

DEWATERING PLAN AND PREDICTION FOR PIT LAKE
FLOODING FOR A QUARRY SITE

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ABSTRACT

DEWATERING PLAN AND PREDICTION FOR PIT LAKE FLOODING FOR A QUARRY SITE

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This study presents the dewatering assessment of a marl quarry with the future pit lake level predictions. The objectives of the study were; (1) to determine the dewatering requirements that would allow for the continuation of the quarrying operations in the deeper parts of the quarry, (2) to design an optimum dewatering system compatible with the site hydrogeological conditions and quarrying plans, (3) to assess the environmental impacts of dewatering on the local water (surface and ground water) resources and users, and (4) to predict the future pit lake level and flooding period for different meteorological conditions.

To these ends, previous investigation reports and maps have been compiled and reviewed and field investigations have been conducted. During the field investigations pumping and observation wells were drilled and installed.

After installation, in situ tests were conducted to determine aquifer parameters. It was found that properties of the material is conducive to the dewatering activities that will be necessary for the deepening of the open pit of the marl quarry.

A groundwater model was developed based on the field data gathered. According to this model dewatering trenches will be needed to dewater the pit. The model predicted that operating these dewatering trenches would create an elongated cone of depression that will sufficiently lower the groundwater table so that quarrying operations can proceed.

Lowering of the water table may produce a negative impact on groundwater resources within the aerial extent of the cone of depression. This potentially negative impact was investigated with model simulations and has been found that the impact to the resources would be negligible.

Three scenarios were evaluated as possible dewatering discharge disposal solutions. The preferred scenario included the discharge of this water to the stream, which is flowing on the western side of the quarry.

The pit will start to fill with water immediately after the dewatering operations stopped. In order to predict the pit lake flooding period and final lake elevation, pit lake hydrologic model was developed. The simulations predict that the final pit lake elevation would be at 991 m. The pit lake will rise to this level at approximately 72 years after closure.

Keywords: Dewatering, Trench System, Pit Lake Flooding, Modelling, Simulation, Marl Quarry.

ÖZ

OCAK SAHASI SUSUZLAŞTIRMA PLANI VE OCAK GÖLÜ OLUŞUMU TAHMİNİ

DURU, Uygur

**Yüksek Lisans, Jeoloji Mühendisliği Bölümü
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Bu çalışma bir marn ocağında yapılacak olan susuzlaştırmanın değerlendirilmesi ve oluşacak ocak gölü seviyesinin tahmini konularını kapsar. Çalışmanın amacı; (1) Mevcut açık ocağın derinleştirilmesine ve derinde bulunan marn tabakalarına ulaşım imkanı tanıyacak susuzlaştırma gereksinimlerinin belirlenmesi, (2) hidrojeolojik saha koşulları ve maden planları ile uyumlu en uygun susuzlaştırma sisteminin tasarlanması, (3) susuzlaştırmanın su (yüzey ve yeraltıları) kaynakları ve kullanıcıları üzerindeki çevresel etkilerinin değerlendirilmesi, ve (4) çeşitli meteorolojik koşullar altında ocak gölünün dolum zamanı ve göl seviyesi yüksekliği tahminlerinden oluşmaktadır.

Bu amaçlar doğrultusunda, önceki araştırma raporları ve haritaları derlenerek incelenmiş ve arazi çalışmaları yürütülmüştür. Arazi çalışmaları süresince, pompa ve gözlem kuyuları açılarak teçhiz edilmiş, akifer

parametrelerini belirlemek amacıyla saha deneyleri yapılmıştır. Bu deneyler sonucunda, ocak içerisindeki malzemenin hidrolik parametrelerinin, ocağının derinleştirilmesi için gerekli olan susuzlaştırma faaliyetlerine uygun nitelikte olduğu anlaşılmıştır.

Elde edilen verilere dayanılarak, sahanın yeraltısuyu modeli geliştirilmiş, ve model sonuçlarına göre, ocağın susuzlaştırılması için drenaj kanallarının yeterli olacağı anlaşılmıştır. Bu kanallar ile drene edilecek su, yeraltısı tablasında elipsel yayılımda bir düşüm konisi meydana getirerek yeraltısuyu seviyelerinin yeterince düşmesini ve ocak içi faaliyetlerin devam etmesini sağlayacaktır.

Su tablasının düşürülmesi, düşüm konisinin alansal yayılımı içinde kalan mevcut su kaynaklarında olumsuz bir etki oluşturabilir. Bu olası olumsuz etki yeraltısuyu modelinde incelenmiş ve kaynaklara olan etkinin gözardı edilebilecek kadar düşük olduğu anlaşılmıştır.

Susuzlaştırma sonucunda elde edilecek suyun deşarjı için üç senaryo değerlendirilmiştir. Uygun görülen senaryo, susuzlaştırma ile elde edilecek fazla suyun, ocağın batısında bulunan çaya boşaltılmasıdır.

Ocaktaki faaliyetler sona erdikten ve susuzlaştırma sistemi durdurulduktan sonra ocak gölü oluşacaktır. Ortalama meteorolojik koşullar varsayılarak yapılan modellemeler sonucunda göldeki su seviyesinin 991metreye ocak kapandıktan 72 yıl sonra ulaşacağı anlaşılmıştır.

Anahtar Kelimeler: Susuzlaştırma, Kanal Sistemi, Ocak Gölü Oluşumu, Modelleme, Simulasyon, Marn Ocağı.

TO MY FAMILY

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I would like to express my thanks to my friends for their help, trust and encouragement.

This thesis is dedicated to my family and I am forever grateful to my parents, Nihal and Kamil Duru, whose foresight and values paved the way for a privileged education, and to my brother Murat; who gently offered counseling and unconditional support at each turn of the road.

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CHAPTER I

INTRODUCTION

1.1 Purpose and Scope

The main purpose of this study is to develop a dewatering plan for a marl quarry, which is operating by the AST Cement Group. The quarry has been in operation since 1983 and annually supplies approximately 600,000 tons marl to the nearby cement plant. The quarry operations are expected to continue for another 30-40 more years.

During the quarrying operations of the 1980s' a pit like on the west end of the existing quarry formed when marl was extracted below the elevation of 992.5 masl (meters above sea-level). AST is planning to mine the central part of the quarry to an elevation of 970 masl. During this study dewatering plan was designed to achieve following:

- dewater the existing pit lake in order to access the marl resources below the pit lake,
- dewater the central part of the quarry to prevent the formation of a pit lake during planned mining operations to the elevation of 970 masl,
- assess the impact of dewatering operations on existing water resources and users and develop plans to minimize such impacts if any,

This study presents the dewatering plans and an assessment of any potential impacts on current water resources and on stake holders. The scope of work for this study includes the compilation and review of existing documents and reports, and a field testing program implemented to characterize the site hydrogeological conditions. In order to characterize the surface and groundwater conditions at the quarry and its vicinity, the discharges of streams, springs, and groundwater levels were measured in. Pumping and observation wells were drilled and completed, and aquifer tests were conducted to determine the hydraulic parameters (i.e., permeability) of the marls.

In order to evaluate alternative dewatering plans and associated impacts during operations and at closure, a numerical groundwater model of the study area was constructed. The model was calibrated to the existing field conditions with respect to observed groundwater levels, existing pit lake levels, and spring discharges. Alternative dewatering plans and the prediction of the final pit lake level were simulated using the groundwater model. The environmental impacts of

the proposed dewatering plan and possible mitigative measures have been quantified with the help of the model.

Current government regulations were also considered in the development and implementation of the dewatering plan of the marl quarry. These include:

- Ambient water (inland water resources) classification limits issued by the Ministry of Environment and Forestry, Turkey in “Water Pollution Control Regulations” (MoEF, 1988)
- Aquatic Life limits and guidelines issued by the Ministry of Environment and Forestry, Turkey in “Aquatic Life Law No. 2872”,
- Irrigation water classification limits issued by the Ministry of Environment and Forestry, Turkey in “Technical Methods for Water Contamination Control Regulations” (MoEF, 1991).
- The Environmental Impact Assessment Regulation issued by Ministry of Environment and Forestry, Turkey, (2003).

