Palestine Polytechnic University

College of Engineering and Technology
Civil & Architecture Engineering Department

Project Title:

Hydraulic Design For a Pumping Station Combined With Conveying Pipeline System Using Computer Models

(Beit Jala case study)

Project Team

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Hebron _ Palestine

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CIVIL &ARCHITECTURAL ENGINEERING DEPARTMENT COLLEGE OF ENGINEERING AND TECHNOLOGY PALESTINE POLYTECHNIC UNIVERSITY

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BY

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A PROJECT REPORT SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

BACHELOR OF ENGINEERING

IN

CIVIL & ARCHITECTURAL ENGINEERING DEPARTMENT

Project Supervisor

Dr-Eng. Itissam Abuiziah

HEBRON- WEST BANK

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CERTIFICATION

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The Senior Project Entitled:

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Prepared By:

Aseel Shalalldah Hadeel Waara

Mohammad Albisher Wesal Ayaydah

In accordance with the recommendations of the project supervisor, and the acceptance of all examining committee members, this project has been submitted to the Department of Civil and Architectural Engineering in the College of Engineering and Technology in partial fulfillment of the requirements of the department for the degree of Bachelor of Science in Engineering.

Project Supervisor **Department Chairman**

الإهداء

الى منارة العلم والامام المصطفى إلى الأمي الذي علم العالمين إلى سيد الخلق إلى رسولنا الكريم سيدنا محمد صلى الله عليه وسلم .

إلى شهدائنا الأبرارالذين جعلوا من أجسادهم جسرا لفكرة الحق العابرة .
إلى ينبوع الأمل الذي لا يمل العطاء، إلى من حاكت سعادتنا بخيوط منسوجة من قلبها إلى أمهاتنا العزيزات.

إلى من سعى وشقي لننعم بالراحة والهناء و لم يبخلوا بشيء من أجل دفعنا في طريق النجاح إلى من علموننا أن نرتقي سلم الحياة بحكمة وصبر ، إلى أبائنا الأعزاء .
الى من كانوا لنا سندا وخير رفيق لنا في درب الحياة ،الى من تحملوا عبء و ضريبة ارتقائنا نحو اجنحة ملائكة الرحمن طلباً للعلم و شاركونا ايامنا بحلاوتها و مضاضتها ، الى احبائنا و شركائنا في الحياة .

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We can find no words to express our sincere, appreciation and gratitude to our parents, sisters and brothers, for their endless support and encouragement, we are deeply indebted to you and we hope that we may someday reciprocate it in some way.

Work Team

ABSTRACT

Hydraulic Design for a Pumping Station Combined With Conveying Pipeline System Using Computer Models

By:

Aseel Shalalldah | Hadeel Waara

Mohammad Albisher | Wesal Ayaydah

Supervisor:

Dr-Eng. Itissam Abuiziah

Beit Jala city like the other Palestinian cities, which are suffering from water scarcity due to conditions of water supply, which is limited, the study aims to design a system for on demand supply for Beit Jala region, which is adequately, satisfy the water requirements for a combination of domestic, commercial, public and firefighting purposes at the right time.

Individual pipelines may contain any of several kinds of pumps at one end; they may deliver water to or from storage tanks, In a water pipeline system, system flow control or active devices (gate valves, Air Valves pressure reducing or pressure sustaining valves…etc.) is an integrated part of its operation, for instance, the opening and closing of valves, and starting and stopping of pumps. When these operations are performed very quickly, they can cause hydraulic transient phenomena. To protect the physical integrity of a pipeline system, there is a need to install surge control devices, such as surge relief valves, surge tanks, or air-vacuum valves, at various points in the system.

To achieve our goals several models will be chosen and used such as WATER CAD and HAMMER CAD.

المقدمة :

مدينة بيت جالا مثل باقي المدن الفلسطينية تعاني من شح المياه بسبب ظروف امدادات المياه المحدودة ، وتهدف الدراسة الى تصميم نظام لدعم العرض والطلب من المياه لهذه المدينة التي تلبي على نحو كاف الاحتياجات المائية لمزيج من الأغراض المنزلية والتجارية والعامة و مكافحة الحرائق في الوقت المناسب .

مجموعة الأنابيب الناقلة تحتوي على أي نوع من الأنواع المختلفة من المضخات في نهاية الخط ، وهذه المضخات توصل المياه من والى صهاريج التخزين ، وفي نظام خط انابيب المياه هناك اجهزة تحكم في التدفق (مخفف ضغط ، او صمامات الحفاظ على الضغط) وهذه الأجزاء لا تتجزأ عن عمل النظام ، على سبيل المثال عمليات فتح واغلاق المحابس ، و بدء و وقف عمل المضخات ، فعندما يتم تنفيذ هذه العمليات بسرعة كبيرة يمكن ان يؤدي ذلك الى حدوث ظواهر هيدروليكية خطيرة ، وللحماية الفيزيائية لنظام خط الأنابيب يكون هناك حاجة لتركيب اجهزة تحكم ، مثل ادوات تخفيف الزيادة في الضغط وصمامات إفراغ الهواء في نقاط مختلفة في النظام .

ولتحقيق أهدافنا سيتم إستخدام العديد من البرامج مثل: GIS , HAMMER CAD , WATER CAD .
CIVIL 3D.

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CHAPTER “1”

“INTRODUCTION”

1.1 General

Water may convey through pipelines by gravity flow or by pumping. The latter system will be significantly more expensive to construct, operate and maintain than similar gravity-flow systems, but in many cases, we are forced to use the pumping stations.

These are some, of many, questions that immediately confront any engineer who is involved in creating the physical infrastructure to satisfy a basic need of mankind: the delivery of water when and where it is wanted. It is the primary objective of these engineers to develop and apply their knowledge to make the system work. How do these systems work? What principles are involved, and how are the systems successfully analyzed and understood? How can the behavior of a preliminary design be evaluated, and how can the design be modified to correct deficiencies?

In our project, we will try to answer these questions by applying several hydraulic principles on the current case study.

1.2 Problem Definition

The water supply for Beit Jala pumped to the water network system by pumping stations directly which cause various problems as energy consuming and in water flow such as water hammer, since the pumping station will not work in its real efficiency and the required duty point at the network level due to the variation of water consumption.

1.3 Objectives of the Project

The main objectives of this work are to:

1) Selecting the suitable Al Mattallah pumping station.
2) Design the convening pipeline system from Al Mattallah pumping station to Beit Jala reservoir.
3) Performing the hydraulic transient system analysis with and without the protection devices.

1.4 Methodology

In carrying out this work, the following research methodologies are adopted:

- Collecting data related to the study area which required to carry out the work.
- Doing the filed survey for the street where the proposed pipeline will pass.
- Estimating the population forecasting for the project period.

- Computing the required water quantity and the flow rate.
- Designing the required pump or pumps.
- Designing the conveying pipe line.
- Using HAMMER model to calculate the hydraulic transients due to pump stop.
- Determination the suitable protecting device to protect the system integrity.

1.5 Phase of the Project

The Project will consist of the four phases as shown in Table (1.1).

1.5.1 First Phase: Data collection and Survey

In this phase, available data and information were collected from different sources. This phase includes the following tasks:-

1) Collecting of aerial and topographical maps for all areas.
2) Doing the field survey for the street where the proposed pipeline will pass.

1.5.2 Second Phase: Preparing Layout for the street where the proposed pipeline will pass and estimating the population and computing the required water quantity and flow rate.

In this phase layout was prepared and put in its final shape and then the population r will estimate and the flow rate will compute. This phase includes the following tasks:

1) Draw the layout of the street where the line will pass and check it more than one time to make sure that is correctly , later compare layout with the real situation in Beit Jala town. Then make adjustment and draw the final layout, this step is the most important one.
2) Determination of the required water quantity and flow rate.

1.5.3 Third Phase: Designing the required pumps and the convening pipeline.

In this phase the necessary calculations needed for the design of main trunks was completed, this phase includes the following tasks:

1) Establish a layout, which includes the street where the proposed pipeline will pass existing elevations.
2) Establish the pipeline in the street layout.
1) Establish the required pumps calculations.

2) Preparing needed different drawings for the designed pipeline.

1.5.4 Fourth Phase: Using HAMMER model to calculate the hydraulic transient due to pump stopping, and determination the suitable protecting device.

After finishing the design steps of the main pipeline and choosing the pump type, the hydraulic analysis for both steady and transient state will be carried out to choose the suitable protection devices depending on the simulation results.

1.5.5 Fifth Phase: Writing the report and preparing maps.

After finishing the main steps and the work, the necessary layout had been made and the final report prepared and submitted to the Department of Civil and Architectural Engineering at Palestine Polytechnic University.

Table 1. 1: Phases of the project with their expected duration

Title	Duration							
	9/16	10/16	11/16	12/16	2/17	3/17	4/17	5/17
Data collection and field survey								
Estimating the population for the project period, computing the required water quantity and flow rate, and preparing layout**.**								
Designing the required pumps and the conveying pipe line								
Using HAMMER model to calculate the hydraulic transient due to pump stop, and determination the suitable protecting device.								
Writing the report and preparing maps								

1.6 Organization of the Project

The study report has been prepared in accordance with the objectives and of work. The report consists of five chapters.

- The first chapter entitled "Introduction" outlines the problem, project objectives, and phases of the project.
- Chapter two entitled " Project Area " which contain general information about Beit Jala town
- Chapter three entitled "General information on drinking water supply" which deals with the reviews literature on hydraulic principles.
- Chapter four entitled "Flow calculation and valve fittings" which contain calculation and methods used to calculation.
- Chapter five entitled "Field Work" which contain field and civil work.
- Chapter six entitled “Using Bentley hammer”
- Chapter seven entitled "Using hammer to analyses the system”
- Chapter eight entitled "Conclusion".

CHAPTER “2”

“PROJECT AREA”

2.1 General

In this chapter, the basic data location, topography, climate, water supply and rainfall quantity of Beit Jala town will be presented and briefly discussed.

2.2 Location and Physical Characteristics

Beit Jala is a Palestinian city in Bethlehem Governorate located at 1.8km (horizontal distance) west of Bethlehem City. Beit Jala is bordered by Bethlehem city to the east, Jerusalem city and Gilo settlement to the north, Al Walaja and Battir villages to the west, and Ad Doha city and Al Khader town to the south (See figure 2.1). Beit Jala is located at an altitude of 779m above sea level with a mean annual rainfall of 563mm. The average annual temperature is 16^{o} C, and the average annual humidity is about 60.5 percent (ARIJ GIS, 2009). And the topography of Beit-Jala as seen in figure 2.2 .

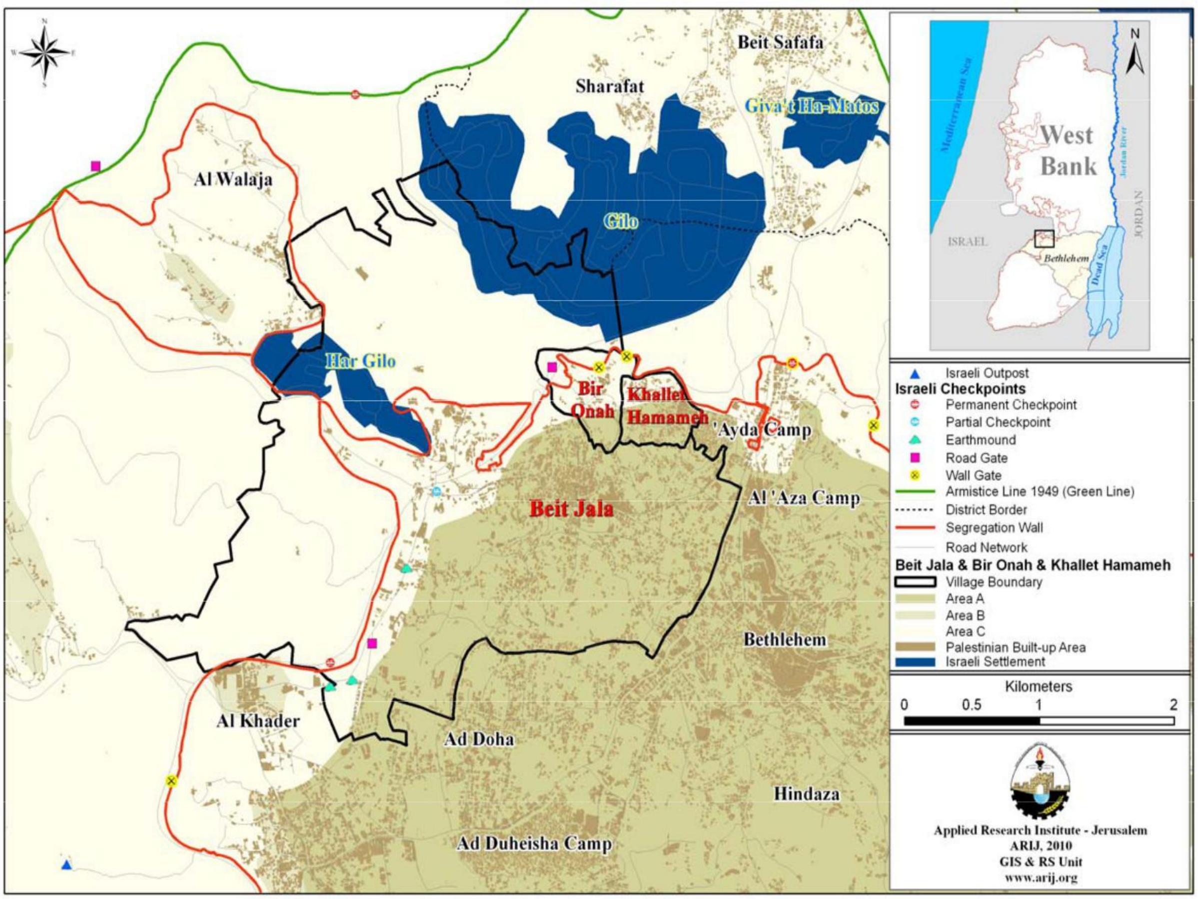


Figure 2. 1: Beit Jala location and borders

2.3 History

The name of Beit Jala is originally an Aramaic name, which means a grass carpet. The city dates back to 1912, and its residents originate from the neighboring Arab countries including Iraq. Beit Jala city includes two other localities: Khallet Hamameh and Bir Onah.

Figure 2. 2: Beit Jala town

2.4 Religious and Archaeological Sites

In terms of religious establishments, there are five churches in Beit Jala: Virgin Mary Greek Orthodox Church, Al Bishara Latin Church, St. Nicolas Greek Orthodox Church.

St. Michael Greek Orthodox Church, and the Evangelical Lutheran Church, in addition to two mosques: Beit Jala Al Kabeer Mosque, and Al Imam Ahmad Ben Hanbal Mosque(See Fig 2.2).

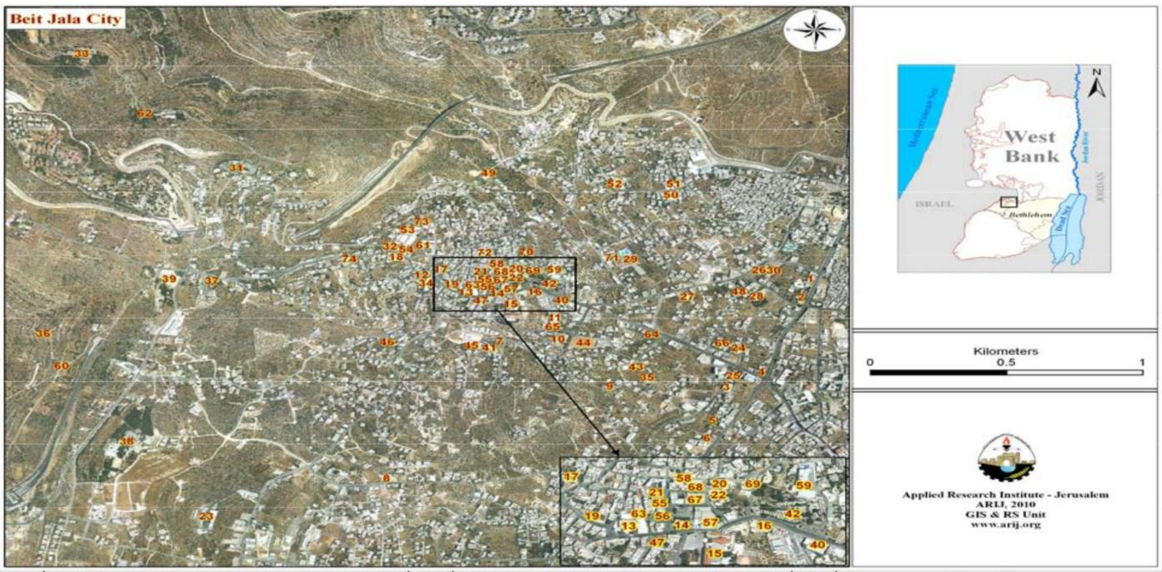
As for the archaeological sites, Beit Jala city is full of significant historical and biblicalsites. Ruins from Roman, Byzantine, Islamic and Crusader times can be found throughoutthe area. In addition, there are many sites of special importance to the followers of thethree monotheistic religions, to whom this land is sacred, including: (Beit Jala City book,1994).

1) Virgin Mary Greek Orthodox Church: located on the main street in the center of Beit Jala city. In term of architecture, the church is considered one of the largest churches in the area. In 1862, the church was constructed as a building at the expense of the Holy Tomb Monastery.

2) St. Michael Greek Orthodox Church: In the past, the church was an old building that included a chapel and two rooms to its western and northern sides, from which were the entrances to its garden.

3) Al Bishara Latin Church: Located at the eastern side of Beit Jala at a place called Iraq Al Jazza. The church was built in 1858 after obtaining a decree from the Turkish government.

4) Evangelical Lutheran Church**:** Located in a prominent place in Beit Jala, on a public street. According to the ancestors, in 1862, the Protestants were given a house in the town, and in 1866, a school was established; it included 25 students, whom later became the nucleus of the Lutheran community in Beit Jala.



No.	Locations	No.	Locations	No.	Locations
1	Bethlehem Bible College	26	Education Directorate	51	UN School
2	Latin Patriachate	27	Olive Press	52	Home for Elderly People
3	Good Shepherds School (Ar Ra'ei as Salih)	28	Al 'Ala'iya School	53	Wadi' Di'mes Boys Elementary School
4	UN Relief Committee Office	29	Olive Tree Tourist Village	54	Cheile Girls High School
5	Baraka B Presbyterian Church	30	Bethlehem Boys High School	55	Police Station
6	Dibs Hospital	31	Arab Society for Rehabilitation Hospital	56	Beit Jala Medical Complex
7	Children of Peace Playground - and Public Park (Duck Garden)	32	Arab Orthodox Sports Club	57	Latin Patriarchate Elementary School
8	Ad Doha Reservoir	33	Cremisan Monastery	58	Health Center (Al Mustawsaf)
9	St. Afram Preparatory School	34	Evangelical Lutheran House	59	Al Quds Open University
10	Iskandar Al Khoury Boys High School	35	Beit an Noor Society for Blind Girls	60	Kebriyanos Spring
11	Latin Patriarchate High School (Fransees Building)	36	Al Makhroor Area	61	Senior Citizens Society (Dar ash Shuyookh)
12	Beit Ibrahim Church (Lutheran Church)	37	DCO/ Beit Jala	62	Virgin Mary Spring (Al 'Adra Spring)
13	Virgin Mary Greek Orthodox Church	38	Talitha Kumi High School & the Headquarters of Talitha Kumi Scouts	63	Post Office
14	Beit Jala Muncipality	39	Hope High School (Al-Amal School)	64	Vocational Training Center
15	Al Bishara Latin Patriarchate Church & Theology Institute (Seminar) & Latin Scouts	40	Life Gate Association for Rehabilitation	65	Talitha Kumi Community College for the Education of Hospitality
16	Latin Cemetry	41	Beit Jala Public Library	66	Sira School for Special Education
17	Beit Jala Mosque	42	Greek Orthodox Cemetry & St. Michael Church	67	Orthodox Charity Society & the Orthodox Scouts' Headquarters
18	Al Harra Theater	43	Al Quds American School	68	Women's Society for Child Care
19	Enad Theater	44	Municipal Playground	69	Bethlehem Music Academy
20	Saint Nicolas Greek Orthodox Church	45	Beit Al Liq'a Center	70	Iskandar Al Khoury Cultural Center & Palestine Youth Union & the Palestinian Child Club
21	Beit Jala Market	46	Yamima Association for Mentally Handicapped	71	The Civil Society for the Visually Impaired & Ash Shurooq School for the Blind
22	Orthodox Center for Education and Culture	47	Greek Orthodox Monastrey Kindergarten	72	Beit Jala/Old City
23	Industrial Zone	48	Beit Jala Company for Pharmaceutical Manufacturing	73	Abu Zgheibra Palace
24	Baptist Church (Al Ma'madaniya)	49	Bir Onah Well	74	Dar Shahwan & an Ancient Stone Olive Press
25	Beit Jala Governmental Hospital (Al Hussein)	50	Ahmad Ben Hanbal Mosque & Kindergarten		

Figure 2. 3: Main locations in Beit Jala city (Palestinian Localities Study, 2010)

2.5 Population

According to the Palestinian Central Bureau of Statistics (PCBS), the total population of Beit Jala in 2007 was 13,845; of whom 6,859 are males and 6,986 are females. There are 3,093 households living in 3,917 housing units.

The General Census of Population and Housing carried out by PCBS in 2007 showed that the distribution of age groups in Beit Jala is as follows: 31 percent are less than 15 years, 59.3 percent are between 15 - 64 years, 6.1 percent are 65 years and older, and 3.6 percent are unknown. Data also showed that the sex ratio of males to females in the city is 98.2:100, meaning that males constitute 49.5 percent of the population, and females constitute 50.5 percent of the population.

2.6 Agricultural Sector

Beit Jala lies on a total area of about 9,749 dunums of which 7,305 dunums are considered arable land, and 913 dunums are residential land (See table 5 and fig 2.3).

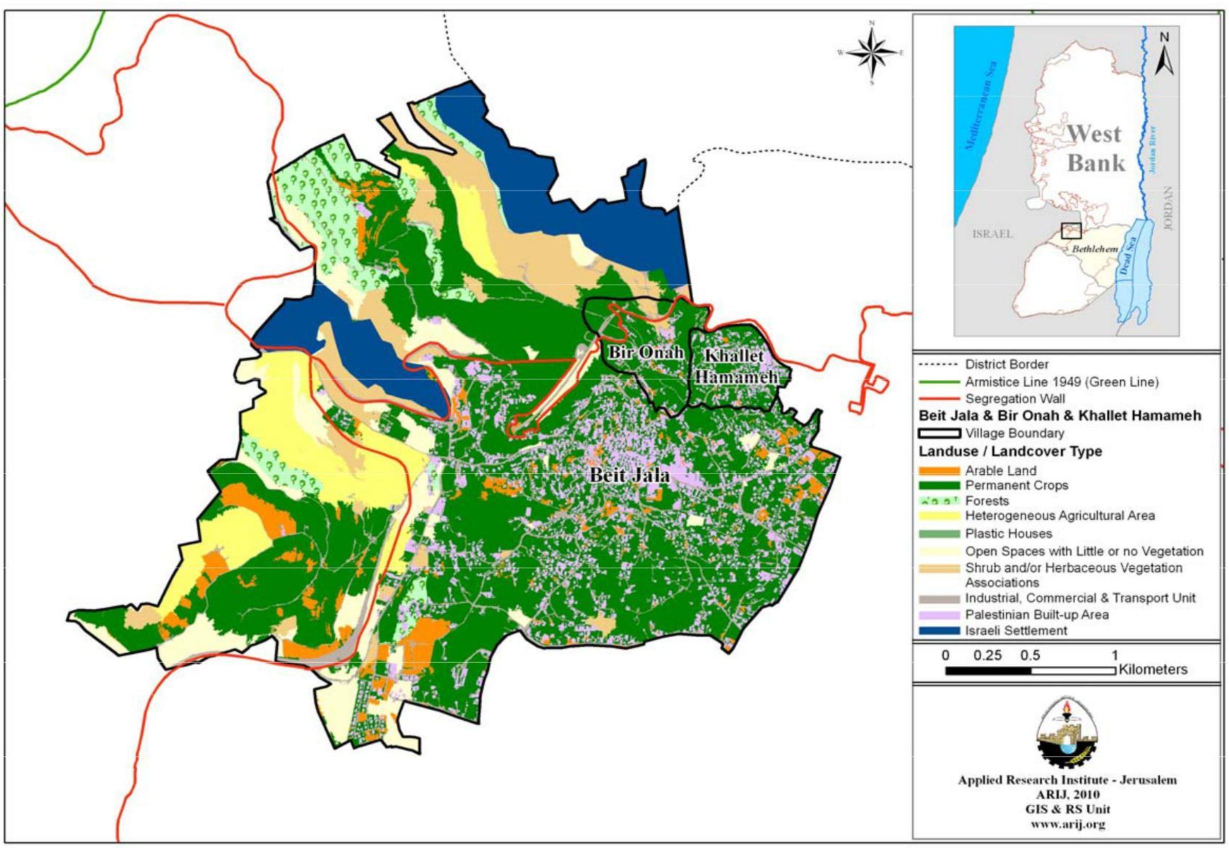


Figure 2. 4: Land use/land cover and segregation wall in Beit Jala city

Table 2. 1: Land use in Beit Jala city in dunum (GIS unit- ARU,2008)

Total Area	Built up Area	Arable Land (7,305)					Area of Industrial, Commercial & Transport Unit	Area of Settlements and Military Bases
		Seasonal Crops	Permanent Crops	Greenhouses	Forests	Open Spaces and Rangelands		
9,749	913	1,136	4,240	1	462	1,466	663	868

Agriculture production in Beit Jala depends mostly on rainwater. As for irrigated fields, they depend on domestic harvesting cisterns.

Table 2.2 shows the different types of rain-fed and irrigated open-cultivated vegetables in Beit Jala. The most common crop cultivated within this area is tomato; there are 0.5 dunums of land on which there are greenhouses planted with tomato.

