1 booster pump and 1 tank) is not extended then the recovery system will not be effective.

In addition, the pollution plume will escape from the wells and the number of threatened agricultural and municipal wells will be increased. The cost for the first stage is around **11,969,344 USD**. Table 8.3 shows the cost of the main components for the first stage. The second stage will include the remaining works. The cost for the second stage is around **16,335,133 USD**.

Table 8.3: Summary cost of the main components for the first stage.

Item No.	Description	Total Rate (USD)
1	General Items	131,200
2	Circular Tank 4000 M3 (1 Tank)	523,695
3	Booster Site (Civil)	281,022
4	Mechanical Building (Civil + Mech)	1,669,400
5	Electrical Building	225,690
6	Guard Room	10,622
7	Recovery Group Wells (4 Wells)	587,751
8	Recovery Single Wells (11 Wells)	1,118,478
9	Monitoring Wells (5 Wells)	111,300
10	Well Networks (around 4.2 Km)	453,310
11	Instrumentation & Automation Scada System	1,321,250
12	Electrical Works	1,885,897
13	Irrigation Network (around 35 Km)	3,649,730
Grand Total for Phase 1		11,969,344

It is also proposed to use two contracting types for the first stage; ***Supply and Install*** for the recovery wells, connection pipes up to the water tanks and the booster pump station. The other contract is the ***Small Works*** contract for the irrigation network. The cost for the ***Supply and Install*** contract is around **8,254,014 USD** and for the ***Small Works*** contract is **3,715,330 USD**.

It should be mentioned that a technical review and re-design was carried out to investigate the technical validity of these stages and contracts. Special attention was given to satisfy mechanical and hydraulic limitations such as minimum and maximum velocities and pressure, etc. Figure 7.19 shows the recovery wells and piping system and monitoring wells to be implemented in the first stage, Figure 7.20 shows the layout of the five booster pumps, and Figure 7.21 shows the project components in the first stage.