1.2 Dewatering Plan Objectives

The objective of this study is to prepare a dewatering plan that will:

- enhance stability of the pit walls,
- minimize the potential of any hazard to both operational personnel and mining equipment,
- be compatible with the hydraulic characteristics of the material to be dewatered,

- have minimum installation, operation and maintenance cost,
- be compatible with the long-term (30 years) operation of the quarry,
- have minimum adverse impacts,
- satisfy the regulatory requirements for dewatering and discharging as specified by the Turkish Government.
- predict the future pit lake level after closure and filling period for different meteorological scenarios.

CHAPTER II

LITERATURE REVIEW

2.1 Dewatering

Dewatering is universally defined to be the temporary or permanent lowering of the water table. Lowering the groundwater table is a frequently encountered problem for the deep excavations below the groundwater level. Construction of commercial and industrial buildings, bridges, dams, powerhouses, as well as mining, require deep excavations below the water table.

The choice of dewatering method is dependent on the nature and permeability of the ground, extend of the dewatering area, depth of groundwater level and the amount by which it has to be lowered and proposed methods of excavation operation. A number of methods are available for controlling the inflow of water into an excavation including;

- Vertical wells drilled behind the highwall of excavation;
- Horizontal drain holes in the highwall;
- In-pit (excavation floor) drainage ditches or sumps;
- Drainage galleries in the highwall or pit bottom.

Vertical wells: Vertical wells are of high initial cost and require a high standard of design and expertise in installation to achieve maximum economy. They should be constructed to the standards of accepted good practice used for the installation of the water wells. Vertical well systems are pumped using submersible pumps of appropriate size and power, installed inside the well screens and connected to riser pipes.

Vertical wells are among the most common methods of dewatering activities. This kind of systems is installed and still dewater the excavations, open pit and underground mines. For instance, coal production and coal recovery at Whitewood open pit coal mine, Alberta, were unacceptably low as a result of poor groundwater and surface water control at the mine. After a feasibility study, which was conducted to determine the most cost effective method to reduce the groundwater inflows to the mine pit and reduce pore water pressures in the mine walls, it has been decided to install a vertical dewatering system just behind the mine wall. The acceptability of this system was also simulated with a finite difference computer model (Sumer et al., 1988).

Horizontal drain holes: Horizontal drain holes/ wells also have high initial cost and require expertise in installation to achieve maximum efficiency. These kinds of systems can be accepted as a passive dewatering system, which is common for unconfined aquifers where groundwater inflow comes mainly from storage as opposed to regional groundwater flow. They are mainly used for dewatering and depressurizing of highwalls of the open pits and underground galleries to dewater the orebody. Horizontal drain holes can also be used to monitor groundwater contamination.

At New York Brookhaven National Laboratory two horizontal wells were installed to access ground water containing radioactive tritium beneath the High Flux Beam Reactor (HFBR). The objective of the installation was to identify potential source areas, including the spent fuel canal and a potential upgradient source known as S3. The wells were designed for monitoring ground water quality, taking into account seasonal fluctuations. However, they could also potentially be used in the future for ground water extraction. The wells were placed both up- and down-gradient of the spent fuel canal, which is suspected to be the main source of the tritium leak (Carelly, Lowe and Pressly, 1998).

In-pit (excavation floor) drainage ditches or sumps: Ditch/sump pumping is the simplest method of dewatering for pit floors and excavations. Its effectiveness is dependent on the hydraulic characteristics of the material and the extension/number of the ditches. Sumps are usually sited at the corners of excavations, below the general excavation level. A pump is provided for each

sump and connected to a discharge pipe. In some cases the hydraulic parameters of the excavated material is not suitable for sump system to lower the water level in extensive areas. In these cases acceptable application is to excavate ditches and connect these ditches to a main sump. In open cuts the most important aspect that should be taken into consideration is the slope stability of the pit walls. Where the slope is too steep or the hydraulic head too large, seepage may occur on the slope and may cause slope failure. The solution is usually to have a shallower slope. In many of the excavations and open pit floors ditches (dewatering trenches) are used widely. For example Galloway and Foster in 1982, during the first stages of a coal mine in Illinois, were used sump drains and ditches to remove the water from the mine site for the Office of Surface Mining. As the open pit advanced sump drains and ditches were also used to dewater the ore body and pit floor. Another application was installed by Penn State University Geology Department to dewater an archeological site in Egypt near the Nile River. Groundwater level was first lowered with the sump system and later additional ditches were used to dewater the whole site (Glesson, 2002).

Drainage galleries in the highwall or pit bottom: Constructing drainage galleries is an expensive but effective method of dewatering. These underground workings are mainly constructed in the pit highwalls or pit bottoms to remove water from the system, before it enters to the pit.

Lafarge, in one of its limestone quarries near Berlin, uses drainage galleries to dewater the site. The greater part of the quarry water is collected by

an 11 km underground system of galleries specially constructed for drainage purposes and is conveyed to a pump sump. From this sump water is discharged into the Kriensee Lake with an automatically controlled pumping station (http://www.readymix-zement.de/pdf/Rohst_e.pdf). Figure 2.1 shows the details of drainage gallery system.

Computer models are very suitable to simulate the groundwater's behaviour within an aquifer. The most effective and efficient dewatering system can be determined by the use of combined simulation and optimization methodology. Tokgöz et. al. (2002) used such a methodology to devise an optimum dewatering plan for the construction of a collector line in Aksaray, Turkey.

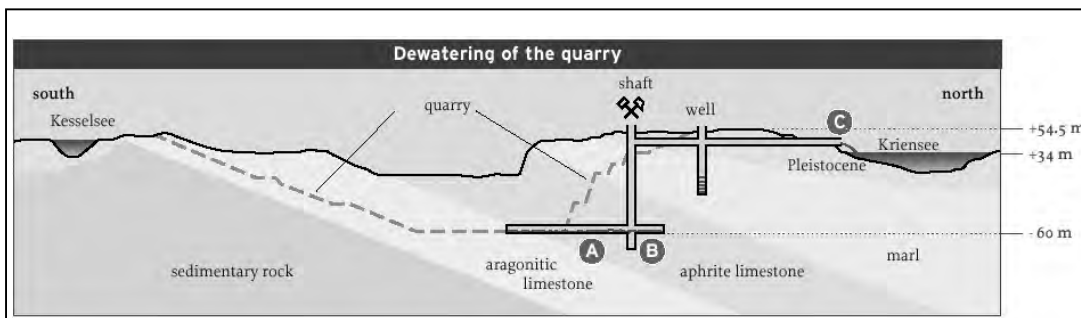


Figure 2.1. Details of drainage gallery system.

2.2 Pit Lakes

Pit lakes begin its existence as an open pit that was dewatered during active mining. Once mining ceases and dewatering pumps are removed, groundwater begins to seep into the pit. Surface water also collects in the pit if it is not diverted. Old stone quarries are the most abundant pit lakes. Coal, uranium and hard rock (e.g., copper, gold, etc.) mining produced open pits which has formed pit lakes after closure. Pit lakes can be classified into two groups as terminal and flow-through. For terminal pit lakes outflow rate is greater than inflow and they maintain a depression in the water table. Flow-through pit lakes are the result of high inflow rates. In the case of flow-through pit lakes, lake water outflows from the pit lake. Furthermore lake water can outflow as surface water from the pit lake. A terminal pit lake poses potential water quality impacts to the lake water itself but not to regional groundwater. A flow-through lake (that is where groundwater flows into and out of the lake) may cause a potential impact to both the pit lake water and down-gradient groundwater (Jonas, from Reno Presentation, 2003).

During the filling of a pit lake following components are important;

- Net groundwater inflow; is the difference between groundwater inflow and outflow rate. In a terminal sink lake there will not be an outflow from the lake water to the surrounding groundwater system.