Table 2. 2: Total area of rain-fed and irrigated open cultivated vegetables in Beit Jala city in dunum (Palestinian Ministry of Agriculture, 2007)

Fruity vegetables		Leafy vegetable		Green legumes		Bulbs		Other vegetables		Total area	
RF	Irr.	RF	Irr	RF	Irr.	RF	Irr.	RF	Irr.	RF	Irr.
21	21.5	0	6.5	0	1.5	0	0.5	0	6	21	36

Rf : Rain-fed, Irr : Irrigated

Table 2. 3: Total area of fruit and olive trees in Beit Jala City (dunum)

Olives		**Citrus**		**Stone-Fruits**		**Pome fruits**		**Nuts**		**Other fruits**		**Total area**	
Rf	Irr.	Rf	Irr.	Rf	Irr.	Rf	Irr.	Rf	Irr.	Rf	Irr.	Rf	Irr.
3,020	0	0	0	605	0	221	0	0	0	261	0	4,107	0

As for the field crops and forage in Beit Jala, cereals, in particular, wheat and barley are the most cultivated covering an area of about 45 dunums, while forage crops, such as bitter vetch and common vetch are the second most cultivated crops (See table 2.4).

Table 2. 4: Total area of field crops in Beit Jala city (dunum)

Cereals		Bulbs		Dry legumes		Oil Crops		Forage crops		Stimulating Crops		Other crops		Total area	
Rf	Irr	Rf	Irr	Rf	Irr	Rf	Irr	Rf	Irr	Rf	Irr	Rf	Irr	Rf	Irr
45	0	0	0	0	0	0	0	24	0	0	0	0	0	89	0

The field survey shows that most of the residents in Beit Jala are rearing and keeping domestic animals such as sheep, cows, goats, broiler and layer chicken, and bees (See Table 2.5).

Table 2. 5: Livestock in Beit Jala city

Cows*	**Sheep**	**Goats**	**Camels**	**Horses**	**Donkeys**	**Mules**	**Broilers**	**Layers**	**Bee Hives**
16	251	320	0	10	6	7	8,900	3,000	30

2.7 Infrastructure and Natural Resources

2.7.1 Electricity and Telecommunication Services:

Beit Jala is connected to a public electricity network; served by Jerusalem Electricity Company, which is the main source of electricity in the city. Approximately 98.9 percent of the housing units in the city are connected to the network, and 0.2 percent are dependent on private generators, while the source of electricity is unknown for the remaining units (1.1%) (Central Bureau of Statistics, 2007).

Furthermore, Beit Jala is connected to a telecommunication network and approximately 99 percent of the housing units within the city boundaries are connected to phone lines.

2.7.2 Transportation Services:

Taxis are considered the main means of transportation in Beit Jala, as there are 50 taxis in the city. As for the road network in the city; there are a total of 60km of paved roads, however, a part of these roads are in need of rehabilitation (Beit Jala Municipality, 2010).

2.7.3 Water Resources:

Beit Jala is provided with water by the Palestinian Water Authority (PWA), through the public water network. About 97.7 percent of the housing units are connected to the water network, 0.7 percent are dependent on rainwater harvesting cisterns, 0.2 percent are dependent on purchase of water tanks, and 0.3 percent are dependent on other water resources, while the source of water supply is unknown for the remaining units (1.2%) (Central Bureau of Statistics, 2007). Based on the PWA estimations, the rate of water supply per capita in the communities provided with water is about 100 liters per day, but this rate varies from one community to another. The quantity of water supplied to Beit Jala in 2006 was about 540,000 cubic meters/year; therefore, the estimated rate of water supply per capita is about 89 liters/day (PWA, 2006).

Here it should be noted that many Beit Jala citizens do not in fact consume this amount of water due to water losses, which are about 31 percent. The losses usually happen at the main source, major transport lines, distribution network, and at the household level (PWA, 2006), thus the rate of water consumption per capita in Beit Jala is 61 liters per day. This is a low rate compared with the minimum quantity proposed by the World Health Organization, which is 100 liters per capita per day.

2.7.4 Sanitation:

Beit Jala city has a 32.5km public sewage network, established between 1995 and 1999. The end of the network is connected to Bir Onah Pumping Station, which pumps the waste water into West Jerusalem private sewage network. According to the results of Community Survey conducted by the PCBS in 2007 and the data provided from PWA, the majority of Beit Jala housing units (74%) use the sewage network as a major means for wastewater disposal, 23.7 percent use cesspits, and 0.2 percent lack waste water collection and disposal service, while the means for waste water disposal in unknown for the remaining units (2.1%) (Central Bureau of Statistics, 2007).

Based on the estimated daily per capita water consumption, the estimated amount of wastewater generated per day, is approximately 574 cubic meters, or 209 thousand cubic meters annually.

At the individual level in the city, it is estimated that the per capita wastewater generation is approximately 49 liters per day. The estimated quantity of wastewater collected through the sewage network per day is about 425 cubic meters per day, or 155,000 cubic meters annually. Also, 50 thousand cubic meters of wastewater are collected annually by cesspits and are discharged by wastewater tankers. Here it should be noted that there is no wastewater treatment either at the source or at the disposal sites, which poses a threat to the environment and the public health.

2.8 Locality Development Priorities and Needs

Beit Jala suffers from a significant shortage of infrastructure and services. Table 15 shows the development priorities and needs in the city, according to the municipality's point of view (Beit Jala Municipality, 2010):

Table 2. 6: Development Priorities and Infrastructural Needs in Beit Jala (Beit Jala Municipality, 2010)

No	Sector	Strongly Needed	Needed	Not apriority
1	Opening and pavement of roads	*		
2	Rehabilitation of Old Water Networks	*		
3	Extending the Water Network to Cover New Built up Areas		*	
4	Construction of New Water Networks			*
5	Rehabilitation/ Construction of New Wells or Springs	*		
6	Construction of Water Reservoirs	*		
7	Construction of a Sewage Disposal Network		*	
8	Construction of a New Electricity Network			*
9	Providing Containers for Solid Waste Collection	*		
10	Providing Vehicles for Collecting Solid Waste	*		
11	Providing a Sanitary Landfill	*		

CHAPTER “3”

“GENERAL INFORMATION ON DRINKING WATER SUPPLY”

3.1 Assessment of Drinking Water Demand

The most important data to be determined at the study level for determination of the drinking water requirements, and therefore the determination of the flow and sizing the structures, which are as follows:

- Population.
- Unit allocations.
- Performance.
- Peak coefficients

3.1.1 Type of water demand or consumption

Generally, several types of water demand In Palestine could be found, whether at the level of an urban or rural agglomeration, depending on the type of consumer

- Domestic consumption.
- Public or collective consumption (municipality, administrations, schools, watering Gardens, hospitals, shops, etc.).
- Industrial consumption.
- Tourism consumption.

These types of consumption differ in their quantities and especially their necessary qualities.

3.1.2 Estimating drinking water requirements

Rigorous quantification of these requests is generally based on a statistics, which came from the latest censuses.

Average domestic consumption is generally related to the number of inhabitants, it is then expressed in liters per day per capita (L/day/inh). This consumption varies depending on several factors: standard of living habits, availability of water, climate, the price of water, the form of the supply of water (individual feed or terminal fountain), etc. On the other hand, it evolves from one year to another inequality of life.

The domestic water requirements of any agglomeration can be estimated by:

- Statistics, concerning average consumption and its evolution of annual population, as well as the total number of inhabitants and the annual population growth rate.

- In comparison with other agglomerations that are considered to be comparable, especially in terms of living standards and climate and for which data statistics are available.

In this regard, the World Health Organization (WMO) fix the standard domestic consumption to 55 L/day.

1. For fountain terminals: 11 L/day.
2. For the rural areas: 50 L/day.
3. For medium cities: 80 L/day.
4. For large cities: 140 L/day.

For other needs as well as public needs, they depend on the type of establishment, as an indication:

1. Hospitals: from 300 to 600 L/day.
2. For street cleaning and watering gardens: from 3 to 5 L/day.
3. For administrations: from 100 to 200 L/day.
4. For schools: 20 to 30 L/day.

3.1.3 Peak coefficients

3.1.3.1 Average consumption

The domestic consumption values shown above are sometimes increased to take account of public consumption and small industries.

The number of future inhabitants (the years of the project life cycle) in an urban agglomeration, No, is determined by:

$$N_o = N(1+a)^n \tag{3.1}$$

Where:

N is the number of inhabitants in any year.

a is the annual growth rate of the population.

n is the number of years between *N* and *No*.

If the agglomeration well managed and planed, then ***No*** will be calculated based on the planned urbanization plan. The average future consumption ***Co*** per capita is given by:

$$C_o = C(1+b)^n \tag{3.2}$$

Where:

C is the average consumption per capita in any given year.

b is the annual rate of change in consumption.

n is the number of years between the year of ***C*** and that of ***Co***.

The average daily consumption (Q ad), during the project year, of any agglomeration will then be calculated by:

$$Q_{ad} = N_o \times C_o \tag{3.3}$$

In some large cities, domestic consumption varies from one neighborhood to another depending on the type of dwelling, density, standard of living, and so on. This must be taken into account, and take variable consumptions:

$$Q_{ad} = \sum N_{oi} \times C_{oi} \tag{3.4}$$

Where:

Noi and ***Coi*** are successively the number of inhabitants and the average daily consumption per inhabitant in the district number "***i***".

3.1.3.2 Maximum demand on daily basis

Water consumption varies depending on the month, the week of the month, the day of the week and the hour of day. Therefore, designing the water supply network and its integrity (pumping stations, pipes, etc.) must be dimensioned in order to carry the maximum daily demand (peak daily demand) of the project year. A daily peak coefficient ***K1*** then could be calculated by the relation:

$$K_1 = \frac{\text{Maximum consumption on daily basis}}{\text{Average daily consumption}} = \frac{Q_{md}}{Q_{ad}} \tag{3.5}$$

The value of this coefficient ***K1*** determined on the statistics basis of the daily variation in the consumption over the 365 days of the year. Generally, this value of ***K1*** Varies from 1.3 to 1.6, depending on the climate and summer activities of the agglomeration (for example, for a touristic areas, ***K1*** is close to 1.6).

3.1.3.3 Peak Hourly

Water networks (network, tanks) must be sized to provide maximum hourly demand (peak hour) for the peak daily of the project years. We also define an hourly peak coefficient ***K2.***

$$K_2 = \frac{\text{Maximum hourly consumption}}{\text{Average hourly consumption}} = \frac{Q_{mh}}{Q_{ah}} \tag{3.6}$$

Similarly, the value of the ***K2*** coefficient is determined on the statistics basis of the hourly variation in consumption. Its value varies from 1.5 to 3.5 depending on the importance of the agglomeration.

3.1.3.4 Water loss

Throughout the world, water losses are occurring at both the end-users plumbing and the water supplier's distribution piping. Water losses are a universal problem and they do occur in both developed and developing countries.

In a drinking water supply system, water losses are located at different levels: intake, treatment plant, pumping stations, reservoirs, conveyance and distribution networks, Valves, seals, counters, etc.

These losses are also of different types: washing and cleaning water (filters and decanters from the treatment plant, reservoirs), leaks in all works and in particular in supply and distribution networks, Accidental losses in case of pipe breaks, emptying of pipes (in case of works, replacement of pipes or valves, connections, etc.).

The volume of these water losses depends on:

- ➢ The age and condition of the network.
- ➢ The competence and efficiency of the network maintenance service (speed of detection of leaks, efficiency of work execution, human resources, Materials, organization, etc.).Simply stated, the problems of water and revenue losses are:

1)Technical: Not all water supplied by a water utility reaches the customer.

2)Financial: Not all of the water that reaches the end user is properly measured or paid-for.

3)Terminology: Standardized definitions of water and revenue losses are essential to quantify and control the losses.

In general, the **K3** value ranges from 1.2 to 1.5:

- ✓ K3 = 1.2 for a new or well-maintained network.
- ✓ K3 = 1.25 to 1.35: for a moderately maintained network, and
- ✓ K3 = 1.5 for a poorly maintained network.

3.1.3.5 Calculation of the flow for various structures in the network:

The calculation rate depends on the type and location of the work to be calculated or dimensioned. The sizing and / or calculation flow of the intake structures (pumping station, treatment plant, reservoirs, supply lines, etc.) is equal to the maximum daily flow *Qdm.*

$$Q_{dm} = K_3 \times K_1 \times Q_{ad} \quad (3.7)$$

3.2 Types of Conveying Drinking Water Supply Systems.

Methods for efficient water distribution system with required adequate water pressure at various points, depends upon the source level, topography of the area and other local conditions, the water may be supplied by the following methods:

1) Gravity System.

2) Pumping System.

3) Combined gravity and pumping system.

3.2.1 Gravitational system

This system is the most dependable technique provided. There are multiple well-protected conduits carrying the flow to the community.

The gravity system is the most reliable. This system is useful in hilly areas where the source of supply is located substantially above the level of the city, such that adequate pressure is obtained in the network directly.

Table 3. 1: Advantages and disadvantages of Gravity System (Afifi 2009)

	Advantages	Disadvantages
1	No energy costs.	Gravity system is less flexible for future extension.
2	Simple operation.	Small gradients available for friction losses require large diameters within the whole system.
3	Low maintenance costs.	Longer pipelines are necessary.
4	No sudden pressure to change.	High pressure for firefighting requires use of motor pumping

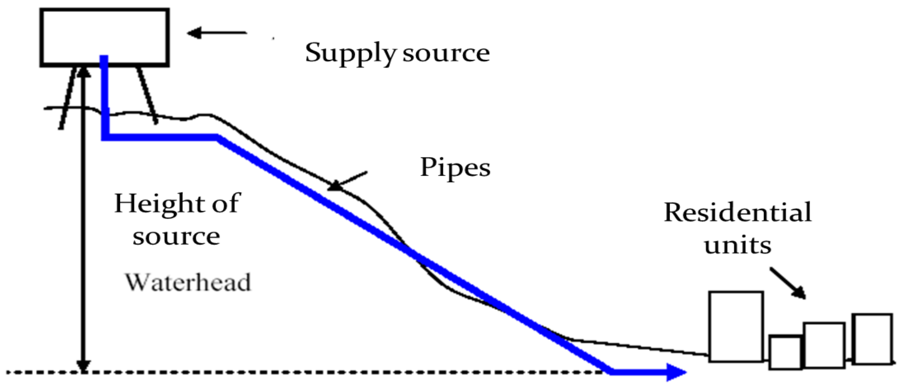


Figure 3. 1: Distribution by Gravity (Afifi 2009)

3.2.2 Pressurized System

Pressurized system means pumping, and there is two types of pumping, Pumping without storage system and pumping with storage system.

- Pumping without storage system: Pumped supply systems operate without or with limited water storage (water towers) in the distribution system. With direct pumping to the system, they have to follow variations in water demand. Proper selections of units have to be done in order to optimize energy consumption, including reserve pumping for irregular situations.

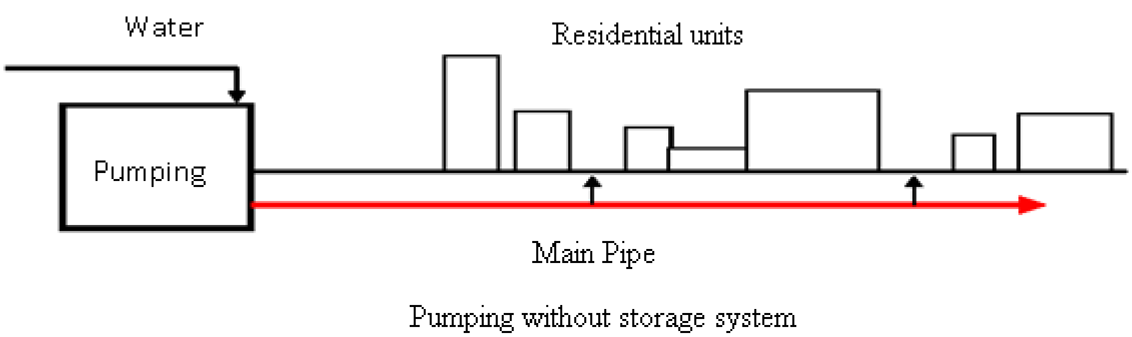


Figure 3. 2: Pressurized System without storage (Afifi 2009)

- Pumping with storage: It is the most common method of distribution. Water is pumped at a more or less uniform rate with flow in excess of consumption being stored in elevated storage tanks distributed throughout the system. During periods of high demand, the stored water augments the pumped flow, thus helping to equalize the pumping rate and maintain more uniform pressure in the system.

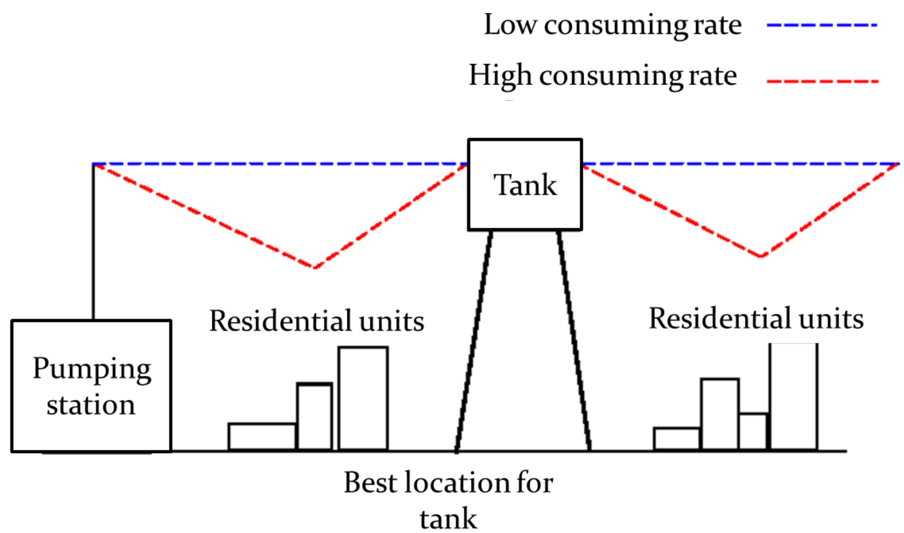


Figure 3. 3: Pressurized System with storage system (Afifi2009)

Advantages of pumping with storage:

1. Economical to pump only during off-peak hour to minimize powering costs.
2. Pumping with storage system is the most common system for large distribution areas

Table 3. 2: Advantages and disadvantages of pressurized system (Afifi2009)

Advantages	Disadvantages
Permitting increased pressure for firefighting.	Power failure and pressure will fluctuate substantially with variation in flow.
	The flow must be constantly varied to match an unpredictable demand.
	Increasing power costs.

3.3 Hydraulic Head

Hydraulic head or piezometric head is a specific measurement of liquid pressure above a geodetic datum. It is usually measured as a liquid surface elevation, expressed in units of length, at the entrance (or bottom) of a piezometer. In an aquifer, it can be calculated from the depth to water in a piezometric well (a specialized water well), and given information of the piezometer's elevation and screen depth.

Hydraulic head can similarly be measured in a column of water using a standpipe piezometer by measuring the height of the water surface in the tube relative to a common datum. The hydraulic head can be used to determine a hydraulic gradient between two or more points.

In fluid dynamics, head is a concept that relates the energy in an incompressible fluid to the height of an equivalent static column of that fluid. From Bernoulli's Principle, the total energy at a given point in a fluid is the energy associated with the movement of the fluid, plus energy from static pressure in the fluid, plus energy from the height of the fluid relative to an arbitrary datum. Head is expressed in units of height such as meters or feet.

The static head of a pump is the maximum height (pressure) it can deliver. The capability of the pump at a certain RPM can be read from its Q-H curve (flow vs. height).

A common misconception is that the head equals the fluid's energy per unit weight, while, in fact, the term with pressure does not represent any type of energy (in the Bernoulli equation for an incompressible fluid this term represents work of pressure forces). Head is useful in specifying centrifugal pumps because their pumping characteristics tend to be independent of the fluid's density.

There are four types of head used to calculate the total head in and out of a pump:

1) Velocity head is due to the bulk motion of a fluid (kinetic energy). Its pressure head correspondent is the dynamic pressure.

2) Elevation head is due to the fluid's weight, the gravitational force acting on a column of fluid.

3) Pressure head is due to the static pressure, the internal molecular motion of a fluid that exerts a force on its container.

4) Resistance head (or friction head or Head Loss) is due to the frictional forces acting against a fluid's motion by the container.

3.3.1 Components of hydraulic head.

A mass free falling from an elevation $z > 0$ (in a vacuum) will reach a speed $v = \sqrt{2g\,z}$, when arriving at elevation z=0, or when we rearrange it as a *head*:

$$h = \frac{V^2}{2g} \tag{3.8}$$

Where:

g : is the acceleration due to gravity

The term $\frac{V^2}{2g}$ is called the velocity head, expressed as a length measurement. In a flowing fluid, it represents the energy of the fluid due to its bulk motion.

The total hydraulic head of a fluid is composed of pressure head and elevation head. The pressure head is the equivalent gauge pressure of a column of water at the base of the piezometer, and the elevation head is the relative potential energy in terms of an elevation. The head equation, a simplified form of the Bernoulli Principle for incompressible fluids, can be expressed as:

$$\text{HGL} = \frac{P}{\gamma} + h \tag{3.9}$$

$$\text{EGL} = \frac{P}{\gamma} + \frac{V^2}{g} + h \tag{3.10}$$

Where:

$\frac{P}{\gamma}$: Pressure head h: elevation head $\frac{V^2}{g}$: Velocity head

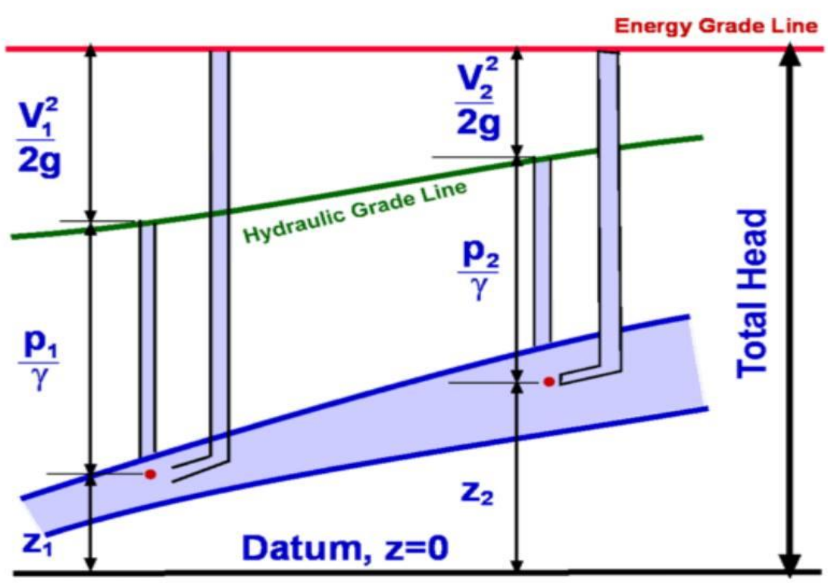


Figure 3. 4: Energy Grade Line (EGL) and Hydraulic Grade Line (HGL)

If the losses are taken into account, the EGL will drop accordingly. Any work extraction along the path as with a turbine, will be seen as a sudden drop in the EGL. Any work addition will be reflected as a sharp rise. HGL follows similar trends.

The Hydraulic Grade Line is a graph of the pressure and gravitational heads plotted along the position of the pipeline or channel.

Energy grade line – Represents the total available energy in the system (potential energy plus kinetic energy). Hydraulic grade line - The hydraulic grade line (HGL), a measure of flow energy, is a line coinciding with the level of flowing water at any point along an open channel. In closed conduits flowing under pressure, the hydraulic grade line is the level to which water would rise in a vertical tube (open to atmospheric pressure) at any point along the pipe. HGL is determined by subtracting the velocity head (V2 /2g) from the energy gradient (or energy grade line). Figure 1 illustrates the energy and hydraulic grade lines for open channel and pressure flow in pipes. As illustrated in Figure 1, if the HGL is above the inside top (crown) of the pipe, pressure flow conditions exist. Conversely, if the HGL is below the crown of the pipe, open channel flow conditions exist.

The relationships between the hydraulic and energy grade lines are as such:

1) If the pipe is not under pressure, the hydraulic grade line is simply the gravitational head.
2) If the pipe is flowing full, the hydraulic grade line will always be below the energy grade line.
3) In a reservoir (i.e. non-moving flow not under pressure), the hydraulic grade line will be equal to the energy grade line.
4) A decrease in flow area will result in a decrease in the hydraulic grade line, and vice versa.

3.3.2 Head losses

When water (or any other fluid) flows through a pipe, the pressure continuously drops in the stream wise direction because of friction along the walls of the pipe. It is common to express this pressure drop in terms of an irreversible head loss. Head is defined as the vertical height of a column of fluid connected to the flow by a static pressure tap. A fluid column that is at rest has a hydrostatic pressure difference from top to bottom of the column given by $\Delta P = \rho g|\Delta z| = \rho gh$, where h = |Δz| is the difference in "head" or elevation of the column from top to bottom. In flow through piping systems, there are two types of irreversible head losses. Major losses, denoted by h major (sometimes hf), are those due to frictional losses through sections of constant diameter pipe, in which the flow is fully developed.