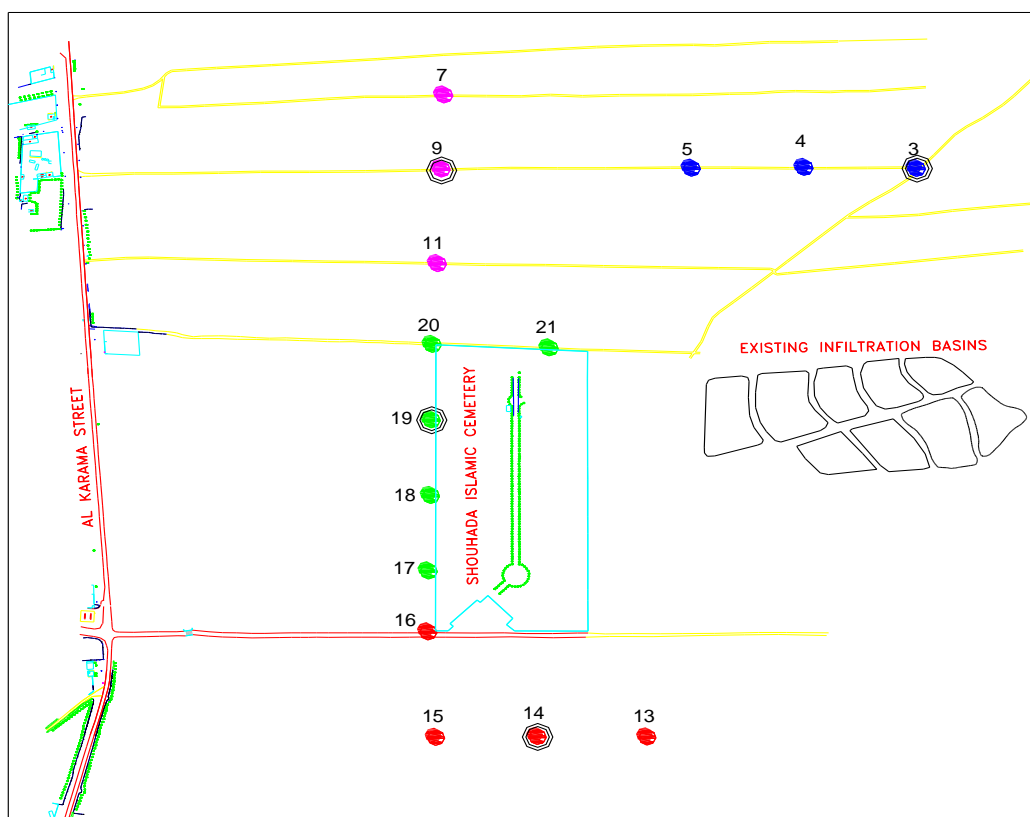


Figure (7.19) Recovery wells in the first stage

Figure (7.20) Layout of the five booster pumps in the first stage

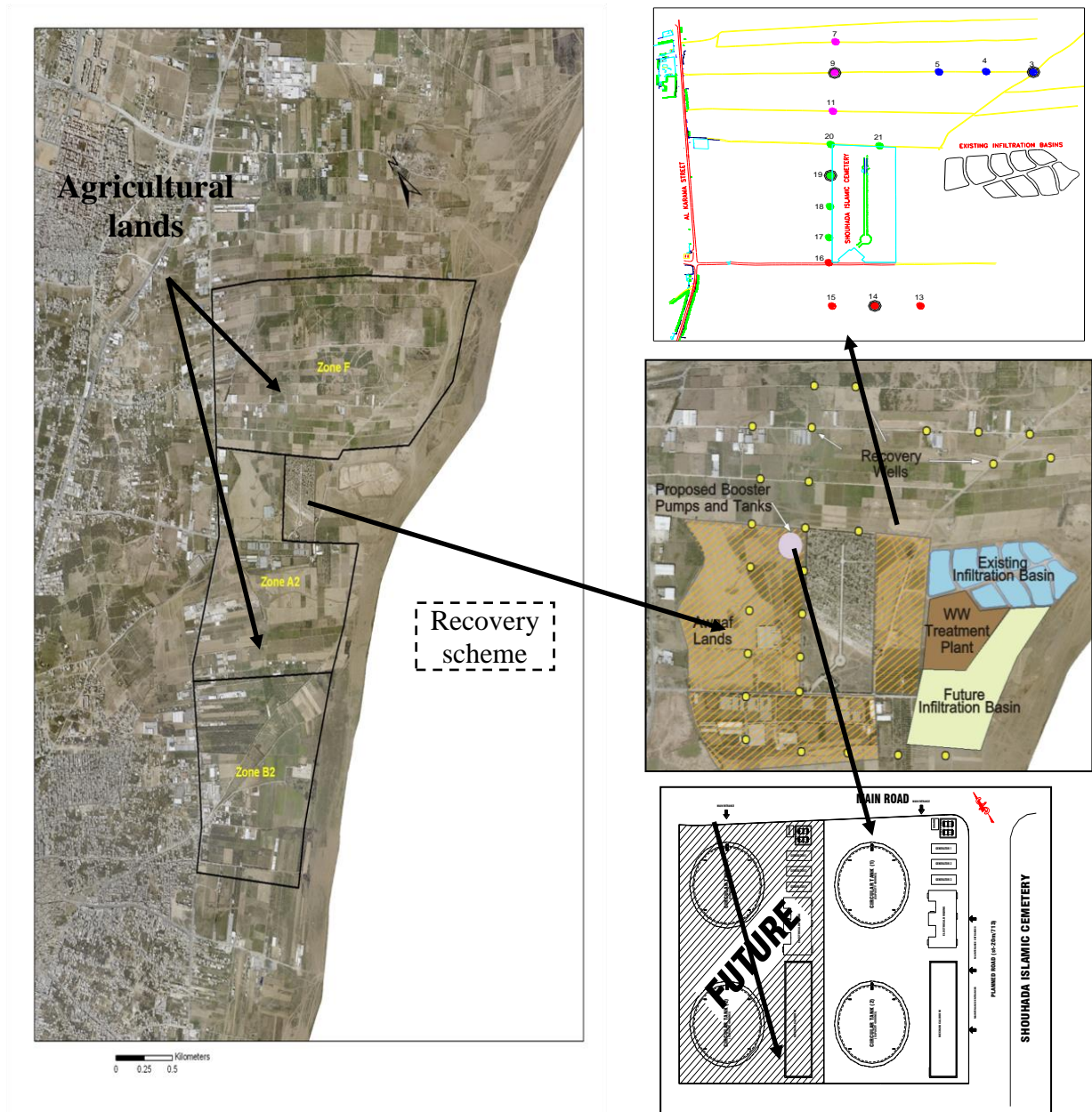


Figure (7.21) layout of the project components in the first stage

9 FINDINGS AND RECOMMENDATIONS

9.1 Concluding Remarks

1. The system design is very complex of multidisplinary nature that required in depth studies before actual commencement of the design of the physical project components. The studies included agricultural concerns, groundwater modeling, irrigation preference, etc.
2. The design of recovery scheme, especially the recovery wells required intensive data from the field and groundwater modeling. Therefore, hydrogeological investigations and pumping tests were performed in order to obtain necessary information. The groundwater model enabled the planning of the recovery scheme where the recovery wells were able to capture all of the pollution. The total number of wells was 27 placed in two rows. The first row of wells will capture most of the pollution since they are operated all the year, while some of the second row wells will be turned off in the winter months. The operation manual will be prepared and the operation of the wells will be recommended.
3. The design of the reuse scheme was based on the amount of recovered water which was equal to 35,600 m³/day plus 10% extra to ensure groundwater direction towards the recovery wells. The components of reuse scheme included two 4000 m³ water tanks each, ten variable speed booster pumps and associated facilities, and six irrigation zones of about 2500 donums each (Total agricultural area around 15,000 donums).
4. In order to accelerate the completion of the project, the design report included detail design drawings to obtain client's comments in the next task.
5. Existing agricultural wells within the recovery scheme need to be stopped to allow the controlled operation of the recovery and reuse project. The project will serve the concerned formers more efficiently.