- Net surface water inflow; is the difference between surface water inflow and outflow rate. Surface runoff and streams (both flows in and out) are the components.

- Direct and indirect precipitation (pit wall precipitation); Precipitation is one of the major component that determines the character of the pit lake. Direct precipitation is the rainfall to the surface of the pit lake and pit wall precipitation is the rainfall between the pit lake and open pit crest.

- Evaporation; is the outflow component of the system. Evaporation rate is calculated from the pan evaporation value and the surface area of the pit lake. Pan evaporation gives a higher value than the real lake evaporation. During the calculations, pan evaporation value generally multiplied with a coefficient smaller than one to obtain the real evaporation value.

Prediction of future pit lake level and filling period for and active mining open pit is basically dependent of the parameters above and net groundwater inflow is the most critical parameter to obtain. It can be obtained with a calibrated groundwater flow model that fully represents the groundwater regime of the site. Furthermore lake simulating groundwater models can be used to predict the final pit lake level and filling period.

In 1998 Shepherd Miller, Inc. (SMI) evaluated the hydrology of a lake that would ultimately form in an abandoned open mine pit. Using analytical modeling and best-estimate input parameters, SMI's initial evaluation indicated that the pit lake would be a terminal sink. Under these conditions, groundwater would flow towards the pit lake in all directions and water in the lake would not

recharge the aquifer system. After calculating the net inflow rates, a water balance model for the future pit lake was developed based on the expected pit geometry and assumed hydrologic input parameters. The model assumed that at any point in time, total inflows minus outflows were equal to the rate of change in the pit lake water volume. The rate of rise in the pit lake water level was then equal to the rate of change in water volume divided by the pit lake surface area. The pit lake depth over time was predicted by assuming a succession of steady-state water balance conditions over short time increments (Niccoli et al., 1998).

On the other hand, D.B. Stone and R.C. Fontaine used a lake simulating groundwater model (LAK2 package of Modflow) to predict the groundwater fluxes during open pit filling and under steady-state pit lake conditions. During their study the simulations have shown that the water table recovery was most rapid immediately after pumping stops, when the hydraulic gradients were steepest. The maximum lateral extent of water table drawdown occurs several years after pumping stops because water continues to be derived from storage as the pit fills. Under steady-state conditions, the lake stage was lower than the elevation of the water table in the pit area prior to mining, and groundwater flow was directed toward the pit lake, because evaporation from the lake surface causes it to act as a groundwater sink (Stone and Fontaine, 1998).

CHAPTER III

DESCRIPTION OF THE STUDY AREA

3.1 Geographical Location, Landscape and Land Ownership

The Marl Quarry is located east of the Y village in the X province of central Anatolia. The quarry is located on the flanks of the northeast-southwest trending mountain range in a relatively flat area at an approximate elevation of 1030 masl. The elevations in the area range from approximately 1400 masl in the mountains to 950 masl at the Skz Stream (Figure 3.1). The topography in the quarry area dips 6-10% and slopes towards the Skz Stream. Closer to the Skz Stream, the topography becomes flatter and slopes 0-4%.

The Marl Quarry is currently being mined at 1000 masl to a maximum depth of 992 masl. The quarry is a “hollow tooth” type. It contains a small pit

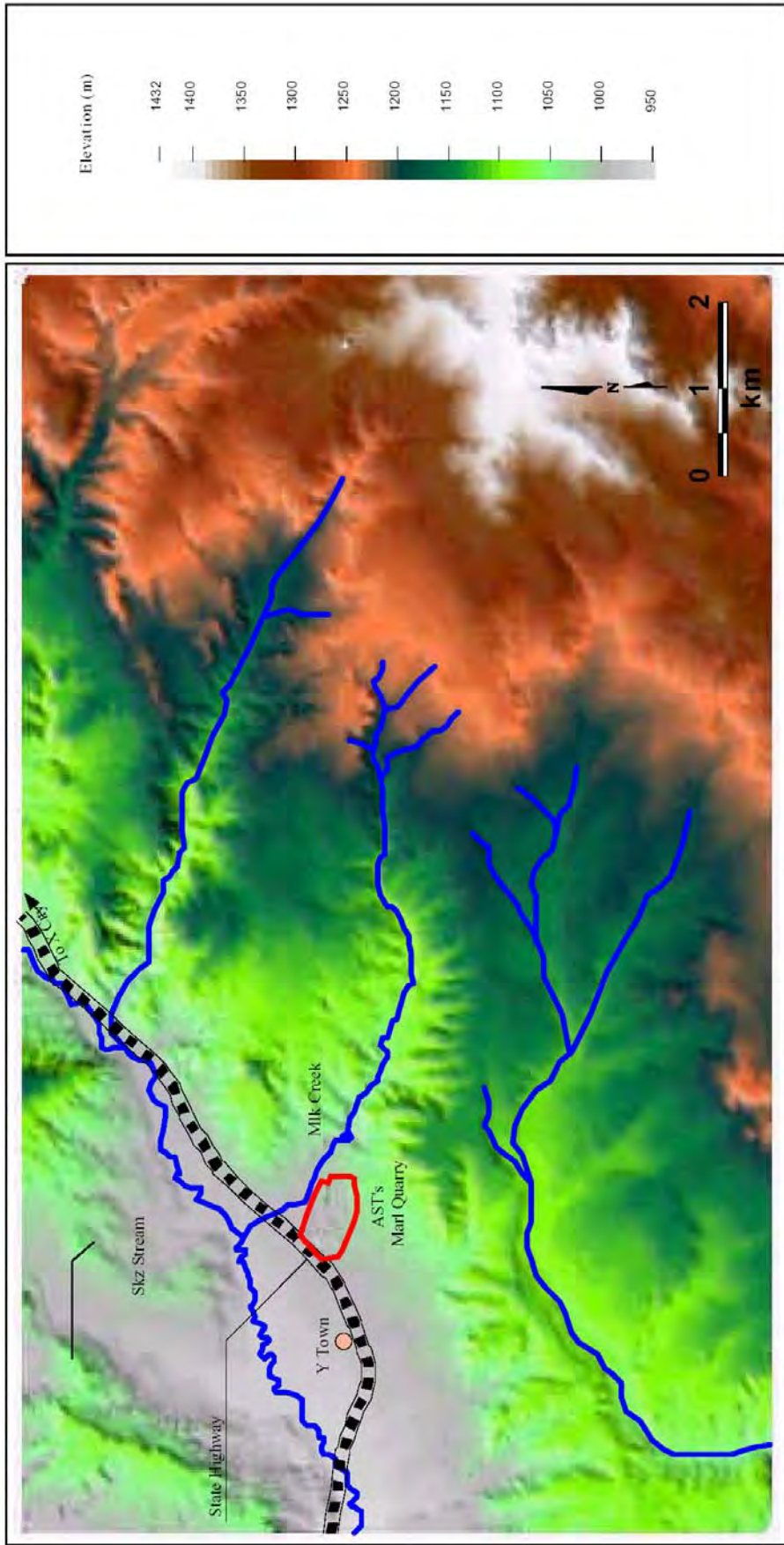


Figure 3.1. Geographical Location and Topography Map of AST's Marl Quarry

lake (24,492 m² surface area) at the western end and has a relatively flat bottom (Figure 3.2).

The licensed area of the quarry is 466,677 m². The quarrying operations are conducted within a 350,438 m² area, which is owned by AST. The remaining area is privately owned by local villagers and is used for dry agriculture (60,158 m² of remaining 116,228 m²). With the expansion project, AST is planning to purchase the remaining properties in order to develop the mine to its ultimate pit limits. This dewatering plan was prepared considering the final pit layout, and assuming that the remaining properties are purchased and are also mined to the existing pit elevations.