3.3.2.1 Linear head loses

1) Physical equation

Darcy Weisbach formula (linear):

$$\Delta h = \frac{\lambda \, L \, V^2}{2 \, g \, D} \quad (3.11)$$

Laminar flow: Re < 2000

$$\lambda = \frac{64}{Re} \quad (3.12)$$

And:

$$Re = \frac{VD}{v} \quad (3.13)$$

To find the friction factor **λ** for different pipes roughness in turbulent flow, Colebrook and white equation had been used

Colebrook and white equation:

- Turbulent flow

➢ Hydraulically smooth pipe

$\frac{kV}{v}$< 5 and 4000 < Re < 10^5

$$\frac{1}{\sqrt{\lambda}} = -2 \log 10 \left(\frac{2.51}{Re \sqrt{\lambda}} \right) \quad (3.14)$$

Or

$$\lambda = \frac{0.316}{Re^{0.25}} \quad (3.15)$$

➢ Semi rough pipe

5 <$\frac{kV}{v}$< 75 and Re> 4000

$$\frac{1}{\sqrt{\lambda}} = -2 \log 10 \left(\frac{k}{3.71 \, D} + \frac{2.51}{Re \sqrt{\lambda}} \right) \quad (3.16)$$

- Rough pipe

$\frac{kV}{v}$> 75 and Re > 4000

2) Empirical formula

a) Hazen Williams Formula:

Where:

Δh = head loss (m)

λ = friction coefficient

L = Length of a pipe section or duct (m)

V = mean velocity in a pipe (m /s)

G = gravitational constant (9.81 m/s²)

D = diameter of a pipe or device

Re = the dimensionless Reynolds number

v = kinematic viscosity of the fluid (for water at 20 °C , v = $1.0 * 10^{-6} m^2/s$)

k = non dimensional loss coefficient (1.0)

Chw = Hazen-Williams coefficients

A = area of a pipe section

R = hydraulic radius

Sf = friction slope

Q = flow of the fluid

n = manning's roughness coefficient

Table 3. 3: Values of Hazen-Williams coefficients and Manning for some materials.

Pipe material	C_{HW}	n
PVC	150	0,008
pig iron or welded steel	140	0,011
Wood, Concrete	130	0,014
Clay, New steel riveted	120	0,016
Font old brick	110	0,017
Corroded cast iron or steel	100	0,020

3.3.2.1 Singular head loses

The resistance is heavily dependent on the path of the fluid as it travels into, though, and out from the valve or any other fittings. With a more constricted path will cause more energy losses. Therefore, select the valve type with care if you desire the system you are designing to be efficient with relatively low energy losses. (This will presented in detailed in chapter four).

3.4 Characteristic of the Pressurized Conduit

3.4.1 HDPE Pipes (High Density Polyethylene):

They can be join by welding or can be joint with detachable joints up to 630 mm Diameter. These are commonly used for conveyance of industrial water. They offer all the advantages and disadvantages in table 3.4.

Table 3. 4 : Advantages and disadvantages of HDPE pipes

Advantages	Disadvantages
1) Good toughness.	1) Need electrical welding
2) Good fatigue strength	2) Need mechanical connections
3) Good temperature resistance	3) Large fittings
4) Lightweight.	
5) Good flexibility and good impact resistance.	

3.4.2 Stainless steel pipe:

Stainless pipes are designed for outdoor installation for laying pipelines and internal plumbing. Stainless steel pipe is used in heating, hot and cold domestic water, because stainless steel does not have a negative impact on water quality. This type of material has a quite big cost.

Table 3. 5 : Advantages and Disadvatages of Stainless steel pipe

Advantages	Disadvantages
1) Low corrosion	1) Difference of heat transfer
2) Appealing appearance	2) Often more expensive
3) Smaller pipes	3) Very difficult to fabricate
4) Strong	
5) Recyclable	
6) Durable	

3.4.3 Steel pipe:

Iron tubes are used in water networks since they bear high water pressure and high temperatures, which make them a good choice to convey hot water in houses. However, iron tubes have a disadvantages, they rust easily and quickly, and so, they should be treated and painted before using them.

Table 3. 6 : Advatages and Disadvatagea of steel pipes

Advantages	Disadvantages
1) Readily available at many hardware stores.	1) Heavy.
2) Rated for pressure.	2) Length of pipe cannot be change
3) Threaded fittings	
4) Durable	
5) Cheap.	
6) It is also available in larger diameters	

3.4.4 Fiber glass pipe:

This martial is widely used where corrosion resistant pipes are required. GRP or FRP can be used as a lining material for conventional pipes to protect from internal or external corrosion. It is made from the composite matrix of glass fiber, polyester resin and fillers. These pipes have better strength, durability, high tensile strength, low density and high corrosion resistance. These are manufacturing up to 2.4 m diameter and up to 18 m length.

Table 3. 7 : Advantages and disadvantages of Fiber glass pipe

Advantages	Disadvantages
1) Light weight Structure	1) Not withstand high pressure
2) Longevity	2) Very expensive
3) Resilience	
4) Efficiency	
5) Versatility	

3.4.5 UPVC pipe (unplasticized polyvinyl chloride):

Plastic is recent material used for sewer pipes. These are used for internal drainage works in house. These are available in sizes 75 to 315 mm external diameter and used in drainage works. They offer smooth internal surface. The additional advantages they offer are resistant to corrosion, light weight of pipe, economical in laying, jointing and maintenance, the pipe is tough and rigid, and ease in fabrication and transport of these pipes.

Table 3. 8: Advantages and disadvantages of UPVC pipe

Advantages	Disadvantages
1) Strong corrosion resistance	1) UPVC monomer and additives may out of pipe.
2) Ease of bonding	2) Not suitable for hot water transport
3) Cheap	3) Require high bonding technology for joints and longer c ring time
4) Hard texture	

Choosing the pipe type:

Choosing the material that should be used in the water network tubes, depending on the water pressure and the tube's radius. The following figure is explaining how the tube's material can be chosen.

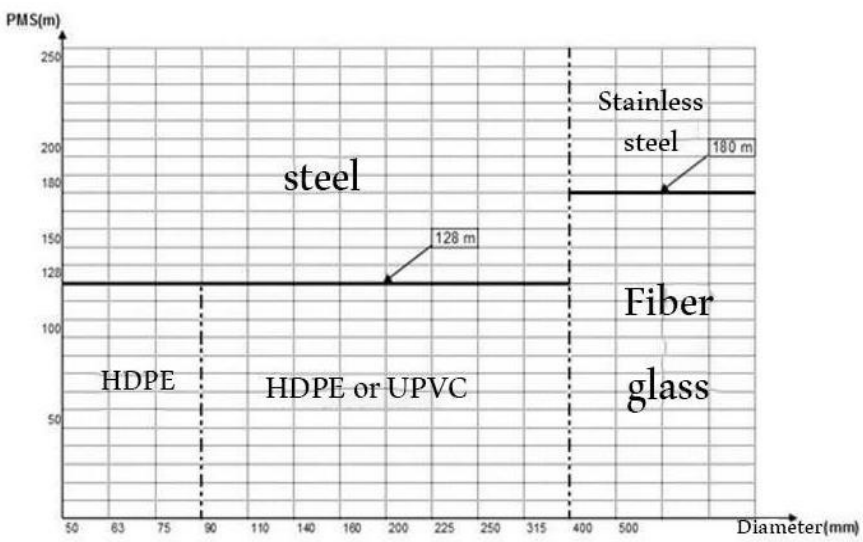


Figure 3. 5 : How the tube's material can be chosen

CHAPTER "4"

"FLOW CALCULATION AND VALVE FITTINGS"

4.1 Population Forecasting

Design of water supply and sanitation scheme is based on the projected population of a particular city, estimated for the design period. Any underestimated value will make system inadequate for the purpose intended; similarly, overestimated value will make it costly. Change in the population of the city over the years occurs, and the system should be designed taking into account of the population at the end of the design period.

Factors affecting changes in population are:

- Increase due to births
- Decrease due to deaths
- Increase/ decrease due to migration
- Increase due to annexation.

The present and past population record for the city can be obtained from the census population records. After collecting these population figures, the population at the end of design period is predicted using various methods as suitable for that city considering the growth pattern followed by the city.

4.1.1 Arithmetical increase method

This method is suitable for large and old city with considerable development. If it is used for small, average or comparatively new cities, it will give low result than actual value. In this method the average increase in population per decade is calculated from the past census reports. This increase is added to the present population to find out the population of the next decade. Thus, it is assumed that the population is increasing at constant rate.

Hence, $dP/dt = C$ i.e. rate of change of population with respect to time is constant.

Therefore, Population after n^{th} decade will be $P_n = P + n.C$

Where, P_n is the population after n decade and P is present population.

4.1.2 Geometrical increase method (or geometrical progression method)

In this method the percentage increase in population from decade to decade is assumed to remain constant. Geometric mean increase is used to find out the future increment in population. Since this method gives higher values and hence should be applied for a new industrial town at the beginning of development for only few decades. The population at the end of nth decade 'Pn' can be estimated as:

$P_n = P\,(1+ I_G/100)^{n}$

Where, I_G = geometric mean

(%) P = Present population

N = no. of decades.

Example:

Considering data given in example 1 predict the population for the year 2021, 2031, and 2041 using geometrical progression method.

Solution:

Table 4. 1: Population counting for geometrical increase method.

Year	Population	Increment	Geometrical increase Rate of growth
1961	858545	-	-
1971	1015672	157127	(157127/858545) = 0.18
1981	1201553	185881	(185881/1015672) = 0.18
1991	1691538	489985	(489985/1201553) = 0.40
2001	2077820	386282	(386282/1691538) = 0.23
2011	2585862	508042	(508042/2077820) = 0.24

Geometric mean $I_G = (0.18 \times 0.18 \times 0.40 \times 0.23 \times 0.24)^{\mathbf{1/4}}$

$= 0.235$ i.e., 23.5%

Population in year 2021 is, $P_{2021} = 2585862 \times (1+ 0.235)^1$

$= 3193540$

Similarly, for year 2031 and 2041 can be calculated by,

$P_{2031} = 2585862 \times (1+ 0.235)^2 = 3944021$

$P_{2041} = 2585862 \times (1+ 0.235)^3 = 4870866$

4.2 Valves and Fittings[1] :

4.2.1 Resistance coefficients for valves and fittings and its applications

Valves are used to control the amount of flow and may be globe valves, angle valves, gate valves, butterfly valves, any of several types of check valves, And many more. See the figures below. The resistance is heavily dependent on the path ofthe fluid as it travels into, though, and out from the valve. With a more constricted path will cause more energy losses. Therefore, select the valve type with care if you desire the system you are designing to be efficient with relatively low energy losses.

Glop valve

It is one of the most common valves and is elatively inexpensive and it is one of the poorest performing valves in terms of energy loss.

Note that the resistance factor K is $K=(L_e/D)\mathbf{f}_T=340\mathbf{f}_T$

If the globe valve were used in a commercial pipeline system where throttling is not needed, it would be very wasteful of energy

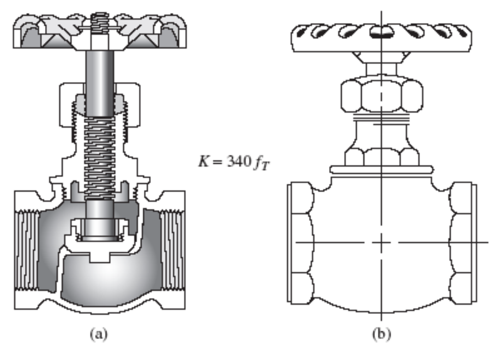


Figure 4.1:globe valve (Robert L. Mott2005)

Check Valves

The function of a check valve is to allow flow in one direction while stopping flow in the opposite direction. Typical use is shown in figure 4.2, in which a sump pump is moving fluid from a sump below grade to the outside of a home or commercial building to maintain a dry basement area.When open, the swing check provides a modest restriction to the

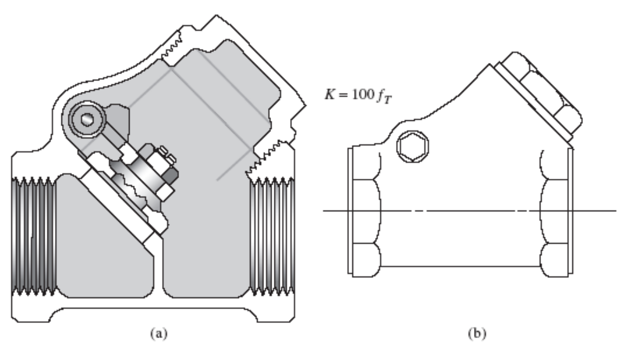


Figure 4.2: Check valve (Robert L. Mott2005)

[1] Applied Fluid Mechanics sixth edition in51 units Robert L. Mott

flow of fluid, resulting in the resistance factor of

$K=(L_e/D)\mathbf{f}_T=100\mathbf{f}_T$

The ball check causes more restriction because the fluid must flow completely around the ball ,see figure 4.3 However, the ball check is typically smaller and simpler than the swing check. Its resistance is

$K=(L_e/D)\mathbf{f}_T=150\mathbf{f}_T$

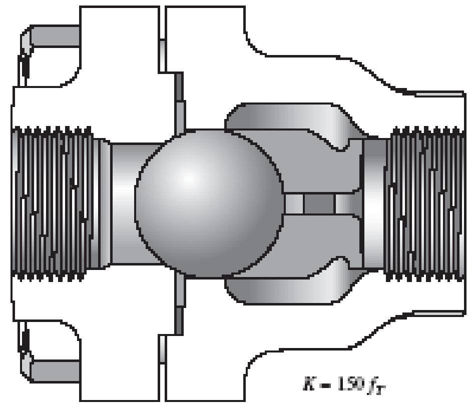


Figure 4.3: check valve- ball type (Robert L. Mott2005)

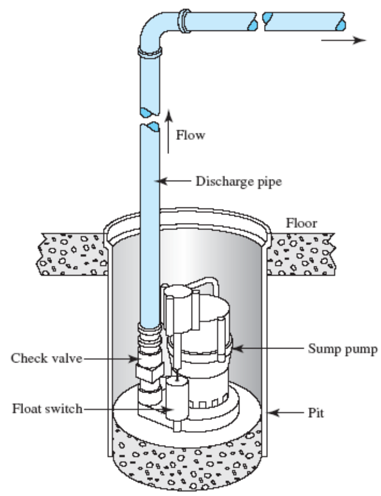


Figure 4.4: Typical use of check valve**(** Robert L. Mott2005)

Foot valves perform a similar function to that of check valves, see Figure 4.5. They are used at the inlet of suction pipes that deliver fluid from a source tank or reservoir to a pump. The resistances for the two kinds of foot valves shown are:

$K=(L_e/D)\mathbf{f}_T=420\mathbf{f}_T$ Poppet disc type

$K=(L_e/D)\mathbf{f}_T=75\mathbf{f}_T$ Hinged disc type

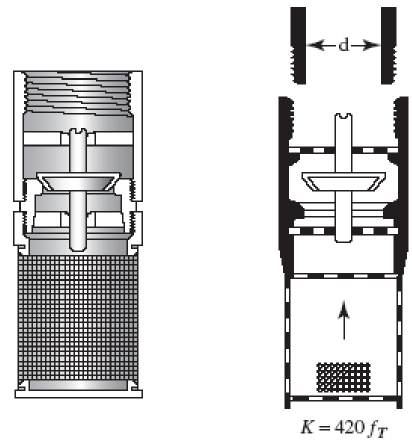


Figure 4.5: Foot valve with strainer-poppet disc type

(Robert L. Mott2005)

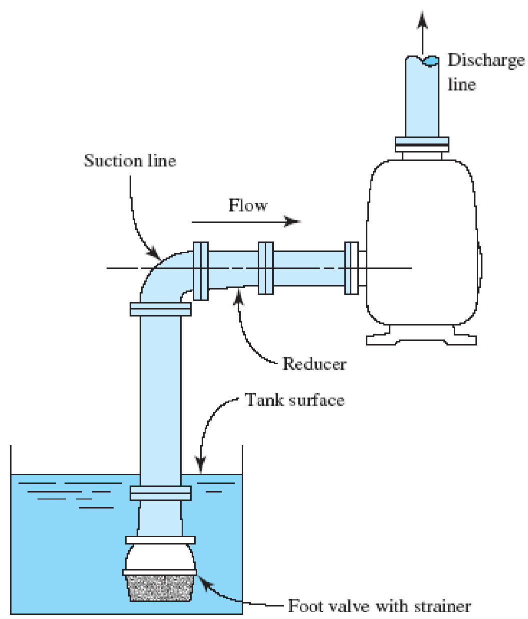


Figure 4.6 : Typical use of Foot valve with strainer
(Robert L. Mott2005)

Butterfly Valves

See Figure 4.7 .Closing the valve requires only one-quarter turn of the handle, and this is often accomplished by a motorized operator for remote operation. The fully open butterfly valve has a resistance of $K=(L_e/D)\mathbf{f}_T=45\mathbf{f}_T$

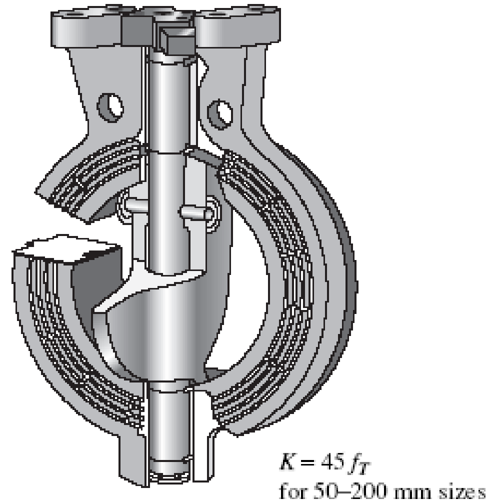


Figure 4.7: Butterfly valve (Robert L. Mott2005)

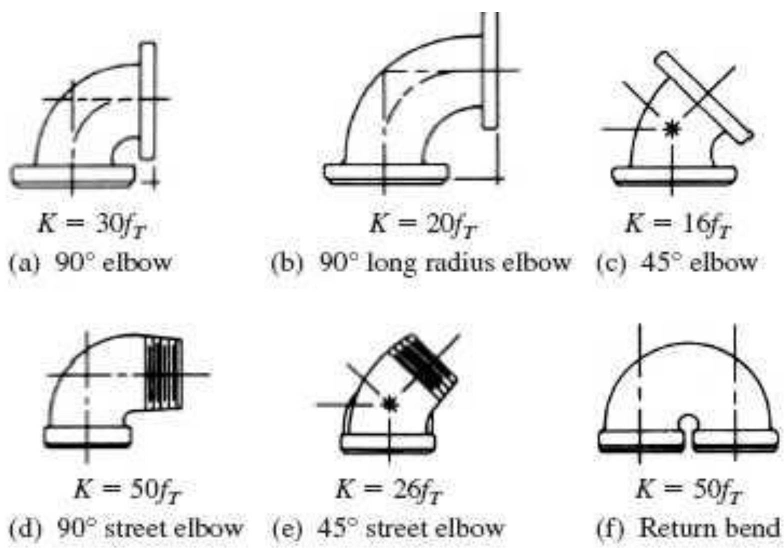


Figure 4.8: Pipe elbows

(Robert L. Mott2005)

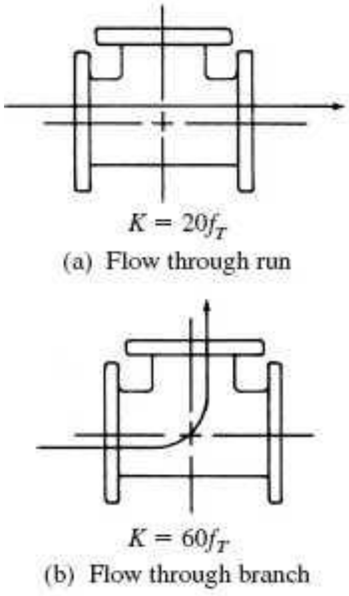


Figure 4.9: Standard tees

(Robert L. Mott2005)

However, the method of determining the resistance coefficient K is different. The value of K is reported in the form:

$$K = \left(\frac{Le}{D}\right) \boldsymbol{f_T} \qquad (\mathbf{4.1})$$

The term f_T is the friction factor in the pipe to which the valve or fitting is connected**,** taken to be in the zone of complete turbulence. Some system designers prefer to compute the equivalent length of pipe for a valve and combine that value with the actual length of pipe.

$$Le = K\frac{D}{\boldsymbol{f_T}} \qquad (4.2)$$

Table 4.2 shows the resistance in valves and fittings expressed as equivalent length in pipe diameters, Le>*D*

Table 4.2: The resistance in valves and fittings (CRANE VALVES ,SIGNAL HILL,CA)

Type	equivalent length in pipe Diameter (Le/D)
Global valve-fully open	340
Angle valve-fully open	150
Gate valve-fully open	8
3/4 open	35
1/2 open	160
1/4 open	900
Check valves-swing type	100
Check valves-ball type	150
Butterfly valve-fully open,50-200mm (2-8 in.)	45
250-350mm (10-14 in.)	35
400-600mm (16-24 in.)	25
Foot valve--poppet disc type	420
Foot valve--hinged disc type	75
90° Standard elbow	30
90° long radius elbow	20
90° street elbow	50
45° standard elbow	16
45°street elbow	26
close return bend	50
Standard tee-with flow through run	20
with flow through branch	60

Table 4.3: The friction factor in zone of complete turbulence for new, clean, commercial steel (source: Applied Fluid Mechanics sixth edition, Robert L. Mott)

Nominal Pipe Size (in.)	Friction Factor fT	Nominal Pipe Size (in.)	Friction Factor fT
½	0.027	3 1/2 , 4	0.017
¾	0.025	5	0.016
1	0.023	6	0.015
1 ¼	0.022	8-10	0.014
1 ½	0.021	12-24	0.013
2	0.019	18-24	0.012
2 1/2, 3	0.018		

Procedure for computing the Energy Loss Caused by Valves and Fittings using Eq 4.1

1) Find L_e/D for valve or fitting from Table 4.2

If the pipe is new clean steel Find f_T from Table 4.3.

2) Compute $K = f_T\,(L_e/D)$

3) Compute $h_L = K(v_P^2/2g)$, where v_P is the velocity in the pipe .

CHAPTER “5”

“FIELD WORK”

5.1. Ductile Iron Pipe

Ductile iron has shown itself to be most suitable material for pipelines. In addition to the intrinsic qualities of the basic metal, the variety of shapes and dimensions of components facilitate pipeline assembly.

The service reliability of a pipeline depends on its ability to resist ageing, exposure to soil, transported fluids or solids, temperature variations, impact and excess pressure.

Figure 5. 1: Sub-project Classic Pipes in Taweelah in the UAE

Figure 5. 2: Sub-project in Taksbt in Algeria

5.2 Field Work

The work began with visiting to the Beit Jala municipality and the contracting company in order to collect the necessary date for the case study and to arrange a field visit to the project area to take a general idea about the pipeline path, the required necessary work, and the tools to achieve our goals. The field surveys for the proposed pipeline were carried out byspectra_sp60 device.

The survey work for the proposed pipeline had been finished in five days under the supervision of Dr. Itissam Abu-Izia hand the Site engineer.

The main surveyed features were:

1. Concert well
2. Curbstone
3. Stone well

5.3 Civil Work

According to the decision made by the engineer of Beit jala municipality about the pipeline alignment and calculated flow, we started to draw the layout by civil 3D model according to manufacturer specifications.

5.4 Pipe Diameter Design:

Calculations which we are going to evaluate from Population Geometric increasing method for the next 30 year .We used this method due to availability of expanding in population domain in this region.

- Number of homes in Beit Jala city = 1580 homes (h) which we counted them from aerial photo of Beit Jala.
- We found from the Palestinian Central Bureau of Statistics general information which we considered in our procedures of solution as follows:

1. The average number of people in each home = 5 persons (p)
2. The average number of floors in each house = 2 floors (f)
3. The average number of apartment in each house = 2 apartments in each house (a)
4. The daily per capita consumption rate = 70 Liter (c)

Then we calculated the average number of resident (P) in Beit Jala as follows :

$$P = h * p * f * a \tag{5.1}$$

P = 1580 × 5 × 2 × 2 = 31600 persons

And then we calculated the increase in population for the next 30 years by using geometrical progression method as follows:

$$\mathrm{P}_n = \mathrm{P}\,(1 + \mathrm{I}_G/100)^n \tag{5.2}$$

Where, I_G = geometric mean (%), according to the Palestinian Central Bureau of Statistics is equal 2%

P = Present population

N = no. of decades.

$P_n = 31600\,(1+ 2/100)^{30}$

P_n = 57239 capita

And then we calculated the total consumption (Q) for the next 30 years as follows:

$$\mathrm{Q} = \mathrm{c} \times \mathrm{P}_n \tag{5.3}$$

Q =70 ×57239 = 4006731.8 L/c.d = 4006.7318 m^3/d

For 24 hours pumping working

Q= 4006.7318 m^3/d /24h=166.947m^3/h

So Q =166.947 m^3/h

And then we calculated the tube diameter of distribution line that will serve the Beit Jala area as follows:

$$D = 1.5\sqrt{Q} \quad (5.4)$$

Such that Q in m^3/s

Q = 166.947 m^3/h

D = 1.5 *√166.947 =1.5*12.9=19.381 cm

The choosing diameter according to standard pipes will be as follows:

D=200mm

5.4.1 Curve design:

The problem that we encountered in curve design is how to join the pipes with each other the following procedures are how to determine the deflection between pipes.

The pipes are connected to each other by joint in standard form as in (see figure 5.4), but the pipe are not necessary to be straight, it follows the train and the topography so, a deflection angle must be exist and should not exceed a threshold values presented in (see figure 5.3) and

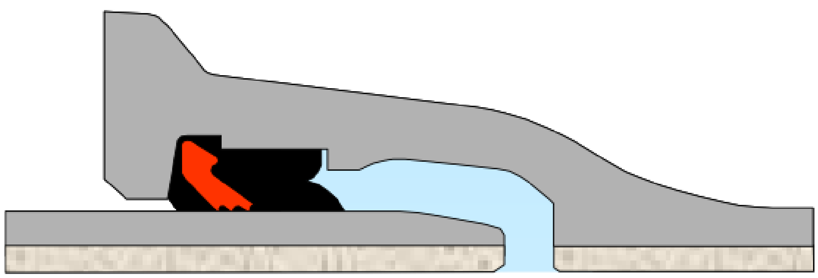


Figure 5. 3: Inner Joint structure(see table 5.1)

Figure 5. 4: Standard Vi Joint for pipes

Table 5. 1: Standard Vi Joint for pipes

DN	Class	Deflection	Axial gap aligned	Axial gap deflected	PFA
mm		degree	mm	mm	Bar
100	C40	5	33	22	16
150	C40	5	38	23	16
200	C40	4	42	22	16
250	C40	4	41	17	16
300	C40	3	38	9	16

These are some examples of the designed curves

1) L= 86.98m R= 531.56m Δ= 9.3759 d= 0.65

2) L= 14.52m R= 189.21m Δ= 4.3964 d= 2.10

3) L= 37.01m R= 189.61m Δ= 11.1849 d= 1.8

4) L= 40.19m R= 165m Δ= 13.9548 d= 2.30

Steps for determination of the deflection angle for the first curve

L= 86.98m R= 531.56m Δ= 9.3759 d=0 .65

- The length of pipe =6m
- # of pipe: 86.98/6= 14.49
- d =9.3759/14.49=0.65^{o}<4^{o}

Therefore, the deflection angle it is in rang.