9.2 Recommendations

1. Operation manual by contractor needs to be carefully prepared in order to ensure proper implementation of irrigation scheme, control and SCADA systems accordance with developed objectives system maintenance.
2. It is necessary to accelerate the implementation of the two stages of the project packages in order to capture ongoing pollution and allow extending the amount of infiltrated water for the design year 2015.
3. Four tender packages are recommended for the implementation as follows:
 - a. First Stage- Package 1 (Supply and Stall): 15 recovery wells and concerned connection pipes, the civil works within the booster pumping station, five booster pumps, one 4000 m³ water tank and 5 monitoring wells.
 - b. First Stage- Package 2: (Small Works): irrigation network for 5000 donums.
 - c. Second Stage- Package 1 (Supply and Stall): 12 recovery wells and concerned connection pipes, the remaining civil works within the booster pumping station, five booster pumps, one 4000 m³ water tank and 5 monitoring wells.

- d. Second Stage- Package 2: (Small Works): irrigation network for 10,000 donums.
- 4. It is urgently necessary to secure the lands that are required for the project.

APPENDIX 1: SUPPORTED DATA

(Submitted Separately)

- Agricultural Report
- Water Demand for Irrigation
- Existing Hydrologic Model

APPENDIX 2: DESIGN CALCULATION AND ANALYSIS

(Submitted Separately)

- Ground Water Wells
- Hydraulic Model Results
- Storage Tanks

APPENDIX 3: DESIGN DRAWINGS

(Submitted Separately)

APPENDIX 4: SOIL INVESTIGATION

(Submitted Separately)

- **Soil report for irrigation network**
- **Soil report for agricultural reuse**
- **Hyd geological investigation and pumping test report**

APPENDIX 5: DIGITAL MAP

(Submitted Separately)

APPENDIX 6: DESIGN OF AUTOMATION SYSTEM
(Submitted Separately)

APPENDIX 7: CALCULATION SHEETS FOR OPERATION AND MAINTENANCE

(Submitted Separately)