3.2 Climate

This region of Turkey is characterized as a continental climate; that is, hot and dry summer months and cold, snowy winter months. There is only one primary meteorological station operated by the State Meteorological Organization. It is in X City which is in the vicinity of the quarry and the plant site. The long-term data (1960-2000) from this station are used to define the climate of the region for this study. The climatic conditions at the project site are assumed to be similar to those at the X City Meteorological Station. This assumption should be valid for most climatological parameters with the exception of local wind conditions.

Long-term monthly average temperatures are given in Figure 3.3. The hottest and coldest months in the region are July and January, respectively. The

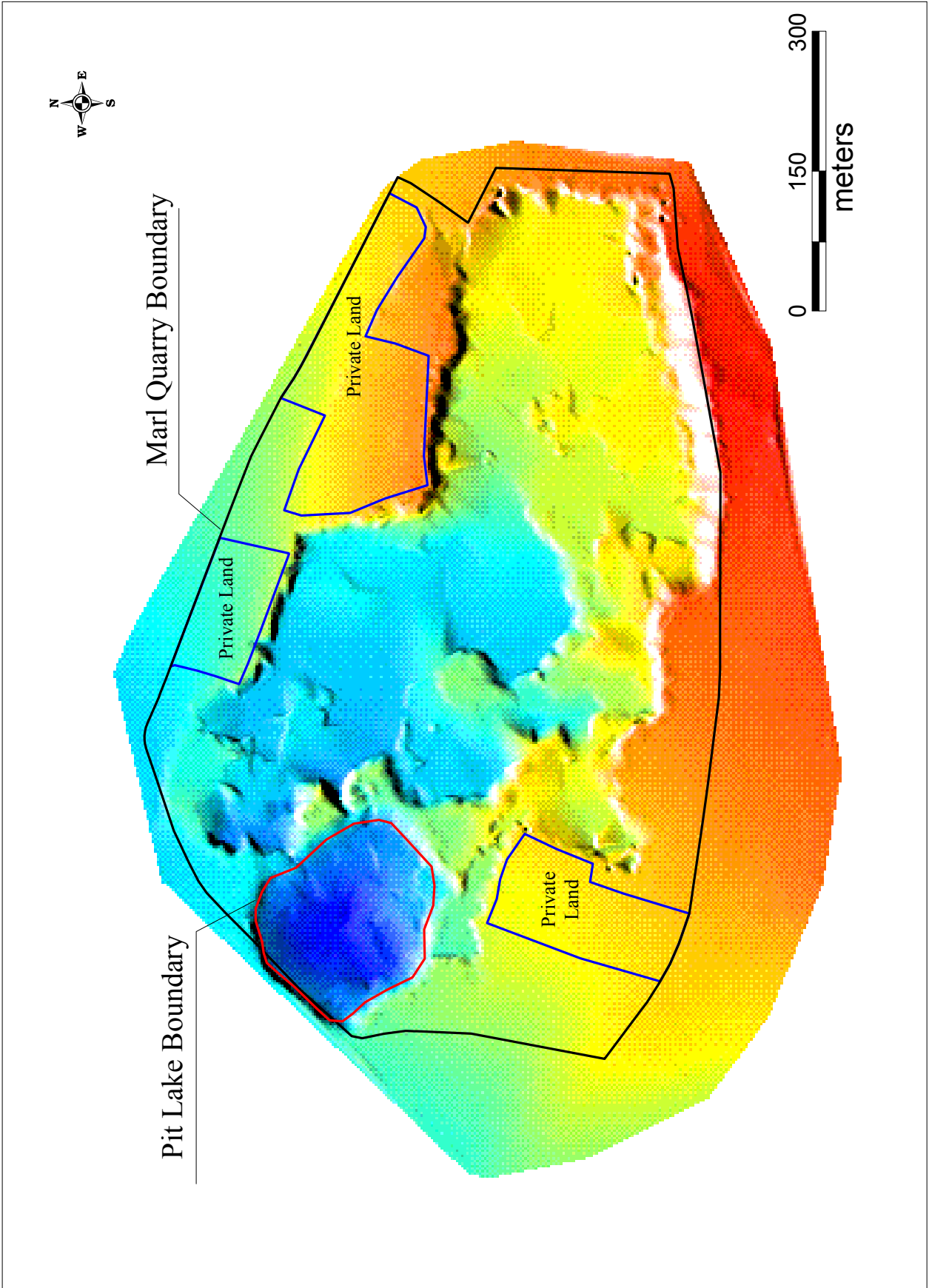
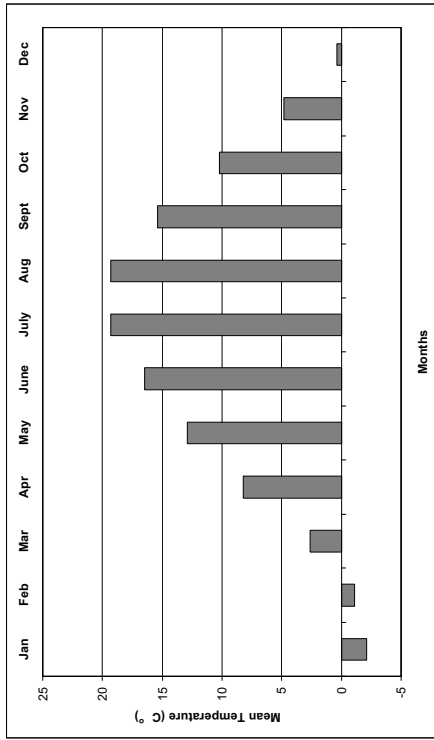
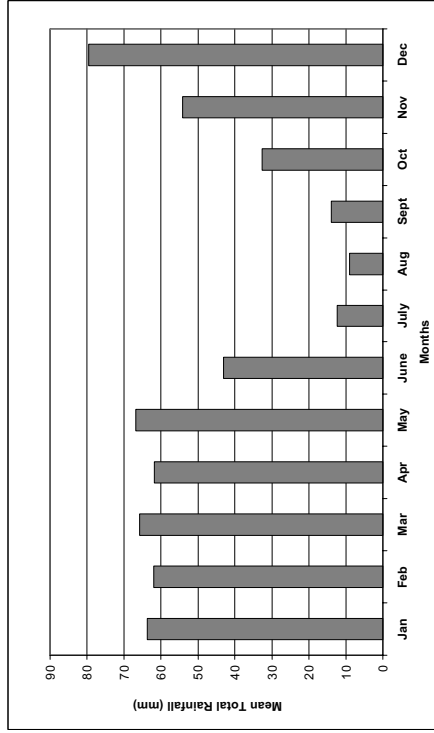


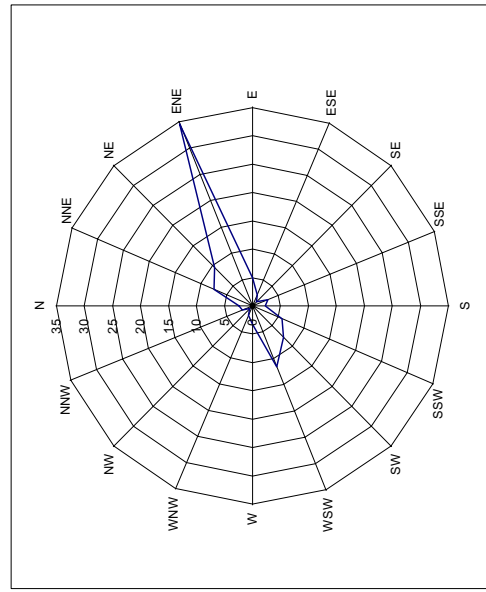
Figure 3.2. Landownership and Relief Map of AST's Quarry Site



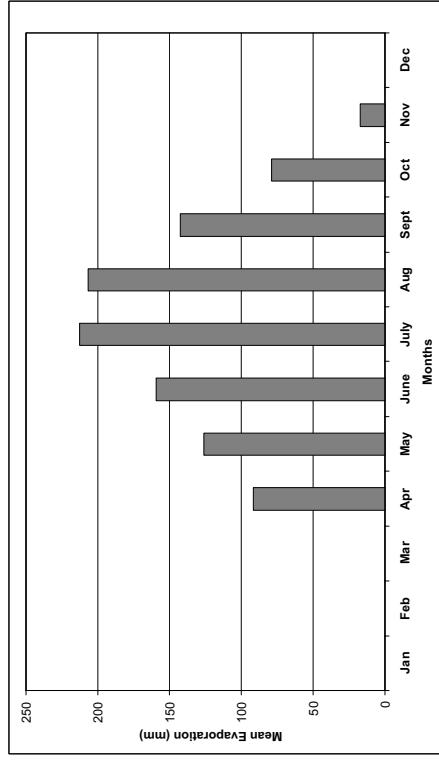
Average Temperature (1936 to present)



Average rainfall (65 years)



Average Wind Frequency (31 years)



Average Evaporation (5 years)

Figure 3.3. Climate Data

average temperatures range from a low of -2.1°C to a high of 19.3°C in July and August. There are 54.3 days in a year with temperatures above 25°C . The temperature is below 0°C for 107.8 days in a year on average.