5.4 Plan and Profile Work:

Using Civil 3D program to create a plan and profile for the path of pipeline starting with entering the points of surveying and connecting them to make the plan profile. (See figure 5.5)

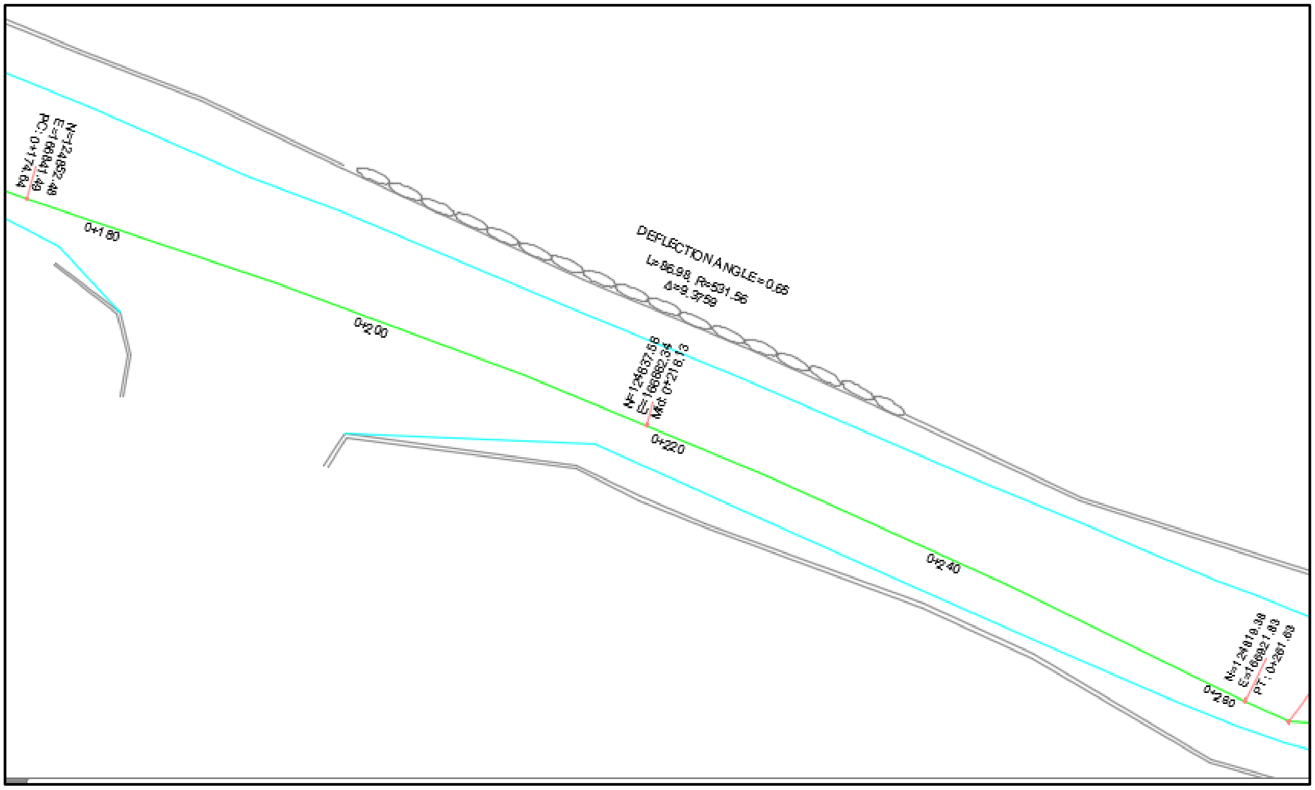


Figure 5. 5: Curve design

5.4GIS Work:

We visited Beit Jala Municipality to get a data about the project, they Provided us with an aerial photo to Beit Jala town and a contour map to identify the topography of the town.(see figure 5.6).

We used this information to create a map for Beit Jala, to create a digital elevation model for it and to use the aerial photo in counting the houses in Beit Jala, then calculate the Census of town. The importance of these maps is allow you to identify the path of project so easily. (see figure 5.7).

Figure 5. 6: Beit Jala topography

Figure 5. 7: Contour map of Beit Jala

Figure 5. 8: Path Line of the Pumping System

CHAPTER “6”

“USING BENTLY HAMMER”

6.1 Using Bentley Hammer.

Bentley HAMMER is a very efficient and powerful tool for simulating hydraulic transients in pipelines and networks, these detailed give your hands on experienced many of Bentley HAMMER's features and capabilities, and it will help you to explore and understand the following topics:

1) Pipeline Protection using Bentley HAMMER by assembling a pipeline using the graphical editor and performing two hydraulic transient analyses, without protection and with protection.

2) Network Risk Reduction using Bentley HAMMER by opening a water distribution network model created in Bentley Water GEMS and performing a hydraulic transient analysis using advanced surge protection and presentation methods.

Another way to become acquainted with Bentley HAMMER is to run and experiment with the sample files, located in the \Bentley\HAMMER8\Samples folder. Remember, F1 key has pressed to access the context-sensitive help at any time.

6.2 Pipeline Protection.

Bentley HAMMER is used to perform a numerical simulation of hydraulic transients in a water transmission main and, based on the results of our analysis, recommend suitable surge-protection equipment to protect this system from damage.

These three steps had been done:

1) Analyzed the system as it was designed (without any surge-protection equipment) to determine its vulnerability to transient events.

2) Select and model different surge-protection equipment to control transient pressures and predict the time required friction to attenuate the transient energy.

3) Present your results graphically to explain your surge-control strategy and recommendations for detailed design.

6.3 Creating or Importing a Steady-State Model.

Creating an initial steady-state model of our system within Bentley HAMMER directly, using the advanced Bentley HAMMER Modeler interface, or import one from an existing steady-state model created using other software.

A hydraulic transient was assembling model using both methods to learn their respective advantages and note the similarities between them.

6.4 Creating a Model.

Bentley HAMMER is an extremely efficient tool for laying out a water-transmission pipeline or even an entire distribution network. It is easy to prepare a schematic model and let Bentley HAMMER take care of the link-node connectivity and element labels, which are assigned automatically. For a schematic model, only pipe lengths must be entered manually to complete the layout. You may need to input additional data for some hydraulic elements prior to a run.

6.5 Selecting The Transient Events to Model.

Any change in flow or pressure, at any point in the system, can trigger hydraulic transients. If the change is gradual, the resulting transient pressures may not be severe. However, if the change of flow is rapid or sudden, the resulting transient pressure can cause surges or water hammer.

Since each system has a different characteristic time, the qualitative adjectives gradual and rapid correspond to different quantitative time intervals for each system. There are many possible causes for rapid or sudden changes in a pipe system, including power failures, pipe breaks, or a rapid valve opening or closure. These can result from natural causes, equipment malfunction, or even operator error. It is therefore important to consider the several ways in which hydraulic transients can occur in a system and to model them using Bentley HAMMER.

The impact of a power failure simulated lasting several minutes. It is assumed that power was interrupted suddenly and without warning (i.e., you did not have time to start any diesel generators or pumps, if any, prior to the power failure). The purpose of this type of transient analysis is to ensure the system and its components can withstand the resulting transient pressures and determine how long you must wait for the transient energy to dissipate.

For many systems, starting backup pumps before the transient energy has decayed sufficiently can cause worse surge pressures than those caused by the initial power failure. Conversely, relying on rapid backup systems to prevent transient pressures may not be realistic given that most transient events occur within seconds of the power failure while isolating the electrical load, bringing the generator on-line, and restarting pumps (if they have not timed out) can take several minutes.

Performing a Transient Analysis In this section, you will first simulate transient pressures in the system due to an emergency power failure without any protective equipment in service. After a careful examination of your results, you will select protective equipment and simulate the system again using Bentley HAMMER to assess the effectiveness of the devices you selected to control transient pressures.

6.6 Animating Transient Results at Points and along Profiles

Bentley HAMMER provides many ways to visualize the simulated results using a variety of graphs and animation layouts. You must specify which points and paths (profiles) are of interest, as well as the frequency to output prior to a run, or Bentley HAMMER will not generate this output to avoid creating excessively large output files. For small systems, you can specify each point and every time step, but this is not advisable for large water networks.

For the same reason, Bentley HAMMER only generates the Animation Data (for onscreen animations) if you select this option in the transient calculation options While you are still evaluating many different types or sizes of surge-protection equipment, you can often compare their effectiveness just by plotting the maximum transient head envelopes for most of your Bentley HAMMER runs. At any time, or once you feel you are close to a definitive surge-control solution, you can use Bentley HAMMER to generate the animation data files by setting Generate Animation Data to True in the Transient Calculation Options. After the run, you can open the Transient Results Viewer from the Analysis menu.

1) In the Transient Results Viewer, on the Profiles tab, select:

– Profile: Main.

– Graph Type: Hydraulic Grade and Air/Vapor Volume.

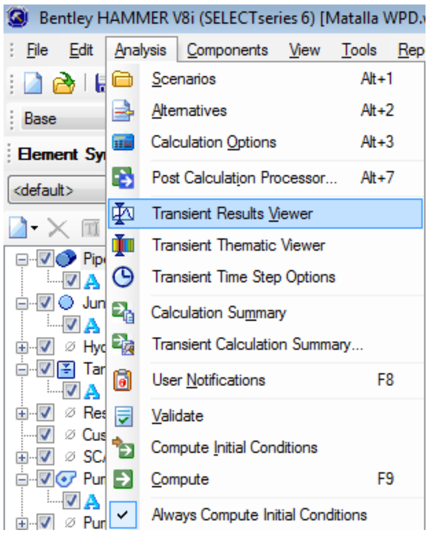


Figure 6.1: Transient results viewer

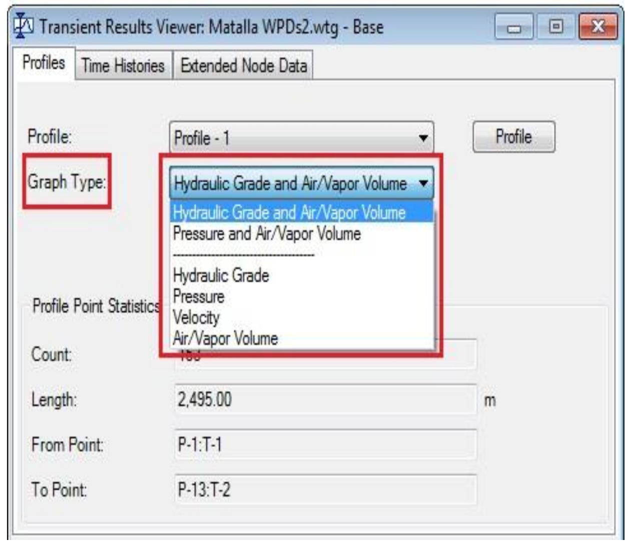


Figure 6.2: Graph type.

2) Click the Animate button. This loads the animation data and Animation Control.

3) On the Animation Controller, click the play button to start the animation.

6.7 Viewing Time History Graphs in Bentley HAMMER

Using the Bentley HAMMER Transient Results Viewer, you can plot a transient history at any point in the system to display the temporal variation of selected parameters (such as pressures and flow).

1) Click the Analysis menu and select Transient Results Viewer.

2) In the Time Histories tab, select: – Time History: P16:HT-1 – Graph Type: Hydraulic Grade, Flow, and Air/Vapor Volume.

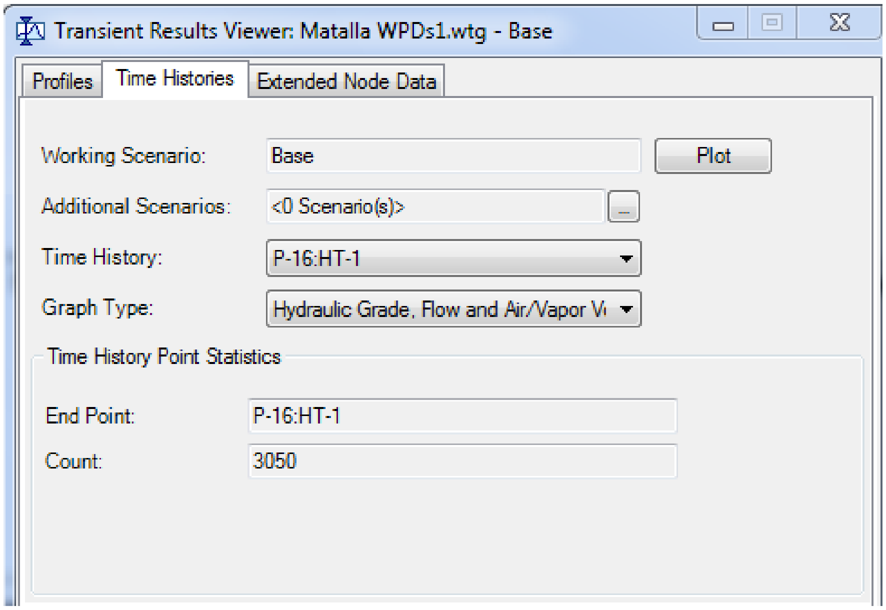


Figure 6.3: Transient results viewer - time histories.

3) Click Plot to display this transient history.

4) To view numerical data for the time history, click the Data tab. From here, you can sort the data by right clicking on the column header and choosing Sort. You can also change the units and precision for the results by right clicking on the column header and choosing Units and Formatting. Click OK to save these settings and leave the Flex Units Manager. From now on, Head will be displayed in ft. and Flow will be displayed in l/s.

CHAPTER “7”

"USING HAMMER TO ANALYSIS THE SYSTEM”

7.1 General Description of Al Matallah Pumping Station

Al matallah pumping station is currently equipped with pumping units 99 m^3\h at 50 mWc .

The pumping station shall discharge water through a steel pipe ND 200 mm (8”) length 2486 m to beit jala reservoir.

The facilities consist of the main following equipment:

1) A supply tank.
2) A pumping room with the pumps, the valves, a pressure tank against water hammer effect, instrumentation and ancillaries, the switch gears.
Hydro and electro-mechanical equipment mainly including:

1) Pumps:
a) Rated point: 99 m^3\h (27.5 l\s) at 50 mWC head;
b) Hydraulic efficiency: not less than 75%
c) Design pressure: NP 16 bars;
2) Motors.

7.2 Hydraulic Analysis for Al Matallah Pumping Station (Steady State).

7.2.1 System Description:

The following figure explains the components of the system being analyzed. We drew this sketch using hammer cad according to earth surface:-

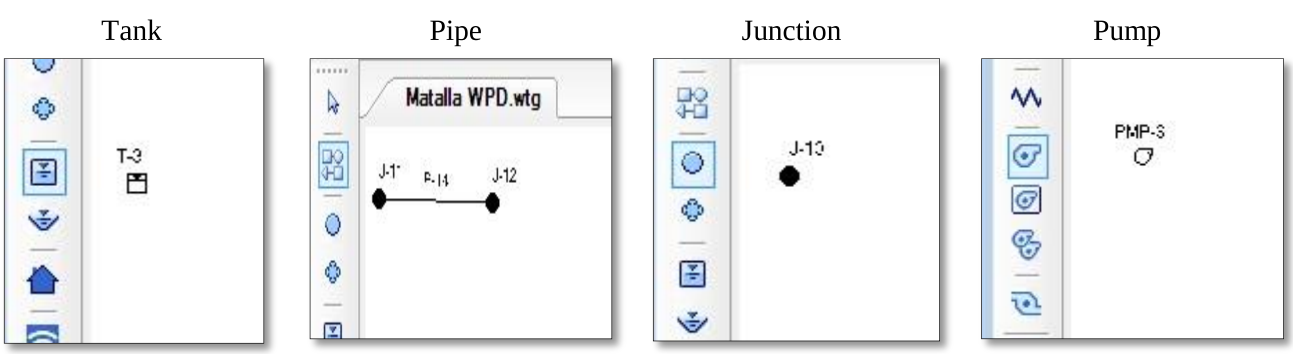


Figure 7.1: Feature of the project.

This figure shows the general shape of the system including pipes, junction, pumps, tank, Flow direction and it is describe approximately the elevations of pipe lines.

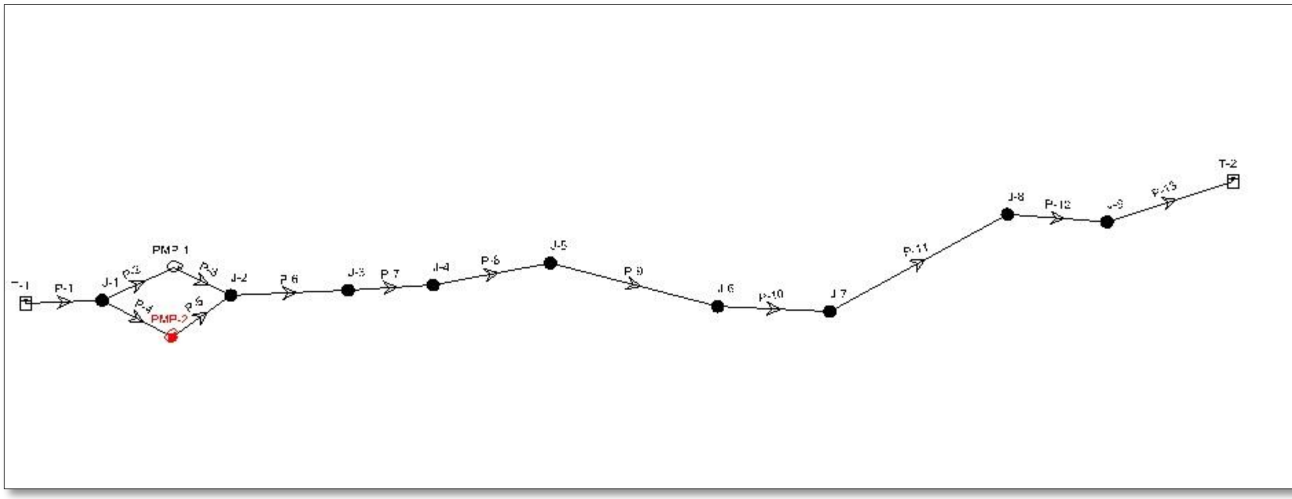


Figure 7.2: System description.

7.3 calculate head lose:

7.3.1 Solution of Colebrook and white equation:

Q(m^3\h)	99
Headless(m)	108

Total flow=2*99m^3/h=0.055m^3/s

Area=$\pi\frac{d^2}{4}$=$\frac{\pi*(0.2)^2}{4} = 0.0314m^2$

Average Velocity=$\frac{Q}{A} = \frac{0.055}{0.0314} = 1.7505m/s$

<u>Input:</u>

e: pipe roughness= 0.0003m

D: Pipe internal diameter= 0.2m

L: Pipe length=2486m

V: Average fluid velocity=1.7505 m/s

v: Kinematic viscosity=0.0000008m^2/s

ρ: Density=998Kg/m^3

Colebrook and white equation.

$$\sqrt{f} \times \left[-2 \times \log_{10}\left(\frac{e/D}{3.7} + \frac{2.51}{Re_{pipe} \times \sqrt{f}} \right) \right] = 1 \qquad (7.1)$$

f= 0.02224

Darcy Equation:

$h_{f=}$f*($\frac{L}{D}$)*$\frac{V^2}{2g}$ $h_{f=}$ 43.228 m (7.2)

Total head lose = 43.228 + (43.228*.20) = 51.87 m

Total head = head lose + geometric head = 51.87 + 55.9 ≈ 108 m

7.3.2 System component:

We draw a simple sketch for the system to show the component of a system:

1) The elevation for two tanks

2) Total head loss

3) Profiles of the system.

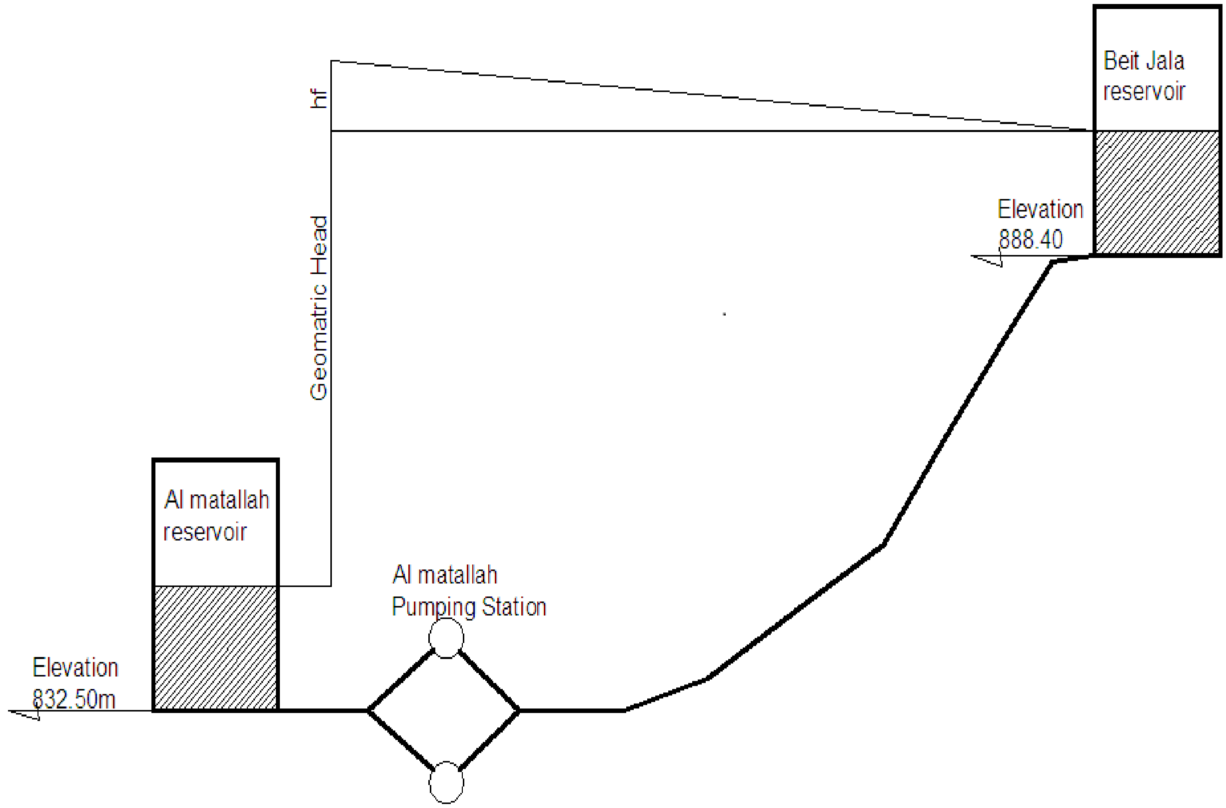


Figure 7.3: System component.

7.4 Pump curve

The following graph between head loss and flow rate shown the one pump behavior, when the flow rate increasing the head loss decreasing, e.g. (when flow rate =25L\s the head loss =108m).

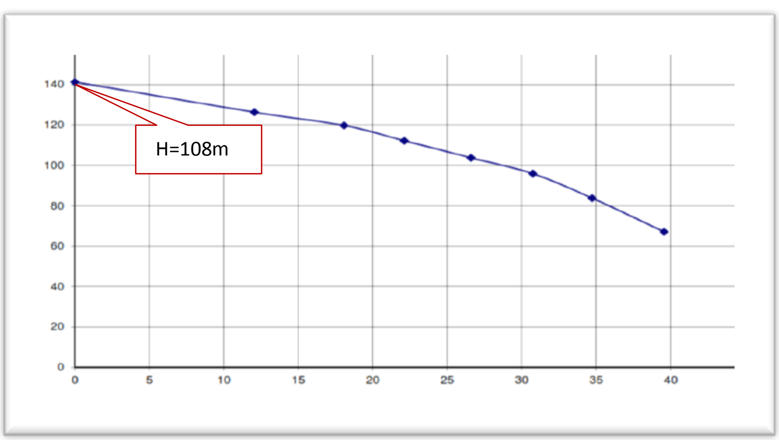


Figure 7.4: Relationship between head and flow rate (for one pump).

This graph explain the behavior of two pumps and system curve between flow rate and head loss , the result of intersection curves between two pumps, this is called duty point as follow.

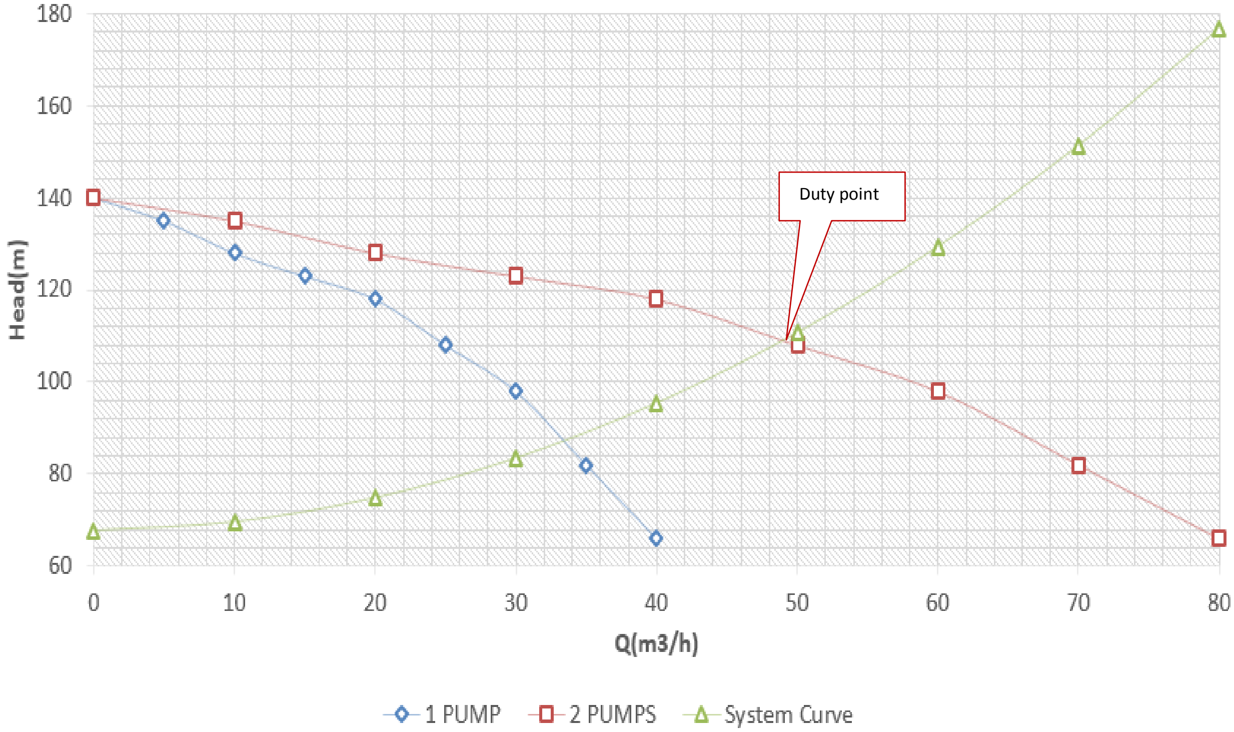


Figure 7.5: Relationship between head loss and flow rate (for two pumps).

The following first curve figure between power and flow rate shows how pump behavior as explain at flow rate =25l\s the power of pump =36kw.

The second curve figure between efficiency and flow rate shows how pump behavior as explain at flow rate =25l\s the efficiency=72.5%.

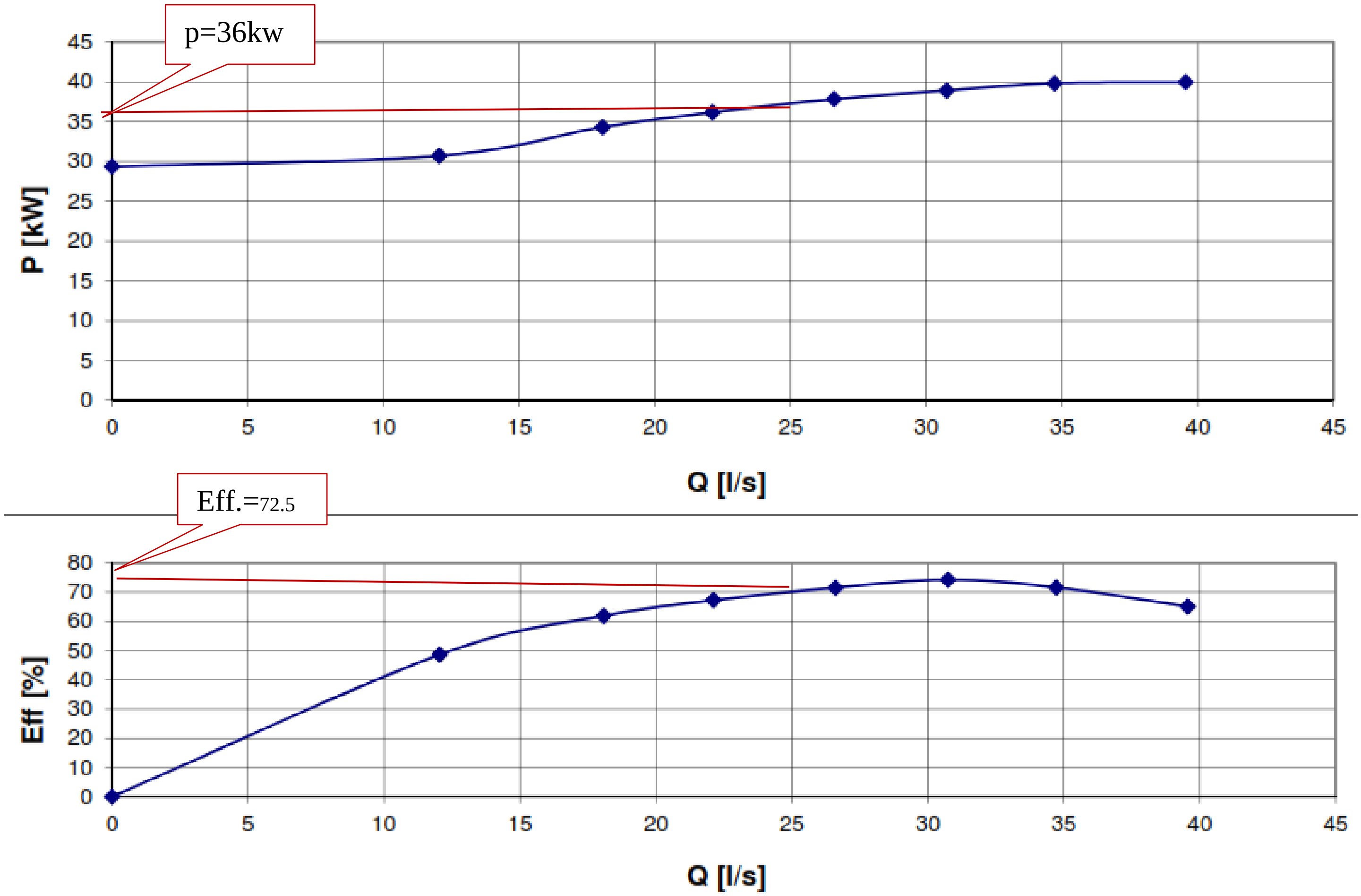


Figure 7.6: Relationship between flow rate and (power-efficiency).

7.5 Transient State Analysis

7.5.1 Expression of the wave speed (a):

Overpressure or depression due to water hammer is strongly dependent on the wave speed propagation in the pipeline. The exact determination of the module is essential for the assessment of overpressures and depressions generated by some sort of maneuver.

The pressures wave speed depends on the fluid characteristics (its density and its compressibility) and those of the pipeline (modulus of elasticity, diameter, wall thickness, etc.)

From the above graphs the required pumps duty point is (108m, 27.5 l/s) with efficiency about 72.5%.

The general expression for the wave speed is:

$$a=\frac{\sqrt{K/\rho}}{\sqrt{1+c\frac{KD}{eE}}} \tag{7.3}$$

Where:

c = 1-μ^2 , the pipeline is anchored against longitudinal movement.

K and ρ = the bulk modulus of elasticity and density of the fluid respectively.

D and e = the inner diameter and thickness of the pipe respectively.

E = young modulus of the pipe material.

C = coefficient that accounts for the pipe support conditions.

c = 1- $.3^2$= .91

$$a = \frac{\sqrt{K/\rho}}{\sqrt{1+c\frac{KD}{eE}}} = \frac{\sqrt{2188128000/998}}{\sqrt{1+.91\frac{2188128000*200}{3.945*207000000000}}} = 1214.04 \text{ m/s}$$

7.5.2 Moment of inertia

1) Pump impeller moment of inertia:

The moment inertia of a pump impeller may be estimate using the method relationship proposed by Wyllie as shown below.

$$P=\frac{\rho*Q*h*g}{n*3.6*10^6} \tag{7.4}$$

$$P=\frac{998*99*108*9.81}{0.7*3.6*10^6} = 41.54 \text{ KW}$$

$I_p = 1.5 * 10^7 *(\frac{P}{N^3})^{.9556}$

$I_p = 1.5 * 10^7 * (\frac{41.54}{1450^3})^{.9556} = 0.46\ Kg.m^3$

2) Motor moment of inertia:

The inertia of the pump motor is typically the largest contributor to the pump moment of inertia similarly to the pump impeller it may be estimated using a relationship presented by Wyllie etc. As shown below:

$$I_m = 118 *(\frac{P}{N})^{1.48} \tag{7.4}$$

$I_m = 118 *(\frac{41.54}{1450})^{1.48} = 0.614\ Kg.m^2$

Total Moment of Inertia = $(I_p + I_m) + 10\% *(I_p + I_m)$

Total Moment of Inertia = (0.457+0.614) + 10% *(0.457+0.614) =1.17 kg.

7.6 System Data

All data that we get it from the field work and the pervious calculation were used and insert in hammer program according to the following:

1) Tanks data

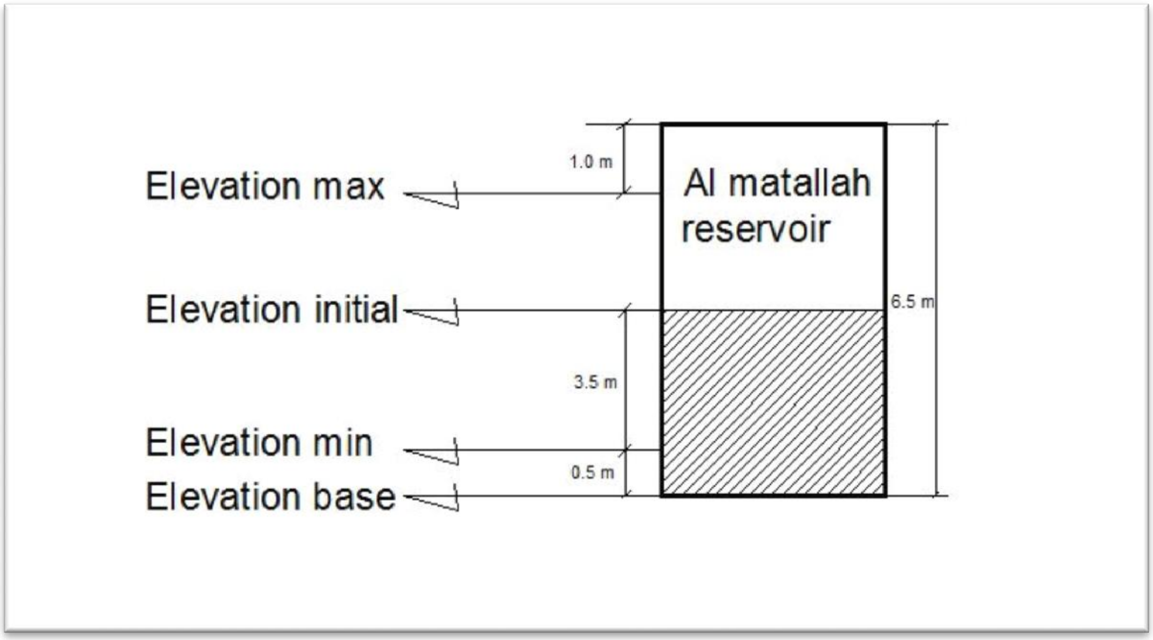


Figure 7.7: Tank data.

For tank's we need data about the elevation of base, minimum, maximum and initial as shown in the figure below:

Table 7.1: Tanks data.

Label	Elevation (Base) (m)	Elevation (Minimum) (m)	Elevation (Maximum) (m)	Elevation (Initial) (m)	Hydraulic Grade (m)
T-1	832.50	833	838	836.50	836.50
T-2	888.40	889	893	893	893

2) Pumps data:

Pumping station are facilities including pumps and equipment for pumping fluids from one place to another, the following figure explain the location of pumps and the flow direction.

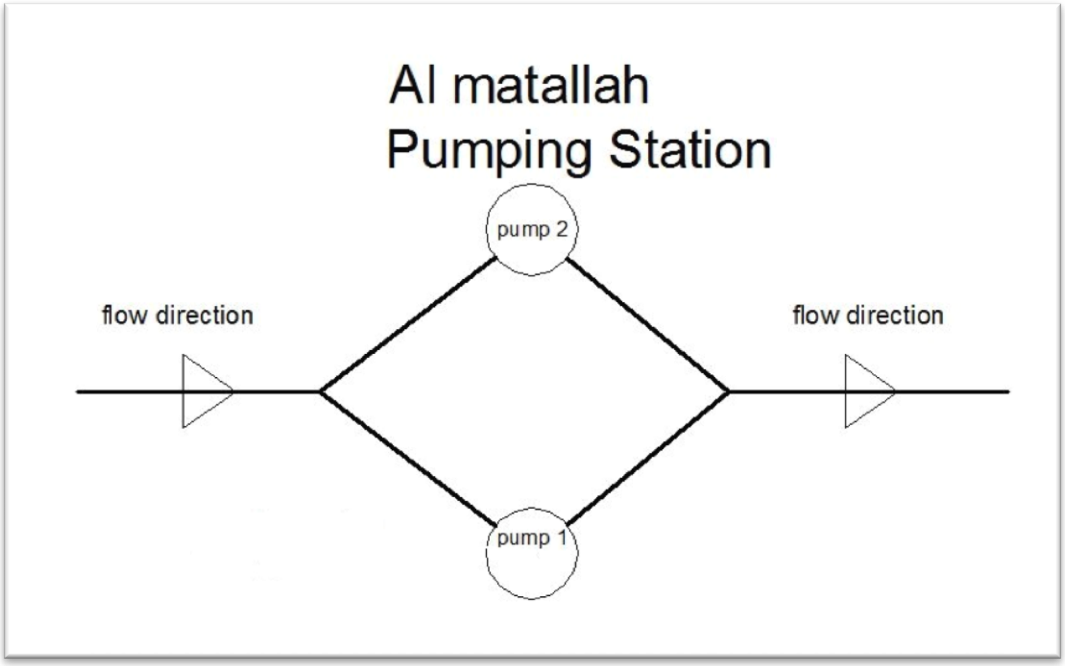


Figure 7.8: Al-Mattallah pumping station

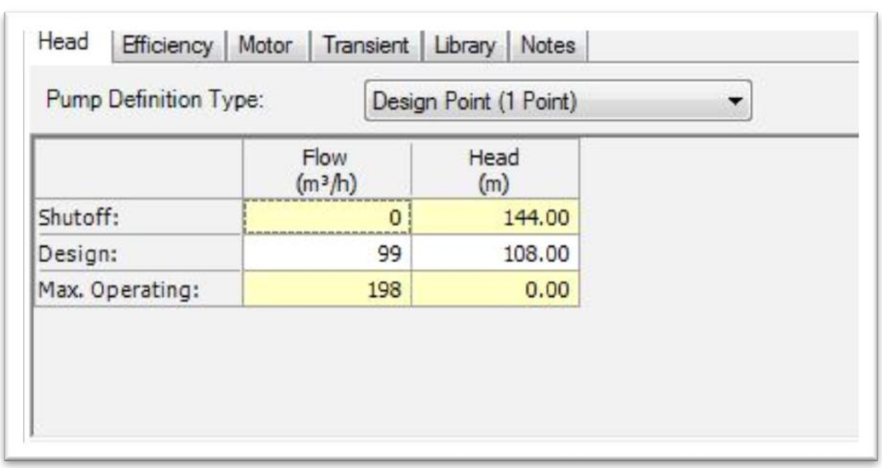
a) Pumps elevation:

Table 7.2: Pump's data.

Label	Elevation (m)	Flow (Total) (m^3/h)	Pump Head (m)
PMP-1	832.50	99	108.0
PMP-2	832.50	99	108.0

b) Pump definition

1) Pump head:



	Flow (m³/h)	Head (m)
Shutoff:	0	144.00
Design:	99	108.00
Max. Operating:	198	0.00

Figure7.9: Pump head.

2) Pump efficiency:

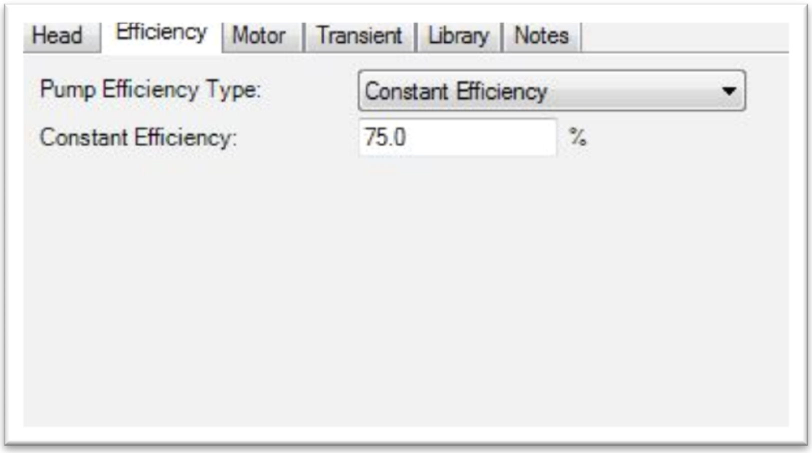


Figure 7. 10: Pump efficiency.

3) Moment of inertia:

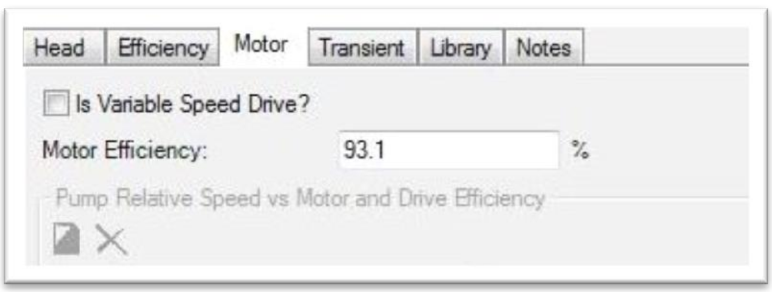


Figure 7. 11: Motor efficiency.

4) Pump transient:

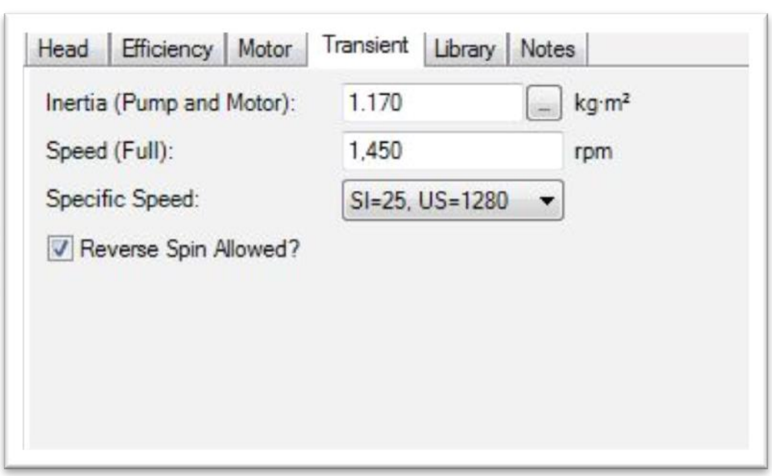


Figure 7.12: Moment of inertia.

4) Pipe data

Pipes used to convey fluids from one location to another, and the parameters that required to input in the hammer cad is: length, calculated diameter, type of pipe, coefficient of darcy-weisbach and wave speed as shown below:

Table7.3: Pipe's data.

Label	Length (m)	Start Node	Stop Node	Diameter	Material	Darcy-Weisbach	Wave Speed
p-1	3	T-1	J-1	200	ductile	0.0003	1214.04
p-2	3	J-1	PMP-1	150	ductile	0.0003	1214.04
p-3	3	PMP-1	J-2	150	ductile	0.0003	1214.04
p-4	3	J-1	PMP-2	150	ductile	0.0003	1214.04
p-5	3	PMP-2	J-2	150	ductile	0.0003	1214.04
p-6	206	J-2	J-3	200	ductile	0.0003	1214.04
p-7	160	J-3	J-4	200	ductile	0.0003	1214.04
p-8	360	J-4	J-5	200	ductile	0.0003	1214.04
p-9	520	J-5	J-6	200	ductile	0.0003	1214.04
p-10	275	J-6	J-7	200	ductile	0.0003	1214.04
p-11	465	J-7	J-8	200	ductile	0.0003	1214.04
p-12	320	J-8	J-9	200	ductile	0.0003	1214.04
p-13	180	J-9	T-2	200	ductile	0.0003	1214.04

4) Junction data:

In a fluid dynamics, pipe network analysis is the analysis of the fluid flow through a hydraulics network, containing several or many interconnected branches (junction), the aim is to determine the flow rates and pressure drops in the individual sections of the network, this is required to insert data includes: elevation, and hydraulic grade which defined as the elevations to which the water would rise.

Table 7.4: Junction's data.

Label	Elevation(m)	Hydraulic grade(m)
J-1	832.50	836.26
J-2	832.50	936.94
J-3	845.64	933.3
J-4	846.15	930.47
J-5	867.53	924.11
J-6	816.55	914.92
J-7	815.43	910.05
J-8	880.30	901.84
J-9	876.40	896.18

7.7 Simulation Results Without Including the Protection Device:

Where the results were obtained by doing these steps:

1) Click the Compute Initial Conditions button. Close the Calculation Summary and the User Notifications dialog.

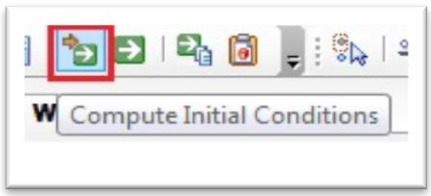


Figure 7.13: Compute initial conditions.

2) Click the Compute button. Close the Transient Calculation Summary and the User Notifications dialog.

Figure 7.14: Compute transients condition.

1) Pipes data results during transients:

This table explain the relationship between the previous data (pipes data, pump data, junctions data and tank data) and the results after calculated. The results is represented in a columns head maximum transient and head minimum transient, all this in the steady state as following:

Table 7.5: Pipe's data.

Label	Diameter (mm)	Material	Darcy-Weisbache (m)	Wave Speed (m/s)	Head (Maximum Transient) (m)	Head (Minimum Transient) (m)
P-1	200	Ductile	0.00003	1214.04	843.41	830.51

		Iron				
P-2	150	Ductile Iron	0.00003	1214.04	846.43	830.51
P-3	150	Ductile Iron	0.00003	1214.04	1050.02	822.5
P-4	150	Ductile Iron	0.00003	1214.04	846.43	830.51
P-5	150	Ductile Iron	0.00003	1214.04	1050.02	822.5
P-6	200	Ductile Iron	0.00003	1214.04	1055.68	822.5
P-7	200	Ductile Iron	0.00003	1214.04	1057.63	835.63
P-8	200	Ductile Iron	0.00003	1214.04	1064.1	836.15
P-9	200	Ductile Iron	0.00003	1214.04	1045.88	822.92
P-10	200	Ductile Iron	0.00003	1214.04	1051.09	820.81
P-11	200	Ductile Iron	0.00003	1214.04	1040.64	825.4
P-12	200	Ductile Iron	0.00003	1214.04	994.97	866.4
P-13	200	Ductile Iron	0.00003	1214.04	979.38	866.4

3) Pumps results during transients:

The following table shows the result data, which belongs to pump which was head maximum, and minimum transient. This is needed to insert in hammer program: elevation, flow rate and pump head.

Table 7.6: Pump's data

Label	Elevation (m)	Flow (Total) (m^3/h)	Pump Head (m)	Head (Maximum Transient) (m)	Head (Minimum Transient) (m)
PMP-1	832.5	110	93.37	1050.02	822.5
PMP-2	832.5	110	93.37	1050.02	822.5

3) Tanks results during transients:

The following table shows the result data which belongs to tank, which was head maximum and minimum transient and this, is needed to insert hydraulic grade as follows:

Table 7.7: Tank's data

Label	Hydraulic Grade(m)	Head (Maximum Transient) (m)	Head (Minimum Transient) (m)
T-1	836.50	836.50	836.50
T-2	893.00	893.00	893.00

4) We updated our report points and report path to reflect the replacement of J1 withHT-1. Click Analysis > Calculation Options and double-click the Base Calculation Options under the Transient Solver.

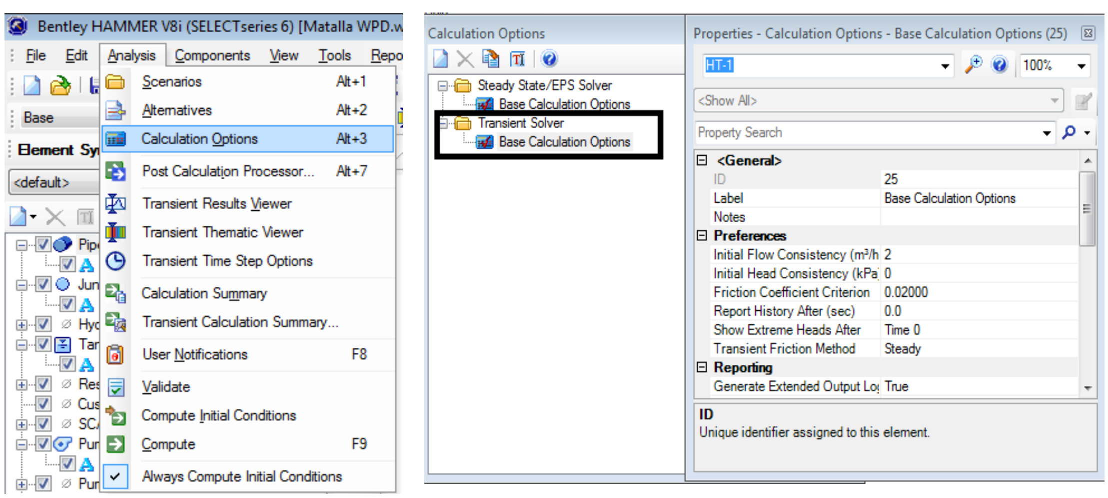


Figure 7.15: Calculation options under the transient solver.

5) Click the ellipsis button in the Report Points Collection field.

6) Add P1 / HT-1 and P2 / HT-1 to the Selected Items list. Click OK.

7) Click View > Profiles and Edit the Main Path Profile. Click Yes when prompted to auto repair the profile. The profile will open and will now include the hydro pneumatic tank, close the Profile and the Profiles manager.