The total average annual rainfall is 564.8 mm for the long term with the majority of the precipitation occurring in the winter and spring. The precipitation is in the form of snowfall in the winter. Monthly average precipitation is presented in Figure 3.3. The precipitation ranges from 9 mm in August to 79.6 mm in December. The first snowfall is observed in November and can continue until May. On average, snowfall can be observed 47 days per year. The average thickness of the snow is 92 cm. The monthly average evaporation rates are also shown in Figure 3.3. The highest and lowest evaporation rates in the region are until May. On average, snowfall can be observed 47 days per year. The average thickness of the snow is 92 cm. The monthly average evaporation rates are also shown in Figure 3.3.

The highest and lowest evaporation rates in the region are observed in July and January, respectively. The annual average evaporation is 1034.6 mm. The evaporation rate is higher than that of precipitation from April through October. Between November and April, there is a water surplus in the region. Most of the recharge to the groundwater system occurs during this period.

An annual wind rose for the X City Meteorological Station is presented in Figure 3.3. The dominant wind direction for X City is east-northeast (34.5%) from X City to Y Village. This dominant wind direction is observed particularly during dry summer months. West-southwesterly (WSW) winds occur 11.5% and

southwesterly (SW) winds occur 7.8% of the time. These are especially dominant during spring and autumn. The average wind speed is 2.9 m/sec. Wind directions and speeds were measured at the X City Meteorological Station.

As the topography of the region may influence the wind patterns by channeling, accelerating, or slowing the wind, it is possible that locally the prevalent wind speed and direction at site may vary somewhat from those of the X City Meteorological Station.

3.3 Geology

3.3.1 Regional Geology

The Marl Quarry is located within the drainage basin of the Mlk Creek, an ephemeral tributary of the Skz Stream. BNG Engineering Co. was studied the area in 2002 and determined three basic rock sequences are exposed in the Mlk Creek watershed area (Figure 3.4). They are, from bottom to top, Cretaceous Basement, Tertiary Sequences (Bl, Cme, Dlm and Zlk Formations), and Quaternary Alluvium Deposit.

3.3.1.1 Cretaceous Basement Rocks

The basement rocks are comprised of Lower Senonien (Upper Cretaceous) intrusive volcanics. The basement rocks extend in an E-W direction in the center

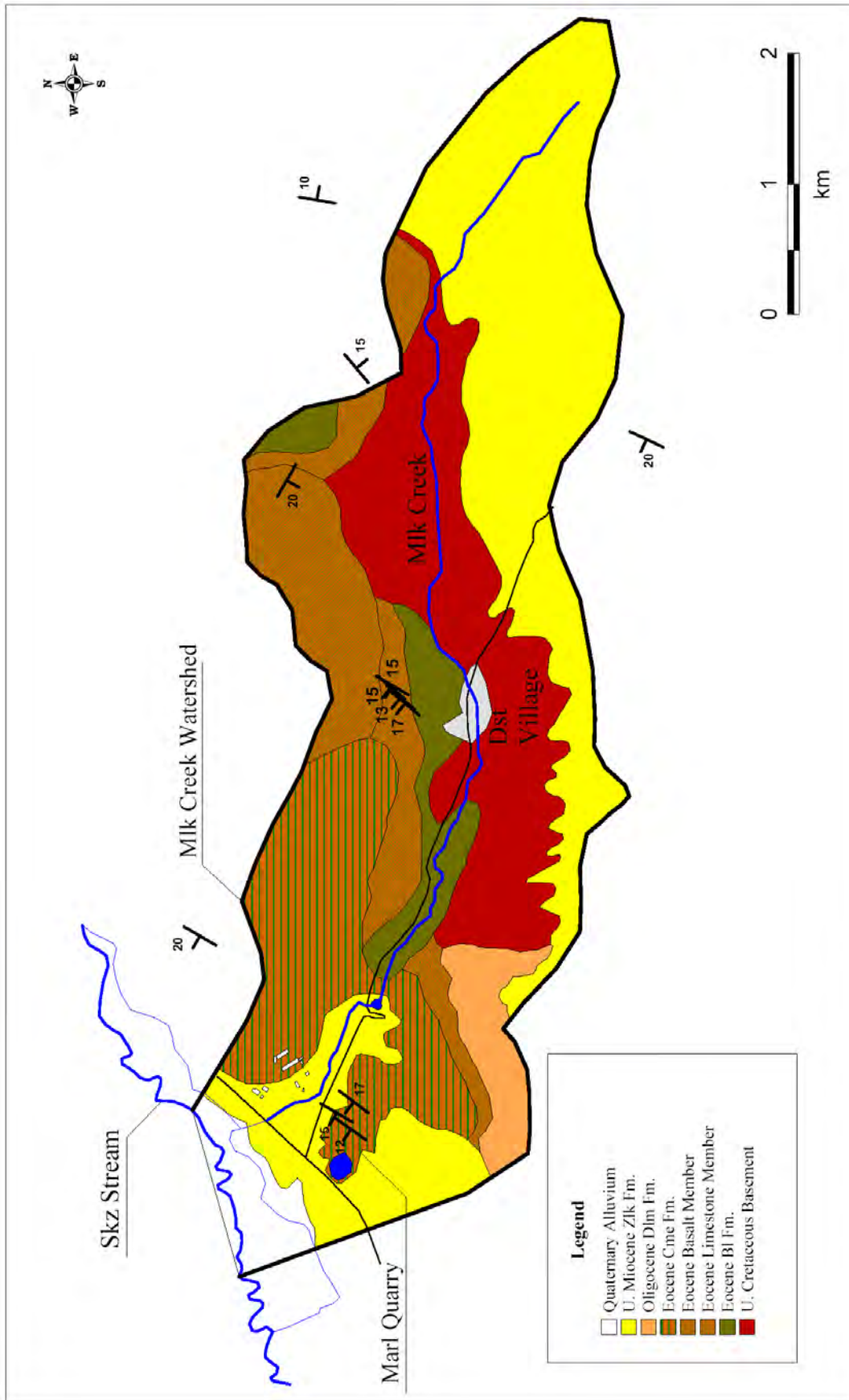


Figure 3.4. Regional Geological Map (After BNG,2002)

of the watershed, east of the marl quarry. They are gray, yellowish gray, reddish gray in color, holocrystalline, and intact. Although some surface alteration can be seen along the Mlk Creek, these volcanic rocks are mainly intact and fresh rock surfaces can be seen from road-cuts. The volcanic basement rocks are overlain by Eocene volcanic and sedimentary rocks.

3.3.1.2 Tertiary Sequence

The Tertiary sequence consists of marine clastics with carbonaceous and volcanic rocks. The formations comprising the sequence are the Bl, the Cme, the Dlm and the Zlk Formations.

The Bl Formation is made up of Eocene continental sedimentary rocks. It is exposed along the Mlk Creek and again north of the Dst Village (Figure 3.4). The formation consists of mudstone, sandstone and poorly sorted conglomerates that are brown, reddish brown, gray and contain angular to subrounded fragments. The Bl Formation unconformably overlies the basement rocks and shows lateral transitions with the Cme Formation.