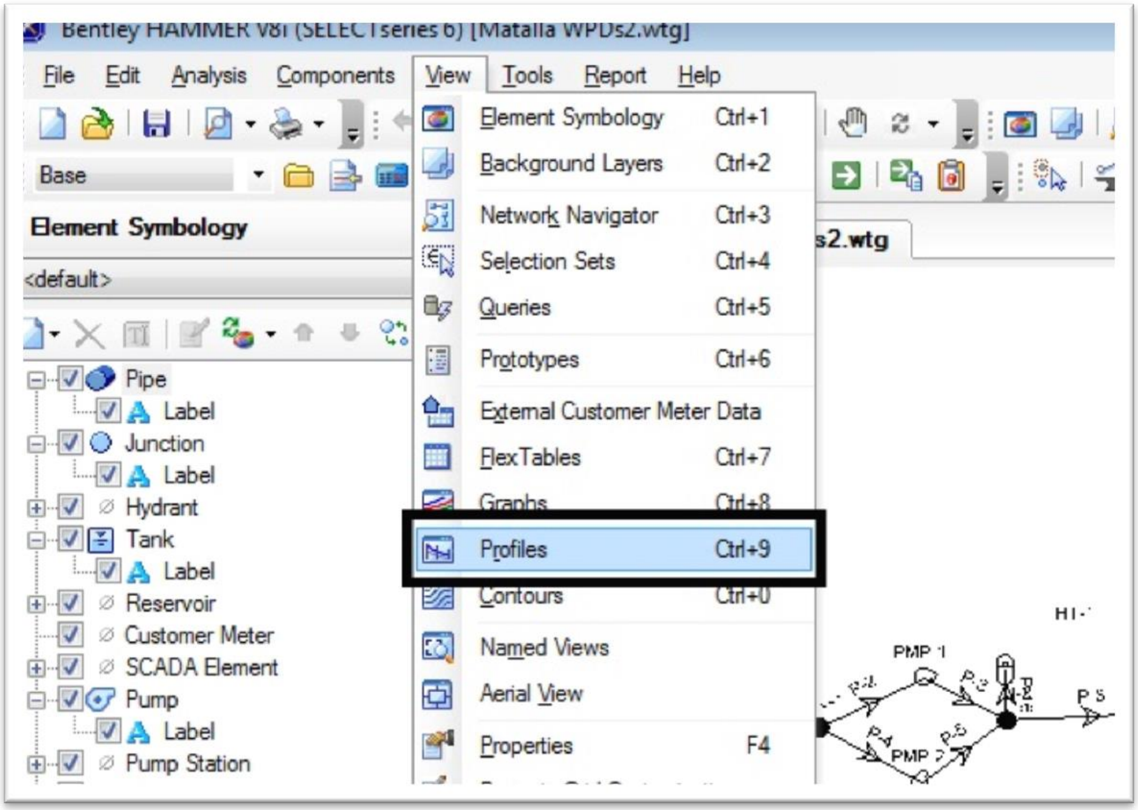


Figure 7.16: Shown the profile in Hammer Cad.

7) Select File > Save As and save the file with a new name: project1_Protection.wtg.
Note: Rather than editing the original model and saving it as a new file, a better way is to create a new scenario in the original model for the transient protection simulation; we will investigate scenarios in project.

7.9 Analysis of the System by Profiles:

1) Pipeline profile VS hydraulic grade line HGL:

The following figure shows the pipeline profile and hydraulic grade line, which is the highest point, equals 936.94 and lowest point equals 888.40, through this figure, we can know elevations of junction from pipeline profile.

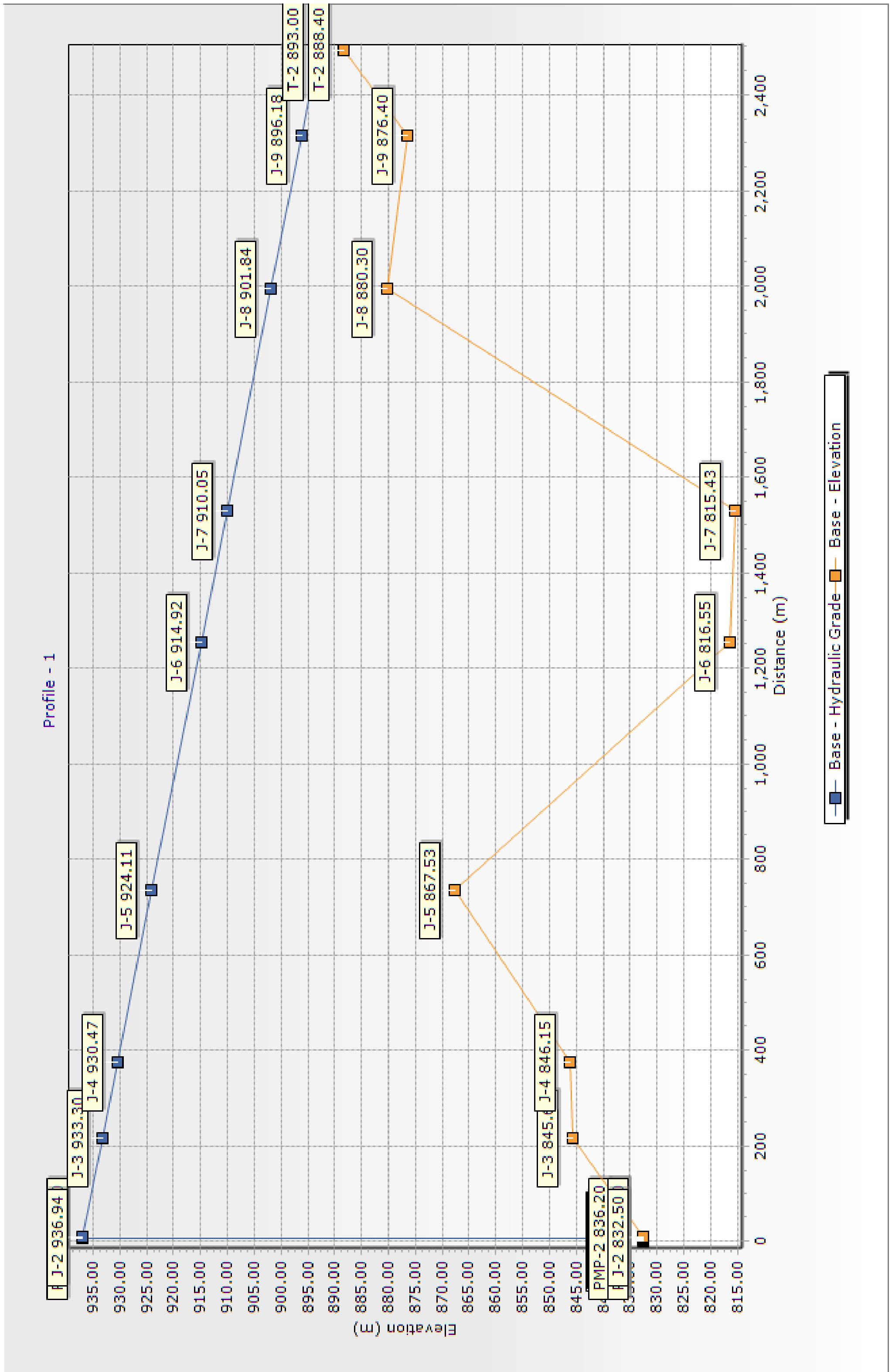


Figure 7.17: Pipeline Profile VS hydraulic grade line HGL

2) Pressure and flow VS time:

The following figure shows the pressure envelope and flow variations with time gust at the pump upstream due to to the pump stopping:

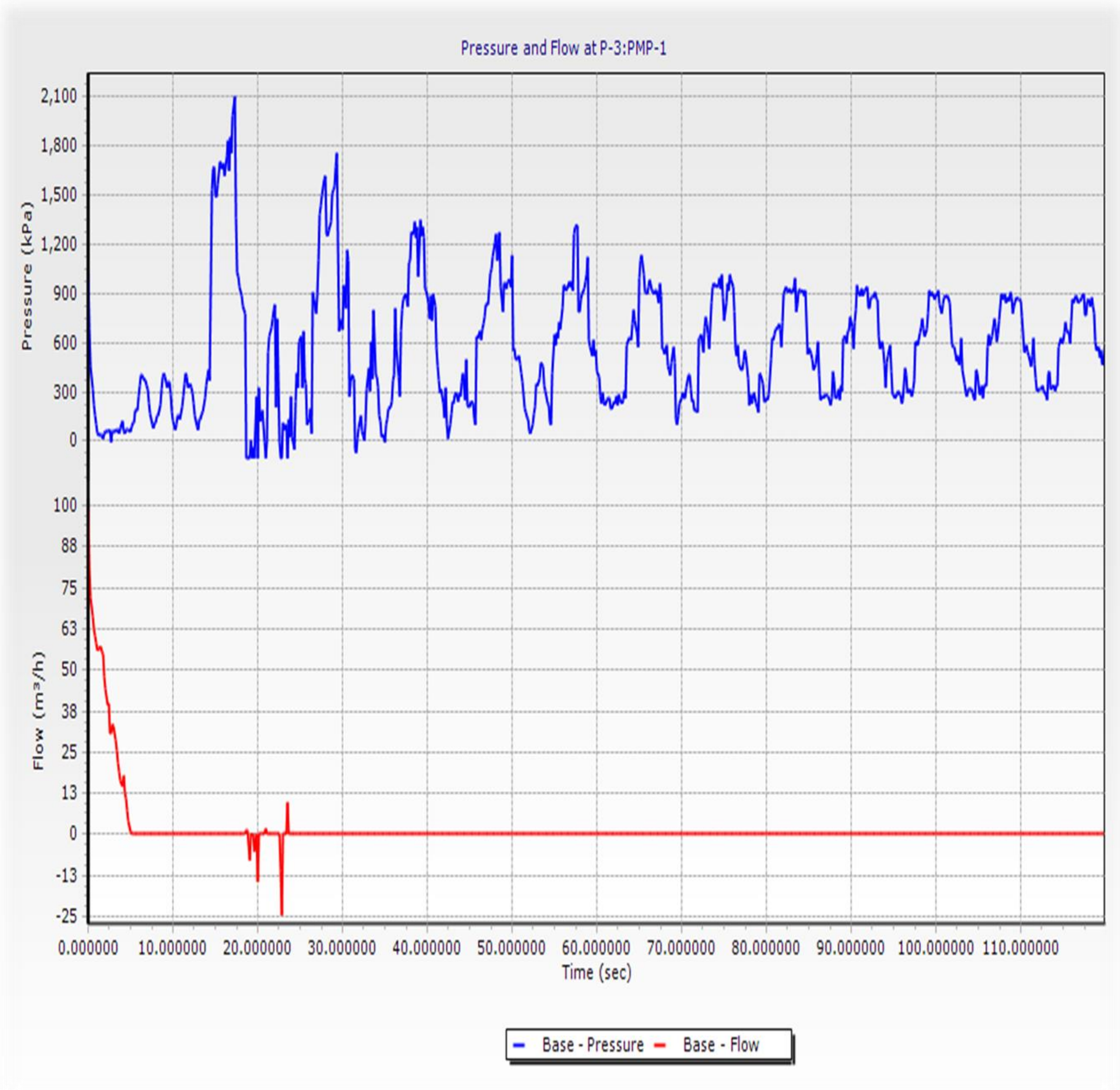


Figure 7.18: Pressure envelope and flow variations with time.

3) Pressure and flow VS time

The following figure shows the pressure envelope and flow variations with time at intermediate due to to the pump stopping:

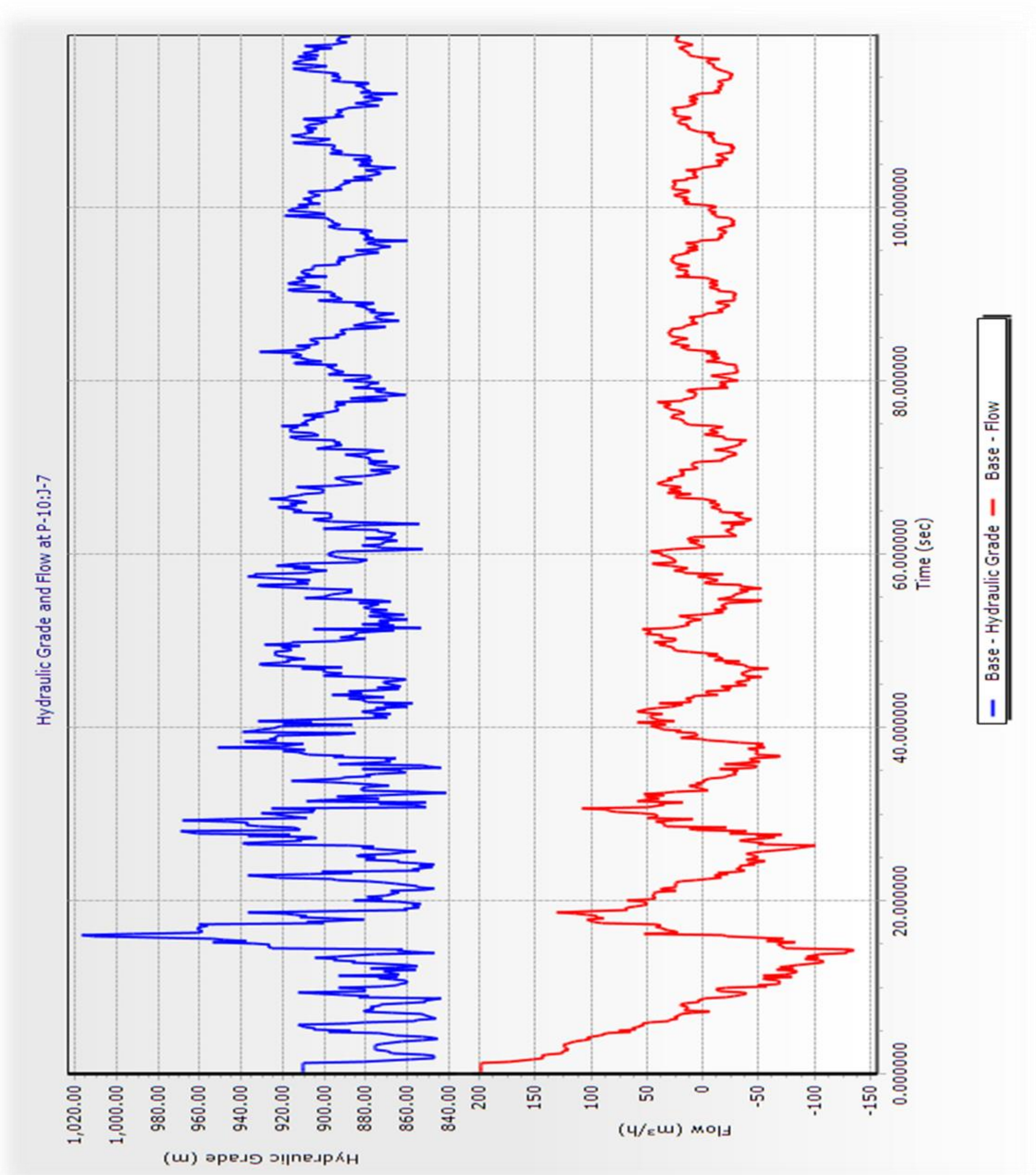


Figure 7.19: Pressure envelope and flow variations with time.

4) The following figure shows the pressure envelope with time at the tank location:

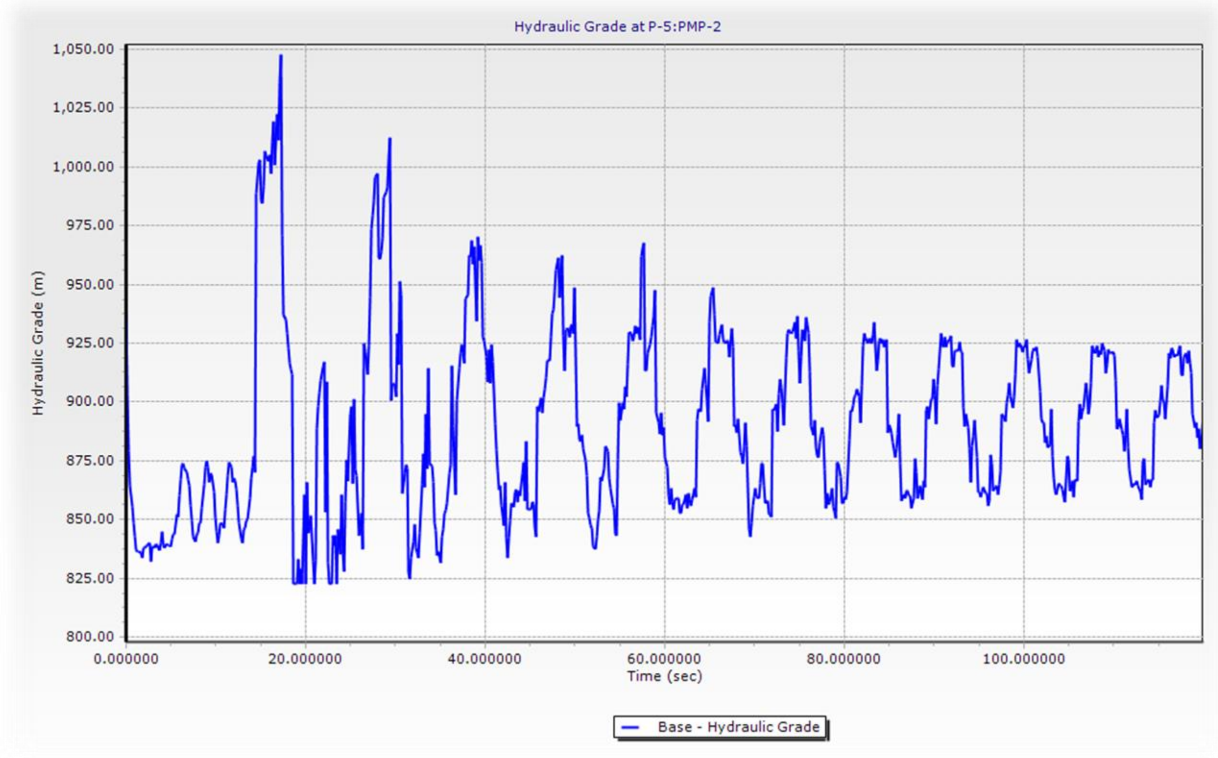


Figure 7.20: Pressure envelope with time at the tank location.

5) The following figure shows the hydraulic grade lines along the entire pipe length:

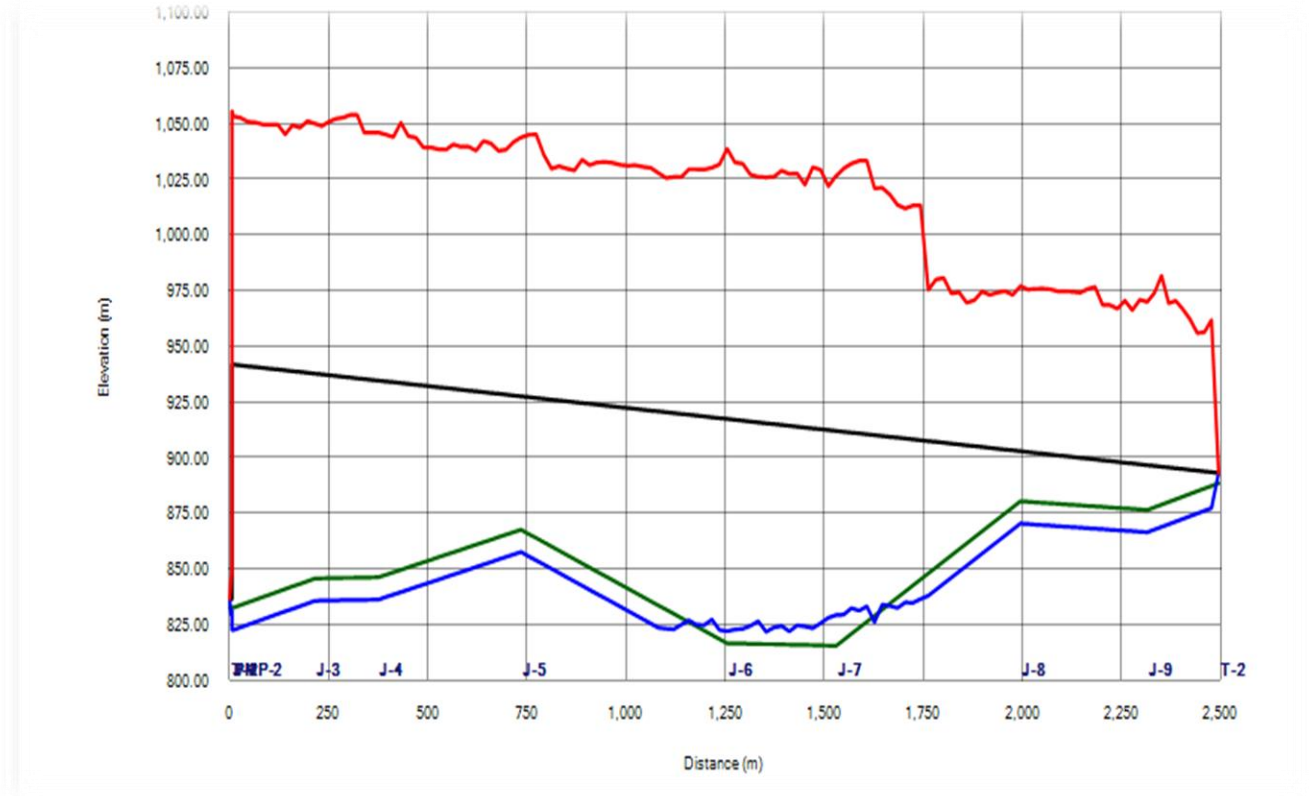


Figure 7.21: Hydraulic grade lines along the entire pipe length.

As it appears that a high pressure transient will be exist also a minimum pressure transient is located below pipe line profile from station 0+000 to station 1+250 and from station 1+500 to 2+1450 which mean a cavitation will be exsist in order avoid the dangers transient effect a protictive devices should be used .

7.10 Surge-Protection Equipment

The ways to control water hammer are classified as either passive or active protections, protections under the first category do not require any special devices and limited to protective measures taken during the design of the facility, this is particularly the choice of the most suitable diameters of the pipes, since the average velocity of the flow is not excessive, the choice of material and class of the pipe rating pressure to reduce pressure changes with a little risk of bursting or crushing, and finally the choice of the valves whose closure durations are relatively slow, when these passive measures show insufficient protection, then it is necessary to install, one or more protective devices against overpressure and\or low pressure occur in the transient phenomena.

A brief over view of various commonly used surge protection devices and their functions is provided below.

Some methods of transient prevention [Walski at 2003]:

- Slow opening and closing of valves: Generally, slower valve operating times are required for longer pipeline systems. Operations personnel should be trained in proper valve operation to avoid causing transients.
- Proper hydrant operation: Closing fire hydrants too quickly is the leading cause of transients in smaller distribution piping. Fire and water personnel need to be trained on proper hydrant operation.
- Proper pump controls: Except for power outages, pump flow can be slowly controlled using various techniques. Ramping pump speeds up and down with soft starts or variable-speed drives can minimize transients, although slow opening and closing of pump control valves downstream of the pumps can accomplish a similar effect, usually at lower cost. The control valve should be opened slowly after the pump is started and closed slowly prior to shutting down the pump.
- Lower pipeline velocity: Pipeline size and thus cost can be reduced by allowing higher velocities. However, the potential for serious transients increases with decreasing pipe size. It is usually not cost-effective to significantly increase pipe size to minimize transients, but the effect of transients on pipe sizing should not be ignored in the design process.

To control minimum pressures, the following can be adjusted or implemented; Pump inertia, Surge tanks, Air chambers, One-way tanks, Air inlet valves, and Pump bypass valves. To control maximum pressures, the following can be implemented; Relief valves, Anticipator relief valves, Surge tanks, Air chambers, and Pump bypass valves. These items can be used singly or in combination with other devices [Walski at 2003].

A. Pump Inertia

Pump inertia is the resistance of the pump to acceleration or deceleration. Pump inertia is constant for a particular pump and motor combination. The higher the inertia of a pump, the longer it takes for the pump to stop spinning following its shutoff and vice versa. Larger pumps have more inertia because they have more rotating mass. Pumps with higher inertias can help to control transients because they continue to move water through the pump for a longer time as they slowly decelerate. Pump inertia can be increased through the use of a flywheel. For long systems, the magnitude of pump inertia needed to effectively control transient pressures makes this control impractical due to the mechanical problems associated with starting high inertia pumps. Therefore, increasing pump inertia is not recommended as an effective option for controlling transient pressures for long piping systems [Walski at 2003].

B. Air Chambers and Surge Tanks

Air chambers and surge tanks work by allowing water out of the system during high pressure transients and adding water during low pressure transients. They should be located close to a point where the initial flow change is initiated.

An air chamber is a pressure vessel that contains water and a volume of air that is maintained by an air compressor. During pump stop, the pressure and flow in the system decreases and as a result the air in the air chamber expands, forcing water from it into the system.

A surge tank is a relatively small open tank connected to the hydraulic system. It is located such that the normal water level elevation is equal to the hydraulic grade line elevation. During pump stop, the surge tank substitutes the pump and by gravity feeds the system with water. This controls the magnitude of the low pressure transient generated as a result of the pump stop.

A. One-Way Tank

This is a storage vessel under atmospheric pressure that is connected to the hydraulic system. It has a check valve (normally closed) connected to it which only allows water from the tank into the system. One-way tanks are primarily used in conjunction with pumping plants [Wylie at 1993].

The significant advantage of using a one-way tank rather than a surge tank is that the check valve allows the one-way tank to have a much lower height [Walski etc at 2003].

B.Pressure Relief and Other Regulating Valves

A pressure relief valve is a self-operating valve that is installed in a system to protect it from over pressurization of the system. It is designed to open (let off steam) when safe pressures are exceeded, then closes again when pressure drops to a preset level. Relief valves are designed to continuously regulate fluid flow, and to keep pressure from exceeding a preset value.

An anticipator relief valve can be used instead of a pressure relief valve to control high pressure transient peaks. It is essential for protecting pumps, pumping equipment and all applicable pipelines from dangerous pressure surges caused by rapid changes of flow velocity within a pipeline, due to abrupt pump stop caused by power failure. Power failure to a pump will usually result in a down surge in pressure, followed by an up surge in pressure. The surge control valve opens on the initial low pressure wave, diverting the returning high pressure wave from the system. In effect, the valve has anticipated the returning high pressure wave and is open to dissipate the damage causing surge. The valve will then close slowly without generating any further pressure surges [M&M Control Service].

Air inlet valves are installed at highpoints along the pipeline system to prevent vacuum conditions and potential column separation. Air enters the pipeline system during low pressure transient, and this air should be expelled slowly to avoid creating another transient condition. Before restarting the pumps, an adequate time should be allowed for the air that entered the pipeline to be expelled. There are varieties of valves that allow air to enter and leave a system, and their names depend on the manufacturer. These valves include air inlet valves, air release valves, vacuum relief valves, air vacuum valves, and vacuum breaker valves [Walski at 2003]..

C.Booster Pump Bypass

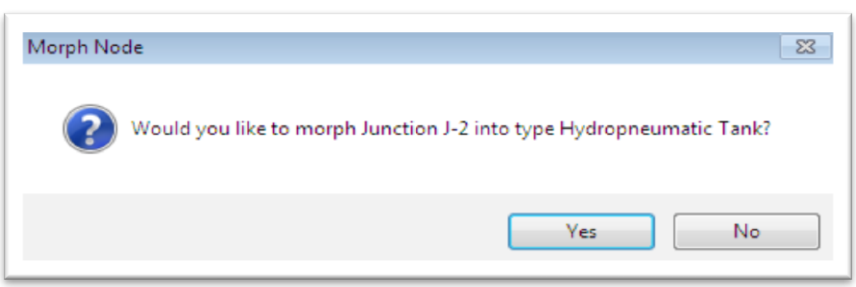
Pump bypass with a valve is another protective device against pressure transients. Two pressure waves are generated as a result of reduction in flow due to booster pump stop; the wave travelling upstream is a positive transient, and the wave that travels downstream is a negative transient. A check valve in a bypass line allows free flow to the pipeline to prevent low pressures and column separation [19]. The effectiveness of using a booster station bypass depends on the specific booster pumping system and the relative lengths of the upstream and downstream pipelines [Walski at 2003].

7.10.1 Using Hydro pneumatic tank in hammer cad

It is clear that high pressures are causing by the collapse of a vapor pocket at Node J1. You could install a Hydro pneumatic Tank at junction J1 to supply flow into The pipeline upon the power failure, keeping the upstream water column moving and Minimizing the size of the vapor pocket at the high point (or even preventing it from Forming). You can test this theory by simulating the system again using Bentley HAMMER and comparing the results with those of the unprotected run:

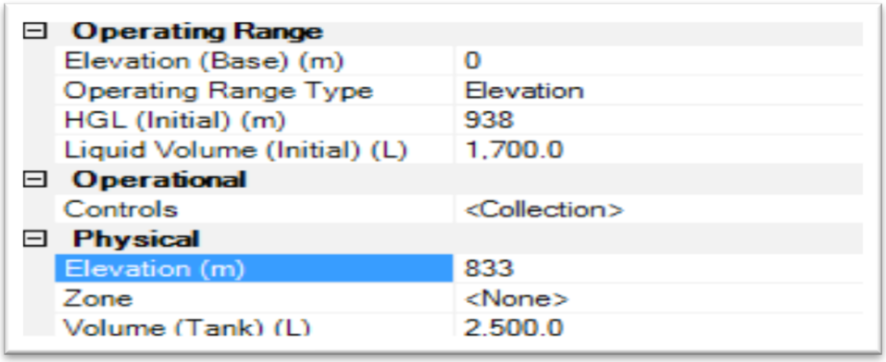
1) Click the Hydro pneumatic Tank button  on the Layout toolbar.

2) Click on J2. A prompt will appear, asking if you'd like to morph J2 into a Hydro pneumatic Tank element. Click Yes



3) Set the Hydro pneumatic Tank element properties in the Properties editor:

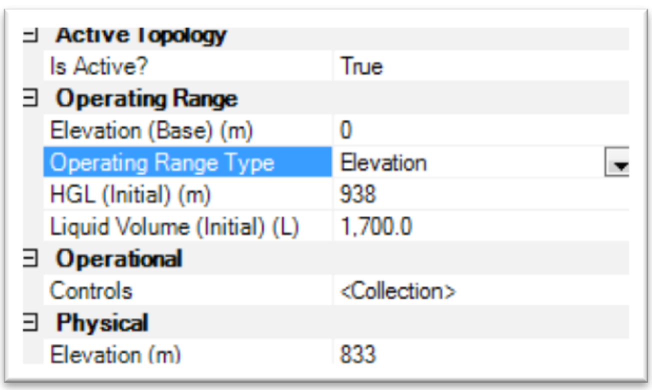
a) Make sure the Elevation (Base) and the Elevation are setting to 832.5 m.



Operating Range	
Elevation (Base) (m)	0
Operating Range Type	Elevation
HGL (Initial) (m)	938
Liquid Volume (Initial) (L)	1,700.0
Operational	
Controls	<Collection>
Physical	
Elevation (m)	833
Zone	<None>
Volume (Tank) (L)	2,500.0

Figure 7.22: Hydro pneumatic tank base elevation.

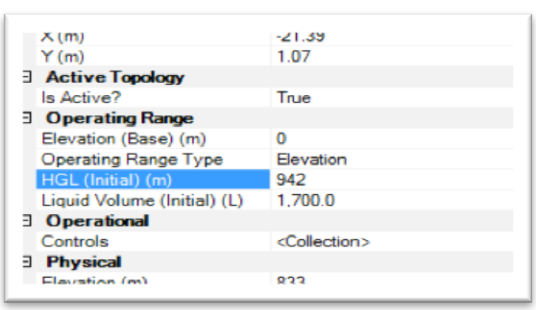
b) Set the Operating Range.



Active Topology	
Is Active?	True
Operating Range	
Elevation (Base) (m)	0
Operating Range Type	Elevation
HGL (Initial) (m)	938
Liquid Volume (Initial) (L)	1,700.0
Operational	
Controls	<Collection>
Physical	
Elevation (m)	833

Figure 7.23: Hydro pneumatic tank operating range type.

C) Set the HGL (Initial) to 942 m.



X (m)	-21.39
Y (m)	1.07
Active Topology	
Is Active?	True
Operating Range	
Elevation (Base) (m)	0
Operating Range Type	Elevation
HGL (Initial) (m)	942
Liquid Volume (Initial) (L)	1,700.0
Operational	
Controls	<Collection>
Physical	
Elevation (m)	833

Figure 7.24: Hydro pneumatic tank initial HG

d) Set the Liquid Volume (Initial) to 1700 L.

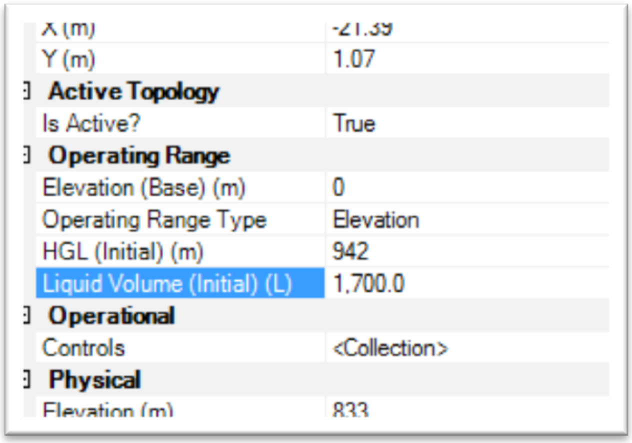


Figure 7.25: Hydro pneumatic tank initial liquid volume.

e) Set the Minor Loss Coefficient (Outflow) to 1.0.

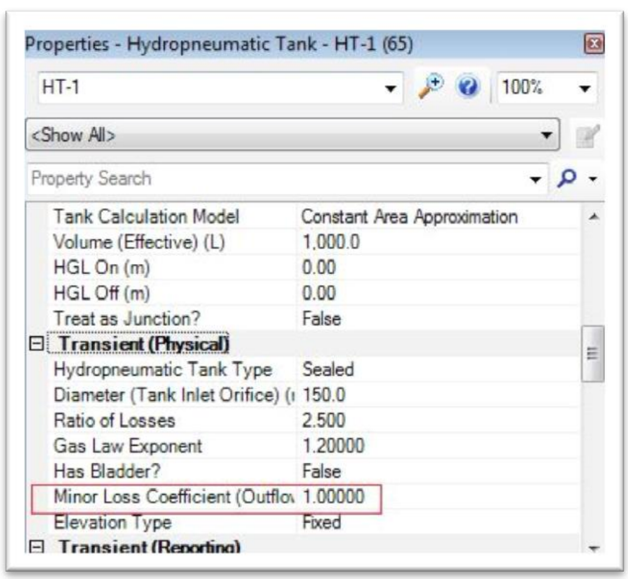


Figure 7.26: Hydro pneumatic tank minor loss coefficient.

f) Set the Tank Calculation Model to Gas Law Model.

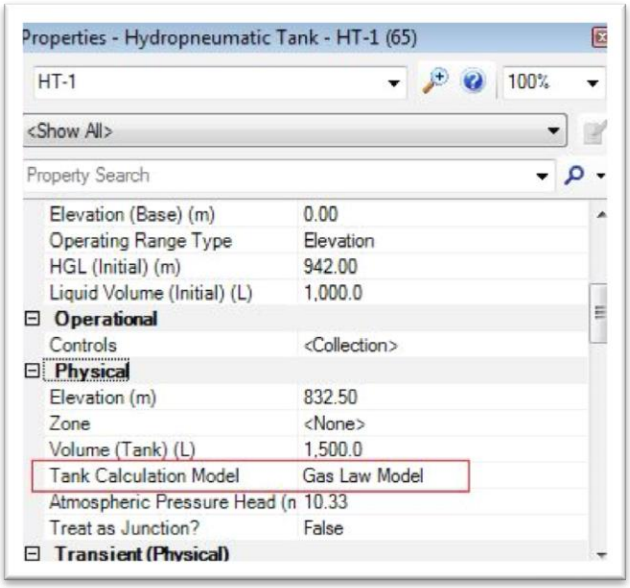


Figure 7.27: Hydro pneumatic tank gas law model.

g) Set the Volume (Tank) to 1500 L.

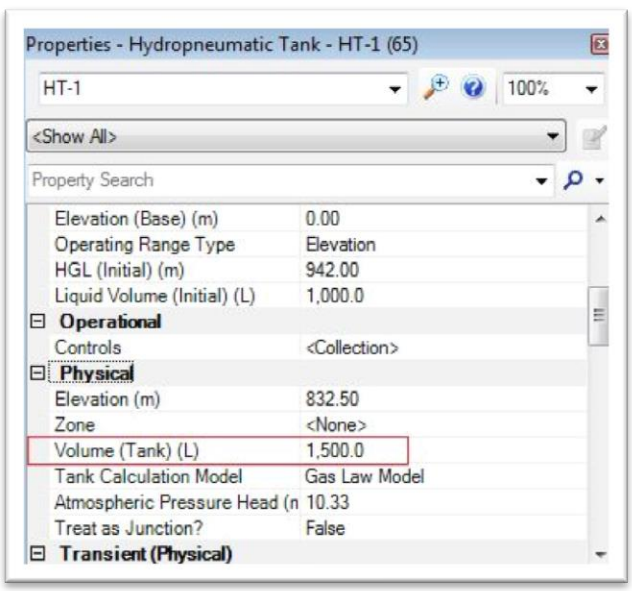


Figure 7.28: Hydro pneumatic tank volume.

h) Set the -Treat as Junction?- Field to true. This means that the hydro pneumatic tank is not included in the calculations of initial conditions. Instead, the HGL in the Hydro pneumatic tank is assumed the same as if there was a junction at the tank location.

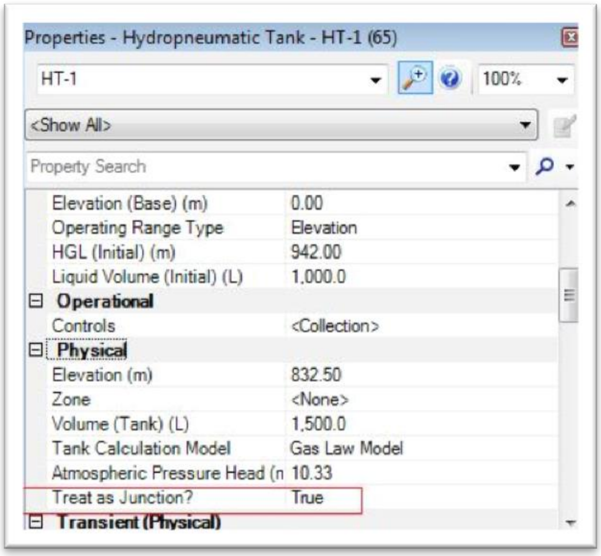


Figure 7.29: Treat hydro pneumatic tank as junction.

i) Set the Diameter (Tank Inlet Orifice) to 150 mm.

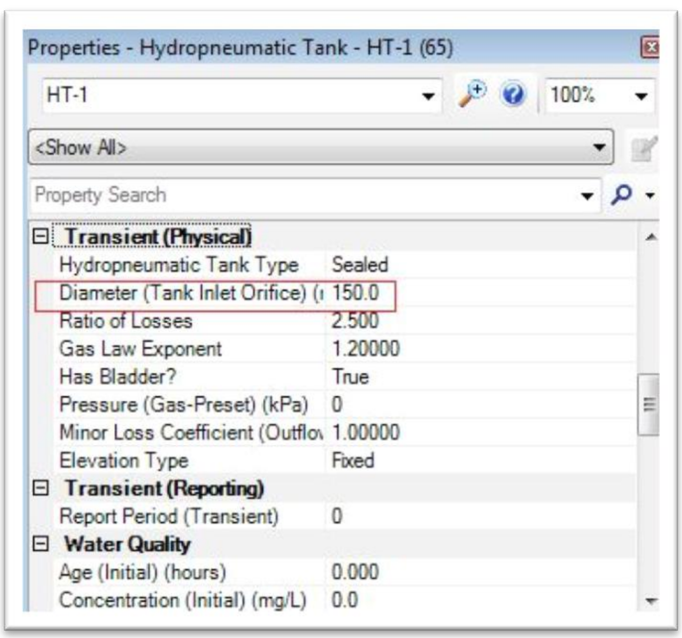


Figure 7.30: The diameter of the tank's inlet.

j) Set the Ratio of Losses to 2.5.

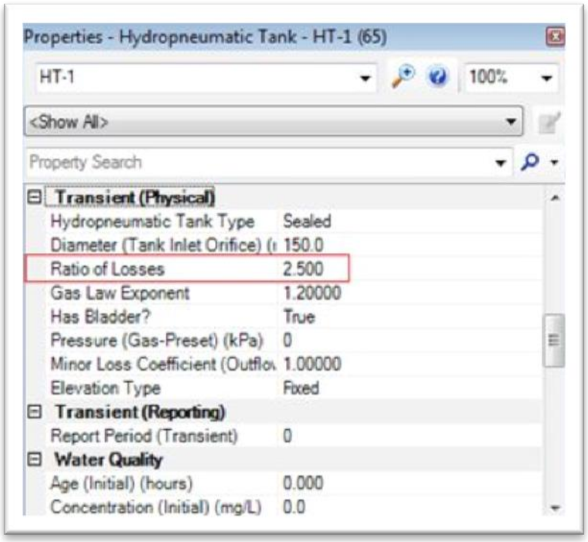


Figure 7.31: Ratio of losses.

k) Set the Gas Law Exponent to 1.2.

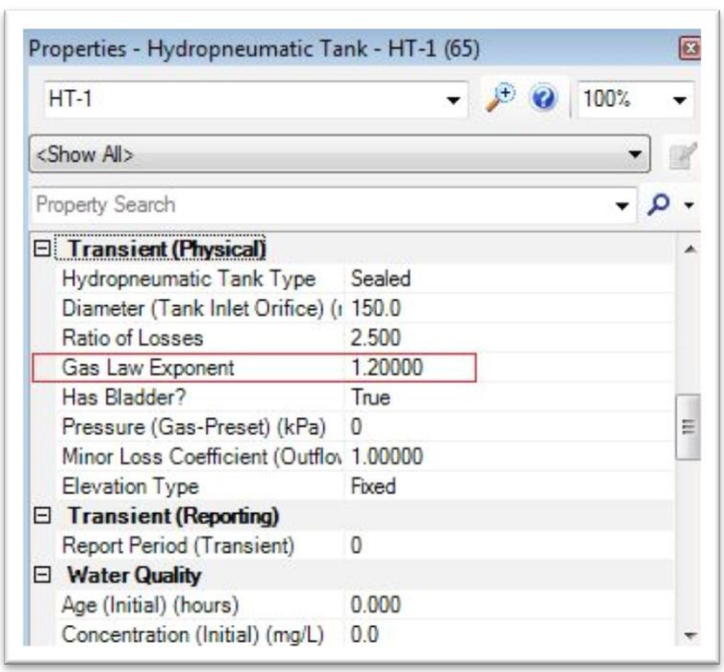


Figure 7.32: The gas law exponent.

7.10.2 Analysis with Surge-Protection Equipment

➢ Case 1:(first scenario) : using one Hydro pneumatic Tank:

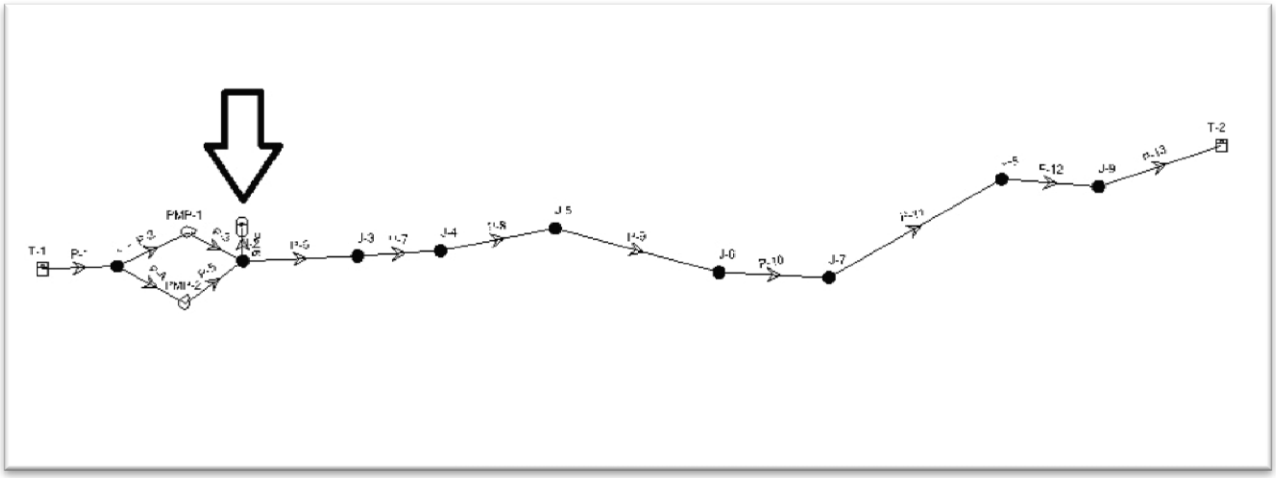


Figure 7.33: System description with hydro pneumatic tank.

Simulation Results:

1) Pipe results:

Table7.8: Pipe results with hydro pneumatic tank.

	Label	Length (User Defined) (m)	Diameter (mm)	Material	Darcy-Weisbach e (m)	Wave Speed (m/s)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
43: P-1	P-1	3.00	200.0	Ductile Iron	0.00030	1,214.04	846.26	825.73
44: P-2	P-2	3.00	100.0	Ductile Iron	0.00030	1,214.04	857.08	822.50
45: P-3	P-3	3.00	100.0	Ductile Iron	0.00030	1,214.04	938.51	871.69
46: P-4	P-4	3.00	100.0	Ductile Iron	0.00030	1,214.04	857.08	822.50
47: P-5	P-5	3.00	100.0	Ductile Iron	0.00030	1,214.04	938.51	871.70
48: P-6	P-6	206.00	200.0	Ductile Iron	0.00030	1,214.04	937.94	869.01
49: P-7	P-7	160.00	200.0	Ductile Iron	0.00030	1,214.04	934.22	869.20
50: P-8	P-8	360.00	200.0	Ductile Iron	0.00030	1,214.04	931.32	870.37
51: P-9	P-9	520.00	200.0	Ductile Iron	0.00030	1,214.04	924.82	873.18
52: P-10	P-10	275.00	200.0	Ductile Iron	0.00030	1,214.04	920.32	876.66
53: P-11	P-11	465.00	200.0	Ductile Iron	0.00030	1,214.04	918.93	875.10
54: P-12	P-12	320.00	200.0	Ductile Iron	0.00030	1,214.04	916.90	873.54
55: P-13	P-13	180.00	200.0	Ductile Iron	0.00030	1,214.04	915.63	877.36
63: P-16	P-16	0.50	100.0	Ductile Iron	0.00030	1,214.04	937.99	872.53

2) Pump results:

Table7.9: Pump results with hydro pneumatic tank.

	Label	Elevation (m)	Flow (Total) (m^3/h)	Pump Head (m)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
31: PMP-1	PMP-1	832.50	106	102.65	938.51	822.50
32: PMP-2	PMP-2	832.50	106	102.65	938.51	822.50

3) Tank results:

Table7.10: Tank results with hydro pneumatic tank.

	Label	Hydraulic Grade (m)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
41: T-1	T-1	836.50	836.50	836.50
42: T-2	T-2	893.00	893.01	893.00

4) Junction results:

Table7.11: Junction results with hydro pneumatic tank.

	ID	Label	Elevation (m)	Hydraulic Grade (m)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
30: J-1	30	J-1	832.50	836.44	846.26	825.73
33: J-2	33	J-2	832.50	937.94	937.94	872.53
34: J-3	34	J-3	845.64	934.22	934.22	869.20
35: J-4	35	J-4	846.15	931.32	931.32	870.37
36: J-5	36	J-5	867.53	924.82	924.82	873.18
37: J-6	37	J-6	816.55	915.42	920.32	876.66
38: J-7	38	J-7	815.43	910.44	918.93	876.93
39: J-8	39	J-8	880.30	902.04	916.90	875.10
40: J-9	40	J-9	876.40	896.25	915.63	877.36

- **Graphical result representation (for case one).**

1) The following figures shows the flow variation at the surge tank location from the result obtained its clear the maximum flow 80 m^3\h and the minimum flow -196 m^3\h.

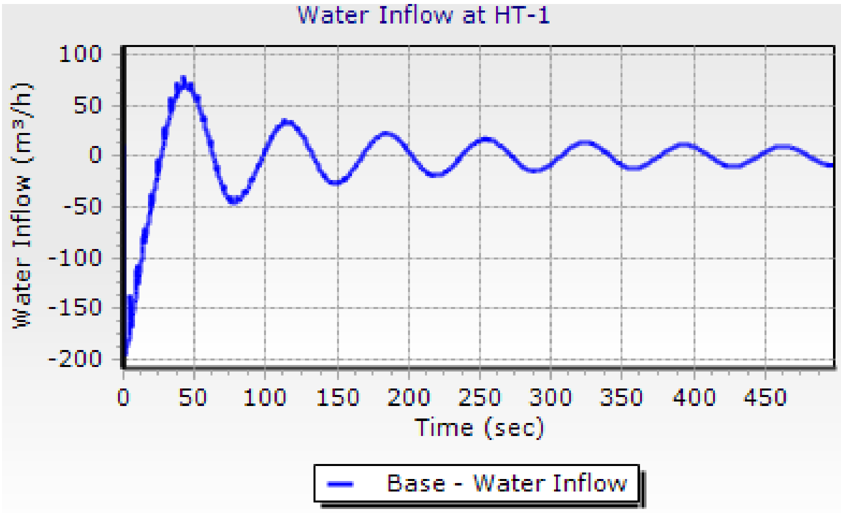


Figure 7.34: Water flow varation at the surge tank location.

2) The following figures show the volume variation at the surge tank location from the result obtained its clear the maximum volume 1500 L and the minimum volume 800L**.**

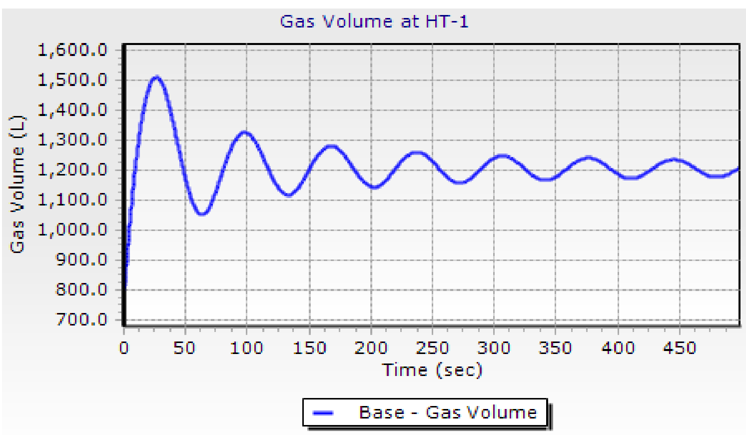


Figure 7.35: Gas volume varation at the surge tank location.

3) The following figures show the pressure variation at the surge tank location from the result obtained its clear the maximum pressure 1136 kpa and the minimum pressure530 kpa.