The Cme Formation out crops at the marl quarry and in the surrounding near by vicinity. The formation is gray, greenish gray, brownish yellow and is mainly made up of, fragmented sandstone, mudstone, and limestone of marine origin with some volcanic intercalations. This formation was developed during a transgressive event during the Eocene Epoch and is composed of conglomerate-

sandstone alternation at its base, sandstone-siltstone- mudstone toward the upper levels. The marl present in the quarry is part of the Cme Formation. They are mostly homogeneous, light green to greenish gray colored, medium to thick bedded and soft to medium in hardness. The Cme Formation has two lithostratigraphic members which are:

The Apn Basalt Member, which is green, grayish green in color and made up of Eocene basalt, agglomerate, tuff and tuffaceous claystone, and The Dul Limestone Member, which is made up of gray, grayish beige-colored clayey and sandy limestones at the bottom and beige, thick-bedded and/or massive fossiliferous reef limestone toward the top.

The Cme Formation has a conformable boundary relationship with the underlying Bl Formation. However, it overlies the basement volcanics to the northeast of the Dst Village.

The Oligocene Dlm Formation is exposed in the southwestern part of the watershed. It is composed of red, reddish brown, gray colored, parallel and cross-bedded lake and stream sediments. It is made up of sandstones with mudstone and claystone intercalations. It unconformably overlies the Eocene and basement rocks.

The Zlk Formation consists of upper Miocene-Pliocene deposits and is exposed in the western and eastern segments of the watershed. It is brown to light brown and is made up of poorly-consolidated and poorly-cemented conglomerates, sandstones and siltstones with angular to subrounded Eocene

limestone fragments. The Zlk Formation unconformably overlies the Dlm and Cme Formations.

3.3.1.3 Quaternary Alluvium Deposit

The Quaternary alluvial deposits mainly consist of gravelly, sandy, and clayey materials. Alluvial deposits are observed along the Skz Stream and Mlk Creek valley bottoms and flat areas. It is mainly conglomeratic but loose and poorly cemented. Gravelly and sandy material of the alluvium has volcanic and sedimentary origin. The thickness of the alluvium may reach up to 15 meters near the Skz Stream.

3.3.2 Structural Geology

The major structural elements observed within the study area are bedding planes and fractures. Twelve bedding plane and 69 fracture dip-strike measurements were carried out in the study area. A rose diagram and stereo-net plots were used to characterize the orientation of the bedding planes and fractures. The field observations show almost a single NE trend in the bedding planes. The bedding planes dip gently towards the NW (Figure 3.5). The dip angles are between 13 and 17 degrees NW in the study area.

Detailed fracture dip-strike measurements were carried out within the quarry (Figure 3.6). In general, the fractures are almost vertical and sometimes