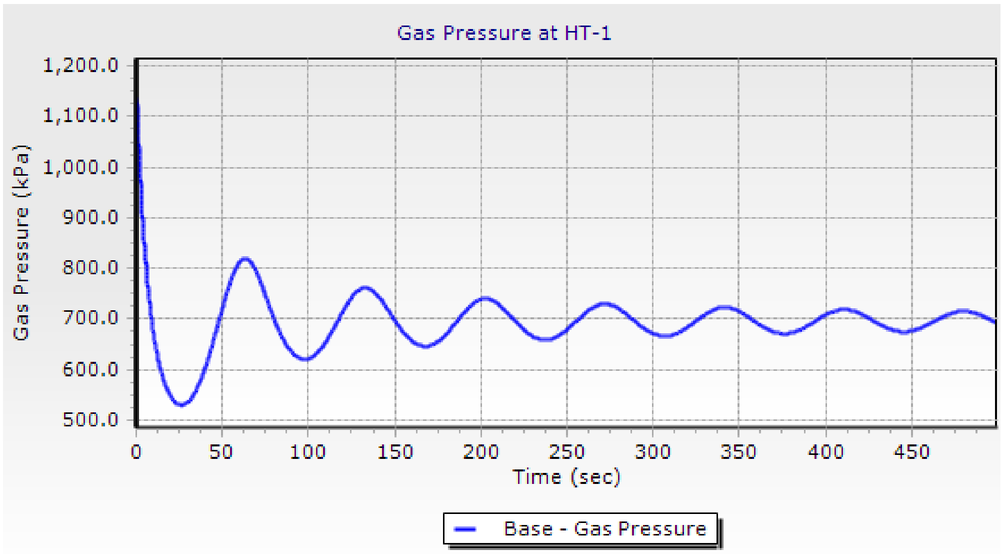


Figure 7.36: Gas pressure varation at the surge tank location.

4) Hydraulic grade:

The following figure shows the hydraulic grade lines along the entire pipe length:

Figure 7.37: Hydraulic grade lines along the entire pipe length

According the previos figure we get proplem between station 2000 m and 2250 m, therefor another Hydro pneumatic tank must be used to solve this problem.

➢ Case 2(second scenario) : using two Hydro pneumatic Tank in the start of the system:

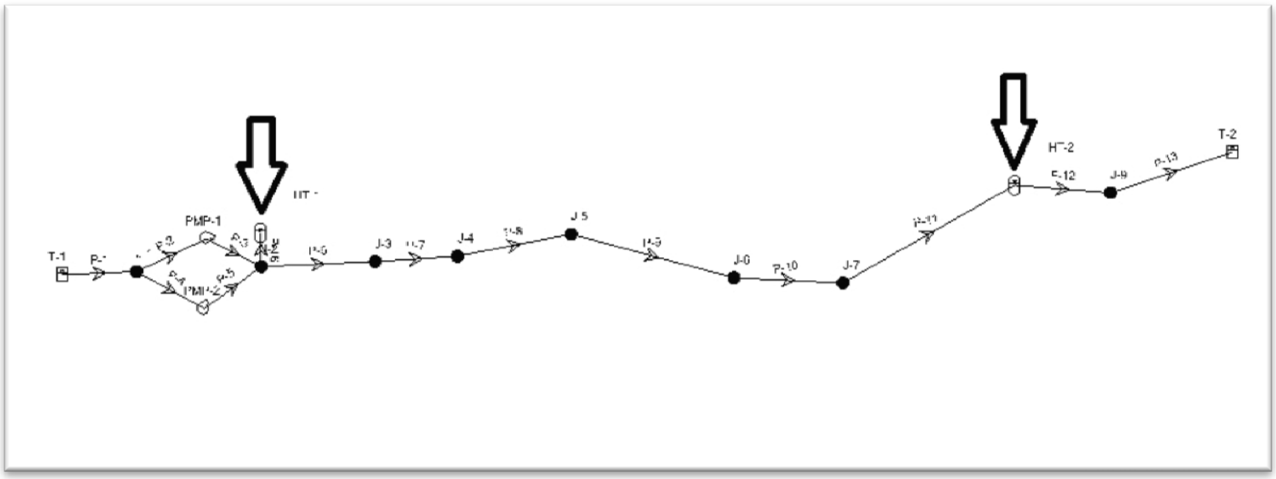


Figure 7.38: System description with 2 hydro pneumatic tanks.

Simulation Results:

1) Pipe results:

Table7.11: Pipe results with hydro pneumatic tank.

	Label	Length (User Defined) (m)	Diameter (mm)	Material	Darcy-Weisbach e (m)	Wave Speed (m/s)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
43: P-1	P-1	3.0	200.0	Ductile Iron	0.00030	1,113.12	845	823
44: P-2	P-2	3.0	100.0	Ductile Iron	0.00030	1,113.12	861	822
45: P-3	P-3	3.0	100.0	Ductile Iron	0.00030	1,113.12	939	877
46: P-4	P-4	3.0	100.0	Ductile Iron	0.00030	1,113.12	861	822
47: P-5	P-5	3.0	100.0	Ductile Iron	0.00030	1,113.12	939	877
48: P-6	P-6	206.0	200.0	Ductile Iron	0.00030	1,113.12	938	877
49: P-7	P-7	160.0	200.0	Ductile Iron	0.00030	1,113.12	934	878
50: P-8	P-8	360.0	200.0	Ductile Iron	0.00030	1,113.12	931	877
51: P-9	P-9	520.0	200.0	Ductile Iron	0.00030	1,113.12	925	880
52: P-10	P-10	275.0	200.0	Ductile Iron	0.00030	1,113.12	915	882
53: P-11	P-11	465.0	200.0	Ductile Iron	0.00030	1,113.12	914	883
54: P-12	P-12	320.0	200.0	Ductile Iron	0.00030	1,113.12	911	886
55: P-13	P-13	180.0	200.0	Ductile Iron	0.00030	1,113.12	907	886
63: P-16	P-16	0.5	150.0	Ductile Iron	0.00030	1,113.00	938	878
66: P-17	P-17	0.5	150.0	Ductile Iron	0.00030	1,113.00	911	887

2) Pump results:

Table7.12: Pump results with hydro pneumatic tank.

	Label	Elevation (m)	Flow (Total) (m³/h)	Pump Head (m)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
31: PMP-1	PMP-1	833	106	102.65	939	822
32: PMP-2	PMP-2	833	106	102.65	939	822

3) Tank results:

Table7.13: Tank results with hydro pneumatic tank.

	Label	Hydraulic Grade (m)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
41: T-1	T-1	837	837	837
42: T-2	T-2	893	893	893

- **Junction results:**

Table7.14: Junction results with hydro pneumatic tank.

	ID	Label	Elevation (m)	Hydraulic Grade (m)	Head (Maximum, Transient) (m)	Head (Minimum, Transient) (m)
30: J-1	30	J-1	833	836	845	823
33: J-2	33	J-2	833	938	938	878
34: J-3	34	J-3	846	934	934	878
35: J-4	35	J-4	846	931	931	878
36: J-5	36	J-5	868	925	925	880
37: J-6	37	J-6	817	915	915	882
38: J-7	38	J-7	815	910	914	883
39: J-8	39	J-8	880	902	911	887
40: J-9	40	J-9	876	896	907	887

- **Graphical result representation (for case two).**

- The following graphs for first tank (HT-1) by using two protected device:

1) The following figures shows the flow variation at the surge tank location from the result obtained its clear the maximum flow 50 m^3\h and the minimum flow -200 m^3\h**.**

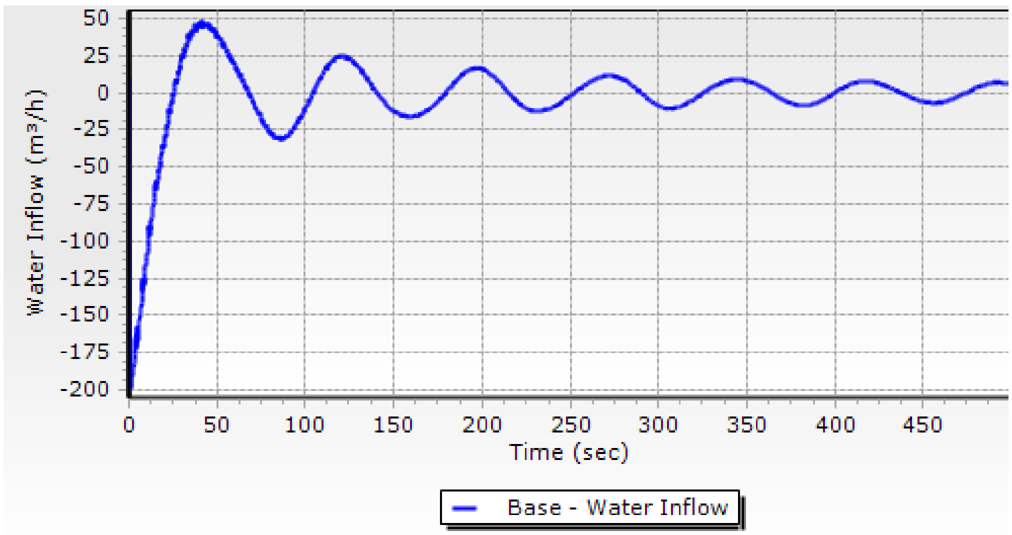


Figure 7.39: Water flow varation at the surge tank location.

2)The following figures show the volume variation at the surge tank location from the result obtained its clear the maximum volume 1440 L and the minimum volume 800L**.**

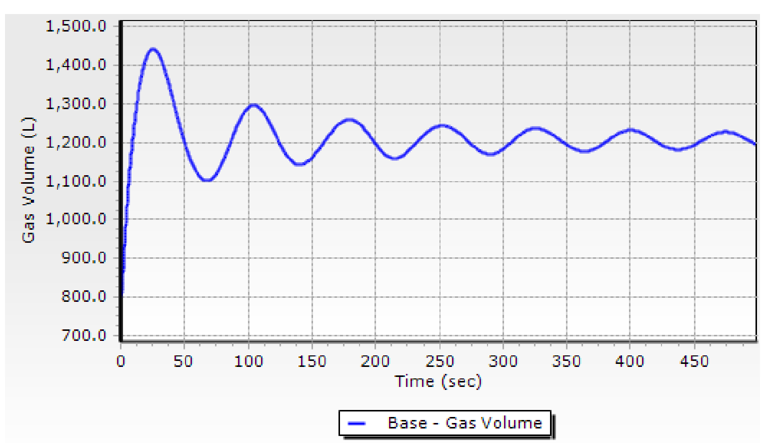


Figure 7.40: Gas volume varation at the surge tank location.

3) The following figures show the pressure variation at the surge tank location from the result obtained its clear the maximum pressure 1136 kpa and the minimum pressure560 kpa.

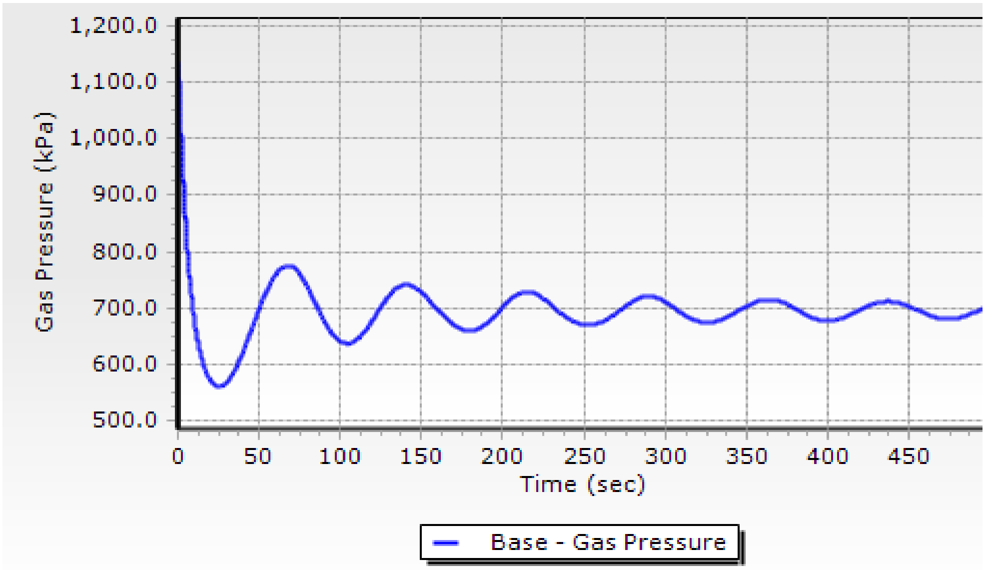


Figure 7.41: Gas pressure varation at the surge tank location.

4) Hydraulic grade:

The following figure shows the hydraulic grade lines along the entire pipe length.

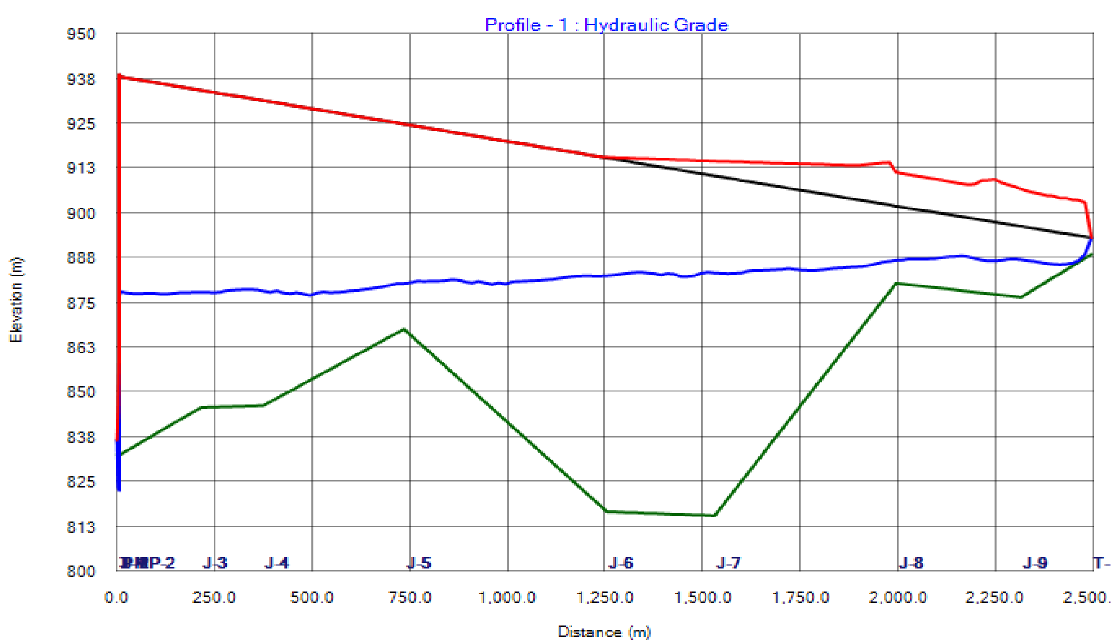


Figure 7.42: Hydraulic grade lines along the entire pipe length.

According the previos figure the cavitation proplem between station 2000 m and 2250 m was repiared after using Hydro pneumatic tank.

- Graphical result representation for second tank (HT-2) by using two protected device:

1) The following figures shows the flow variation at the surge tank location from the result obtained its clear the maximum flow 60 m^3\h and the minimum flow -80m^3\h.

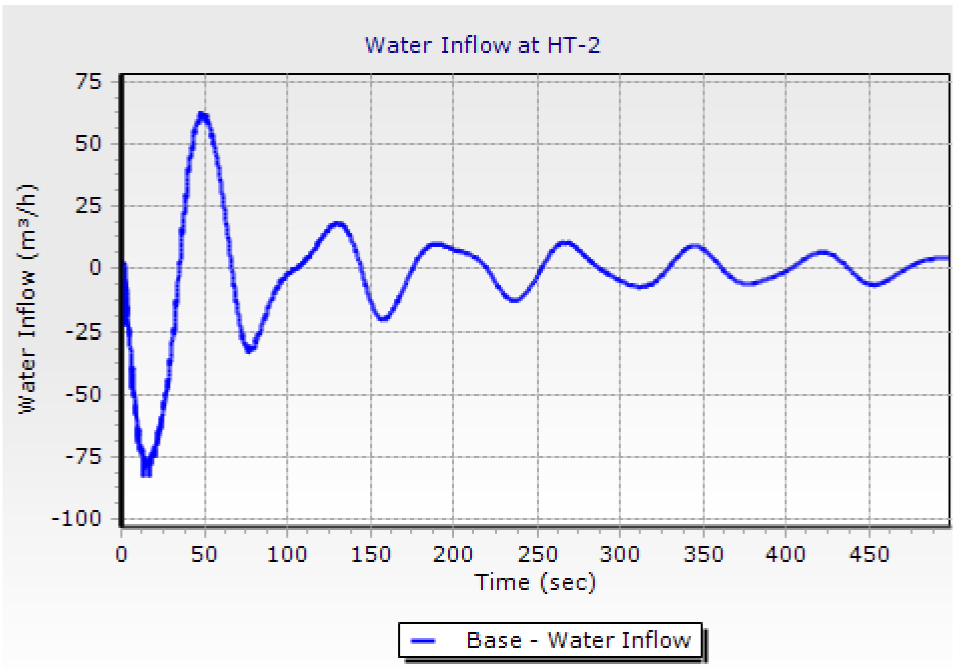


Figure 7.43: Water inflow varation at the surge tank location.

2)The following figures show the volume variation at the surge tank location from the result obtained its clear the maximum volume 1218 L and the minimum volume 750L.

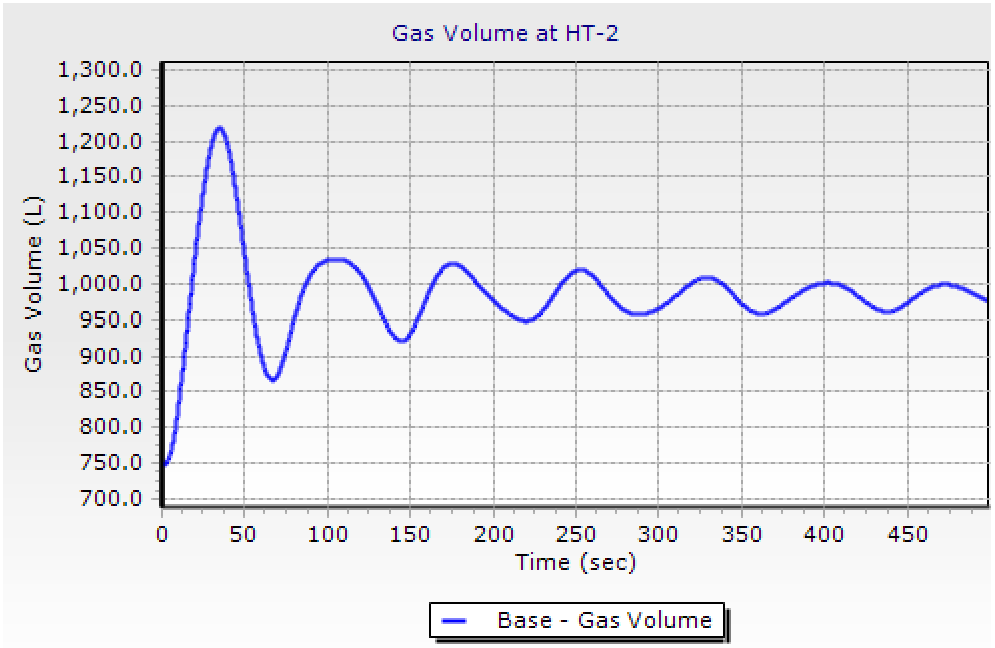


Figure 7.44: Gas volume varation at the surge tank location.

3) The following figures show the pressure variation at the surge tank location from the result obtained its clear the maximum pressure 1136 kpa and the minimum pressure560 kpa.

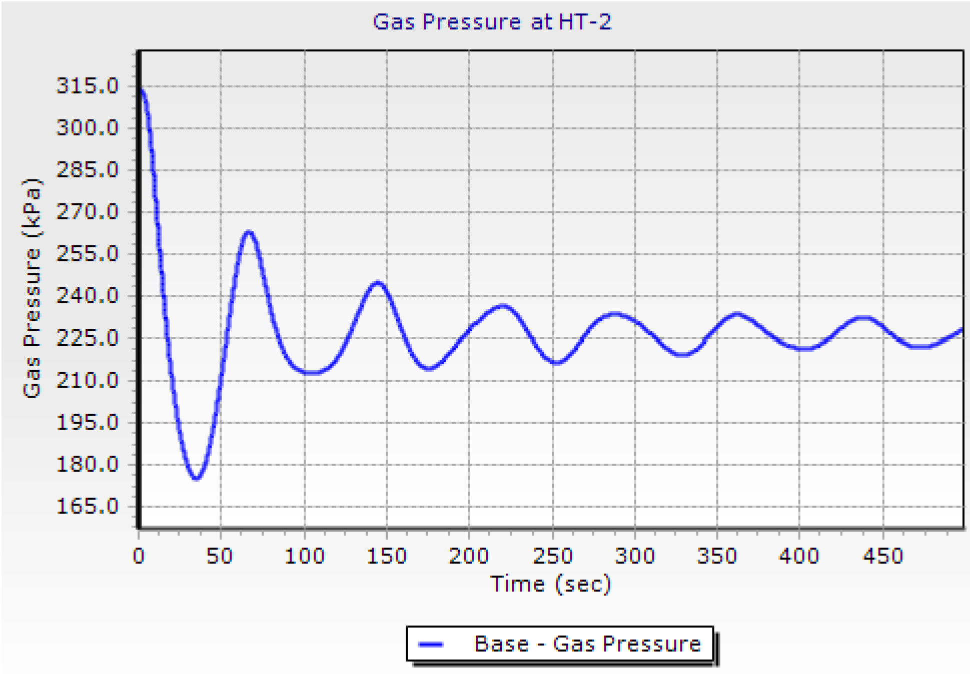


Figure 7.45: Gas pressure varation at the surge tank location.

4) hydraulic grade:

The following figure shows the hydraulic grade lines along the entire pipe length

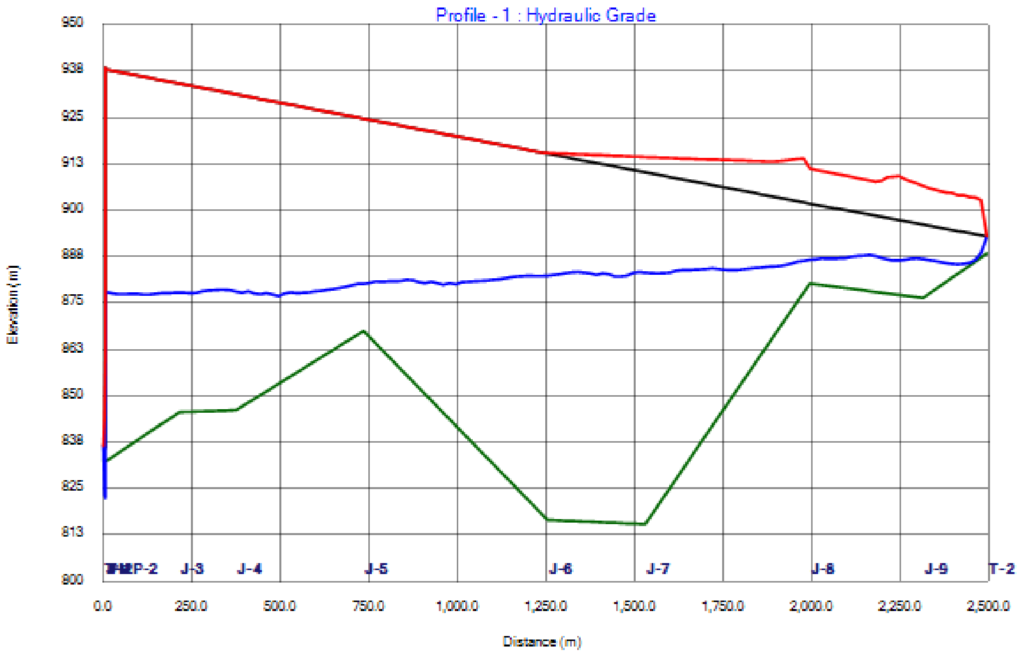


Figure 7.46: Hydraulic grade lines along the entire pipe length.

From the result obtained we noticed that using two surge tanks is sufficient to protect the pipeline.

CHAPTER “8”

“CONCLUSION”

8.1 Conclusion

In this project, the trial is made to create hydraulic design for a pumping station combined with conveying pipeline system using computer model for Beit Jala town, considering the street where the proposed pipeline will pass and its elevations, topographical features, accelerated development and growth of the town.

By applying several hydraulic principles on the current case study the result brought out many important conclusions, the main conclusions draw from the present study are summarized below:

1) The water supply in Beit Jala pumped to the water network system by pumping stations directly which cause various problems as energy consuming and in water flow such as water hammer, since the pumping station will not work in its real efficiency and the required duty point at the network level due to the variation of water consumption.

2) The pipe diameter calculated using population geometric increasing method for the next 30 years; we find it equal to **200** mm, we used this method due to availability of expanding in population domain in this region.

3) The pipes are connected to each other by joint in standard form, but the pipe are not necessary to be straight, it follows the train and the topography so, a deflection angle must be exist and should not exceed a threshold values

4) Valves are used to control the amount of flow and may be globe valves, angle valves, gate valves, and butterfly valves, any of several types of check valves.

5) The path of the pipeline is from AL-Mattalah pumping station which has the lowest elevation to Beit Jala Reservoir, which has the highest elevation.

6) The material that should be used in the water network tubes depending on the water pressure and tubes radius. So ductile iron is the most suitable material for pipeline, In addition to the

intrinsic qualities of the basic metal, the variety of shapes and dimension of components facilitate pipeline assembly.

7) Al matallah pumping station is currently equipped with pumping units 99 m^3/h at 50 m WC.

8) The hydraulic efficiency not less than 75% and the design pressure 16 pars.

9) It is clear that high pressures are causing by the collapse of a vapor pocket in the system, You could install a Hydro pneumatic Tank in the system to supply flow into The pipeline upon the power failure, keeping the upstream water column moving and Minimizing the size of the vapor pocket at the high point (or even preventing it from Forming).

10)The programs we used in this design to achieve our goals are GIS, Civil 3D, HAMMER CAD.

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Appendix

1. **APPENDIX A**