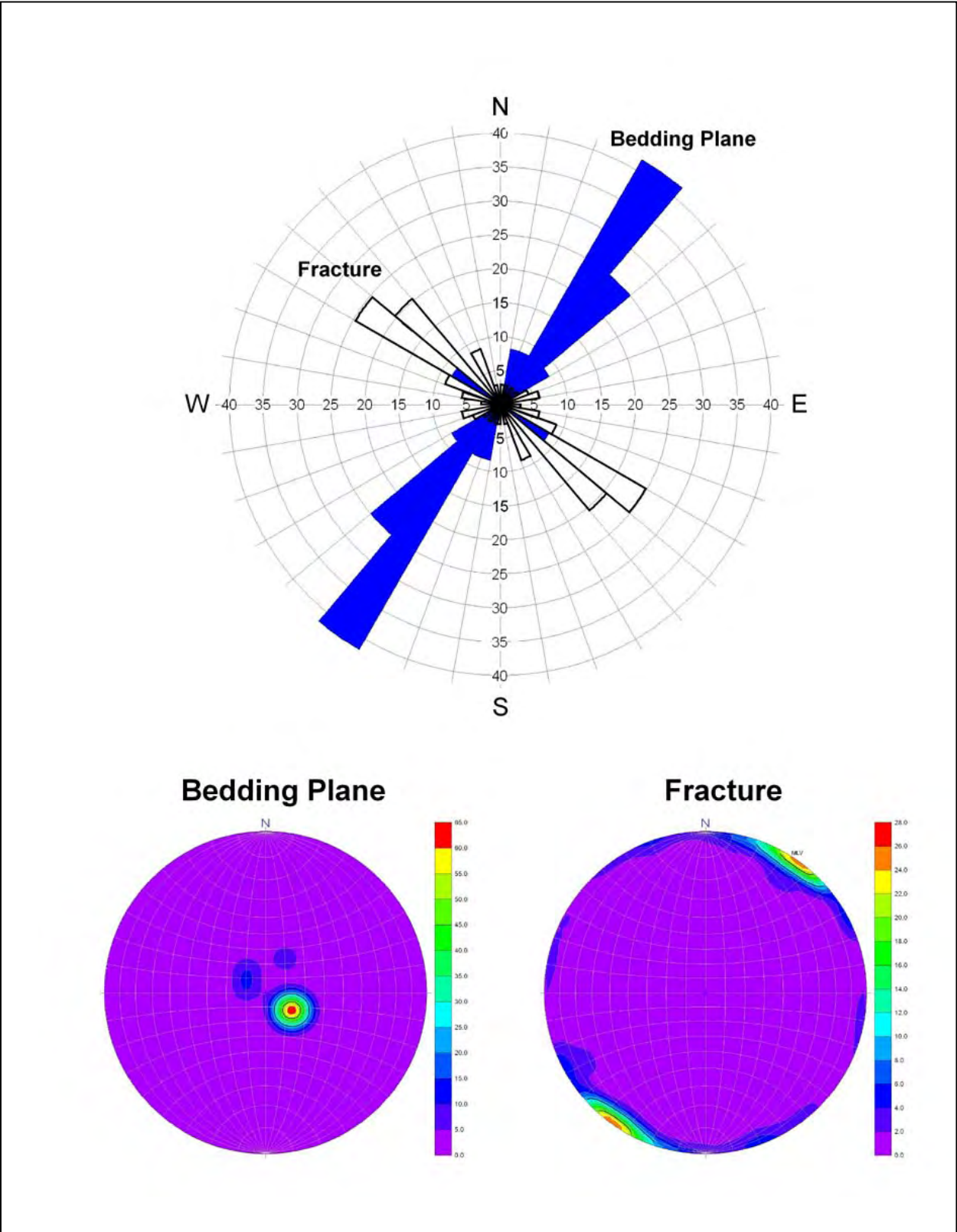


Figure 3.5. Steronet Plot of Fracture and Bedding Planes

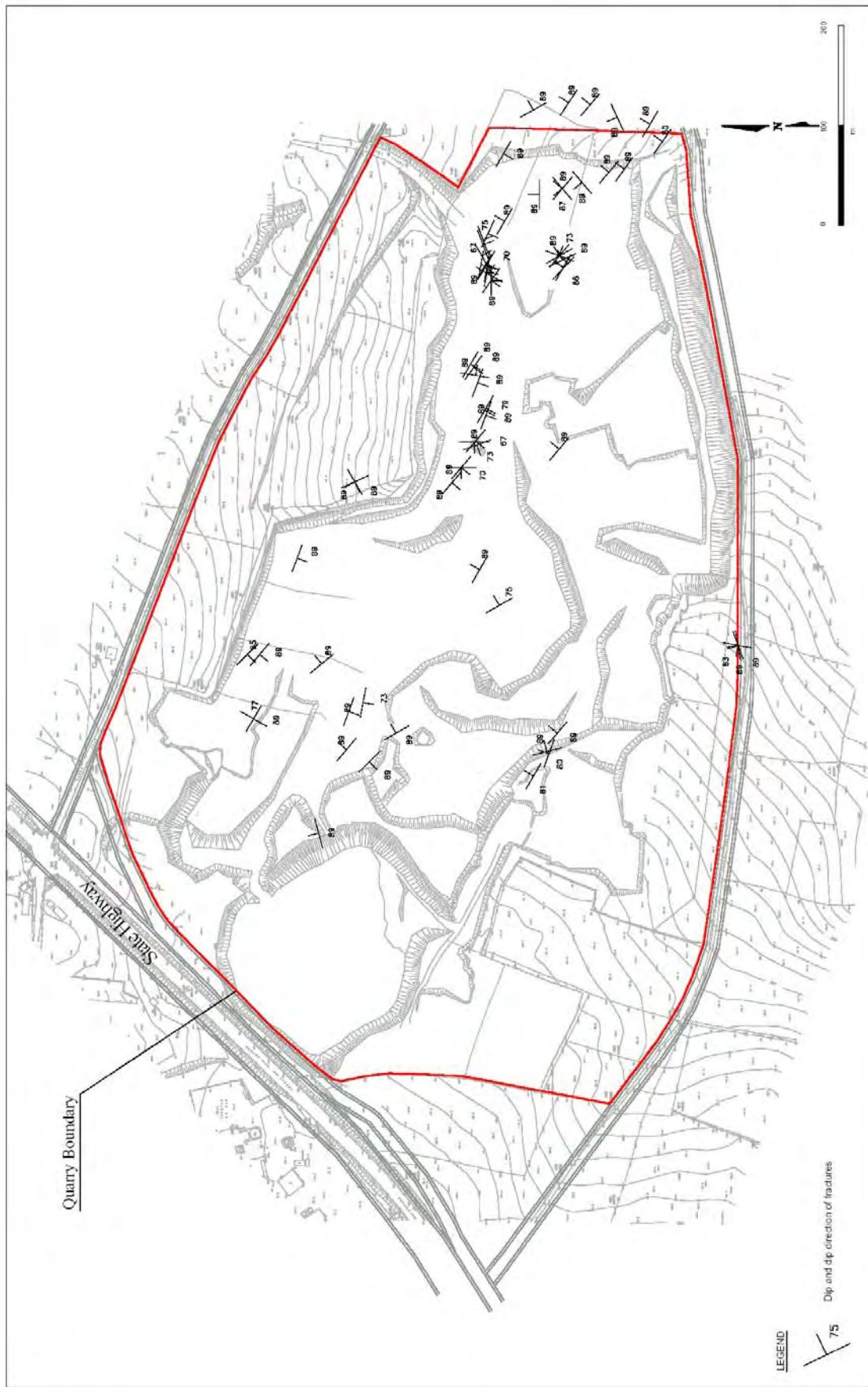


Figure 3.6. Fracture Orientations

filled with oxidation residue. The major strike orientation of the fracture system is perpendicular to the strike of the bedding (i.e. northwest). Fracture spacing was observed to be 50-100 cm and is well-developed (Figure 3.7).

3.4 Hydrogeology

3.4.1 Surface Water

The main surface water course in the vicinity of the project area is the Skz Stream which flows in NE-SW and is located about 500 meters to the northwest of the mine pit. As there was no available data for the long term discharge rates of the Skz Stream, a stream flow measurement study was conducted from two measurement points. These points are located at the upstream (SW-2) and downstream (SW-1) ends of a stream segment receiving base-flow from the Mlk Creek watershed. Figure 3.8 shows the location of surface water measurement stations.

The analyses of the stream-flow measurements at SW-1 and SW-2 show that the Skz Stream is gaining stream within the project site (Table 3.1). Stream-flow measurements were made with a water velocity probe. Measurements details can be found in Appendix A.

The Mlk Creek is an ephemeral stream in the immediate vicinity of the project site. It is E-W trending and drains the entire study area. During field investigations, discharge and water quality measurements from this creek were



Figure 3.7. Fracture System

taken. According to local villagers, the creek becomes dry during the summer and fall seasons. It only flows during the winter and spring. Some flood events have also occurred in the area in the past. Local people have constructed a small scale dam to control the possible flood events in the basin. This structure also protects the quarry from any flood event from the upstream side of the basin.

Table 3.1: Surveyed Water Resources in the Watershed

ID	pH	EC (microS/cm)	S (‰)	T (oC)	ORP (mV)	DO (mg/l)	DO (%)	Q (lt/sec)	DTW (m)
SW2	8.29	1001	0.3	7.2	205	8.75	87	236	
SW1	8.24	1015	0.3	6.4	204	8.31	81.5	266	
PW	8.26	3920	2	14.8	125	1.44	17.8	-	0.83
AST Well									11.46
AST Dug Well 1									5.85
AST Dug Well 2									8.22
Dug Well									2.62
Fountain 1	8.13	684	0.1	9.4	104	7.71	82.6	0.01	
Fountain 2	8.17	626	0.1	8.6	107	8.12	90.1	0.02	
Fountain 3	7.58	743	0.1	12.9	290	7.53	85.5	0.034	
Dst Spring	7.42	885	0.2	10.5	237	3.19	34.7	1.17	
Mlk Creek	8.01	831	0.2	8.8	240	8.05	85.3	0.4	
Fountain 4	7.87	1375	0.4	7.9	256	9.09	92.6	0.04	
Trench									2.31
Seepage	7.88	1132	0.3	10	62	9.77	104	0.02	
Pit Lake	9.23	5580	2.9	6.9	195	10.58	105	-	
Y Spring	7.61	812	0.2	10.2	211	4.27	46.1	1.7	

The pit lake within the quarry is another major surface water feature in the watershed area. There is no information or data available regarding the filling of the pit lake but according to the personal communication with the AST staff, the lake began to form during early 1980's, shortly after the excavation of the west side of the quarry reached a depth of 980 meters. The flows that contributed to the formation of the pit lake probably included inflow of groundwater, direct precipitation and the surface water run off from the surrounding quarry.

The volume of water in the lake was calculated using depths to the lake bottom at 38 locations taken during the field survey. These depth measurements were contoured using Mapinfo Software. The lake volume was calculated to be 111,500 m³ with an average depth of 7 meters and a surface area of 22,750 m². A water sample was collected from the lake to be analyzed for general chemistry parameters (major ions) and dissolved metals. The chemical analyses results can be found in Appendix A. Also field measurable parameters for the lake were measured during the field studies and can be seen in Table 3.1. The electrical conductivity of the water was measured at different depths using a point source bailer. The results show a uniform electrical conductivity distribution with depth.

3.4.2 Springs

Springs in the project area were located using 1/25000 topographic maps and then characterized during a seep survey conducted at the site in December 2003. A total of 27 points were visited and those that flowed are listed in Table

3.1 together with the fountains. Locations of these springs are also shown in Figure 3.8.

There are two significant springs in the watershed area, both of which are unnamed. One of them is located in the Dst Village. The discharge rate was 1.1 l/s during the time of visit (December 20003). The other one is located between the AST's marl quarry and the Dst Village. These springs will be referred to as Dst and Y springs hereafter. The Y spring is diverted into a channel to supply water to the Y Mosque. It had a discharge of 1.7 l/s at the time of measurement. During field studies three new water points were identified in addition to those shown on the topographic maps. These springs were generally located in the steep valleys and either dry or muddy and surrounded by established or emergent wetland/marsh type vegetation.

3.4.3 Wells

During the field studies, four water wells were identified in the watershed (Figure 3.8). Three of these wells are located within the property boundaries of AST. Two of them were dug wells and the third was a drilled and equipped well with a submersible pump. The fourth, a shallow dug well, was located on the eastern edge of the watershed area. Information about these wells was compiled and given in Table 3.1. AST has two additional water supply wells, which have been drilled in the alluvium of the Skz Stream. These wells are pumped at a total rate of 1.3 l/s and the water is delivered to the water treatment plant of AST with a

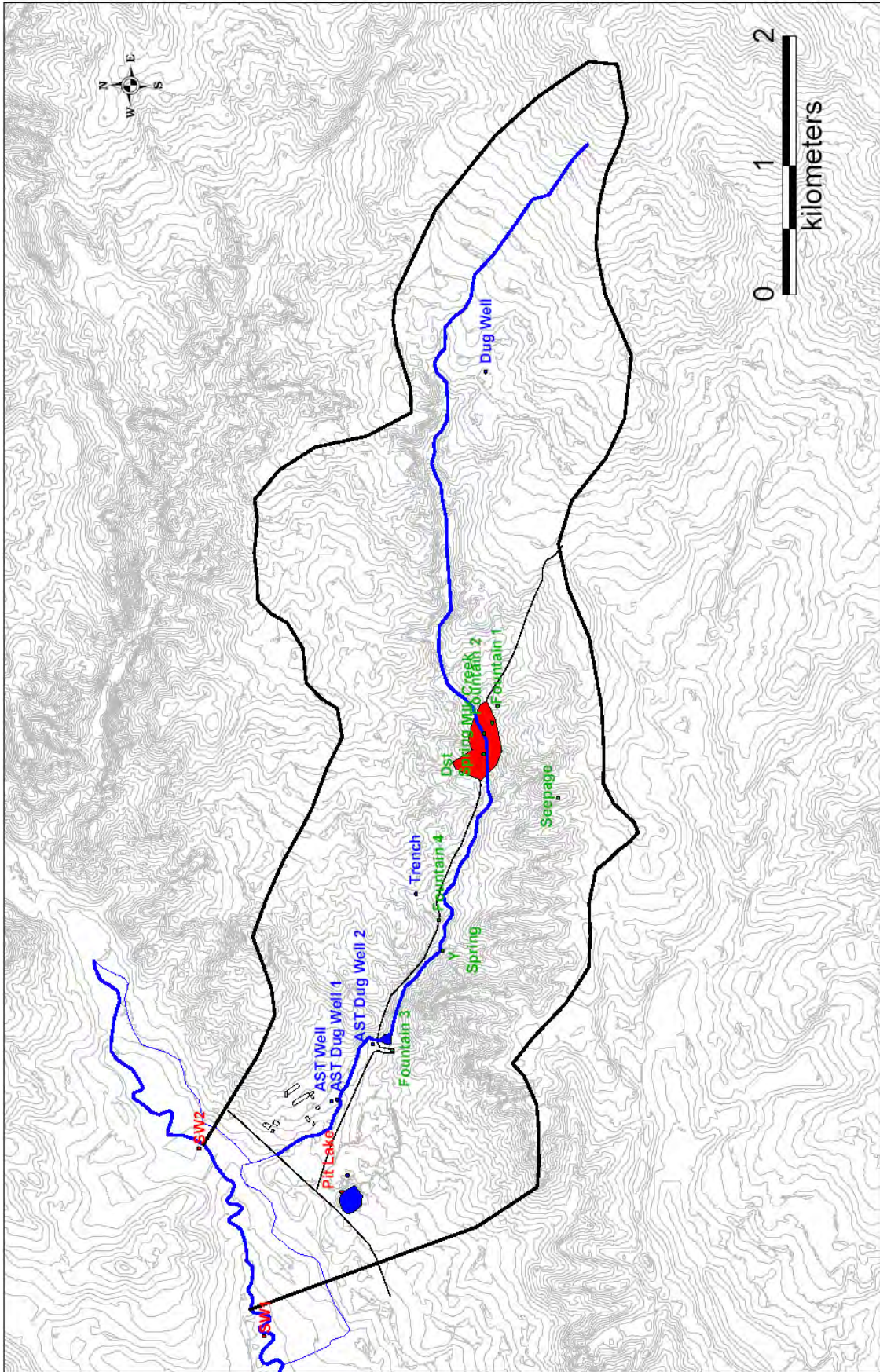


Figure 3.8. Locations of Hydrogeological Features in Milk Watershed

pipeline. In addition to these water wells, AST has drilled a number of exploration boreholes in the quarry and its vicinity. Six of these located within the quarry have been used for groundwater monitoring.

For this study four test wells were drilled and completed in the quarry and two of the previously-drilled AST exploration wells were converted into groundwater monitoring wells. Details concerning these wells are given in Chapter IV.

3.4.4 Groundwater

The water bearing units have been divided into three groups in order to assess the dewatering potential. These are the shallow alluvial deposits, the Cme Formation, and the basement rocks underlying the Cme Formation.

3.4.4.1 Alluvium Unit

The alluvial aquifer exists to the west of the quarry around the Skz Stream and is made up of poorly-graded sands and gravels, with some clay and silt. These poorly-sorted sediments are mainly derived from the volcanic and sedimentary units. The alluvium thickness increases in the direction of the Skz Stream and reaches a maximum of 15 meters (Tahir, 1987).

The alluvium is an unconfined aquifer with a fairly shallow water table. The groundwater levels range from 5 to 10 meters below ground surface with an

average saturated thickness of between 5 and 10 m. The depth to water was measured as 6.17 m in one shallow dug well located in the west of the Y village. There were no available data for the seasonal fluctuations in groundwater levels but it can be concluded that the fluctuation is about 5 m, from the personal communication with the local people.

There were no data available with respect to the aquifer characteristics of the alluvium. Due to the clayey and silty nature, the expected transmissivity value should not be very high.

Based on field investigations, groundwater flow mainly follows the topography of the land surface. The flow is from the mountains towards the Skz Stream. The measured hydraulic gradient is approximately 0.025 in the alluvium unit.

Local villagers do not use the groundwater from the alluvium unit in the project area. The main water source for the villagers is the municipal water, supplied by a pipeline. However the water to the AST facilities is supplied via two water wells in the alluvium unit.

3.4.4.2 Cme Formation

Cme Formation consists of fine grained sedimentary rocks. This formation is mainly made up of marl in the study area. It underlies the alluvium in the plain area and exposed in the low lying hills to the east. According to field

observations, there are two main features that effect the hydraulic properties of the marl:

- surface weathering, and
- well developed fracture system

Surface weathering can be observed in areas where the marl is exposed to atmospheric conditions. It is believed that the weathering of the marl causes an increase in the permeability of the unit. It can be concluded from the pit walls that effect of the weathering on the marl extends down to about 20-30 m below ground surface.

Fracture system controls the permeability of the marl. The main fracture system was developed in the NE-SW direction. Presence of the oxide residues accumulated on the fracture planes indicate the movement of water in these fractures. Depth to water is about 10m from the ground level and is about 1m from the pit floor. The average hydraulic gradient in this unit is about 0.04.

3.4.4.3 Basement Rocks

The Basement rocks are composed of Tertiary sedimentary and volcanic rocks and the Cretaceous volcanic rocks. The two significant springs (Y and Dst) that are present in the study area are discharging from basement rocks.

The surface weathering can be observed at the outcrops of this unit. There was no available information about the hydraulic characteristics of these rocks

from the previous studies but the physical appearance of these rocks gives an idea of low permeability.

A groundwater level contour map for the watershed area has been prepared with the data collected from the surface water, spring and groundwater features that have been surveyed in the watershed (Figure 3.9). During the contour preparation for the groundwater table Mapinfo software was utilized. The data was processed with triangulation method and plotted on the topographic map. As a second process the groundwater level contours were modified by hand, according to the topography of the basin. Then this map was digitized again to use it in the groundwater model.

According to the groundwater contour map;

- the groundwater levels are changing between 970 meters and 1370 meters, in the basin,
- levels are changing from 992 meters to 1010 meters for the AST Quarry site,
- based on the levels obtained from groundwater contour map, hydraulic gradient is estimated to be around 0.06 for the basin ,
- the hydraulic gradient is about 0.04 for the quarry site,
- Mlk Creek is a gaining creek along the watershed. It loses its gaining character as it intersects with the Skz Stream,
- similarly Skz Stream shows a gaining character in the study area,
- Existing pit lake is a flow through pit lake.

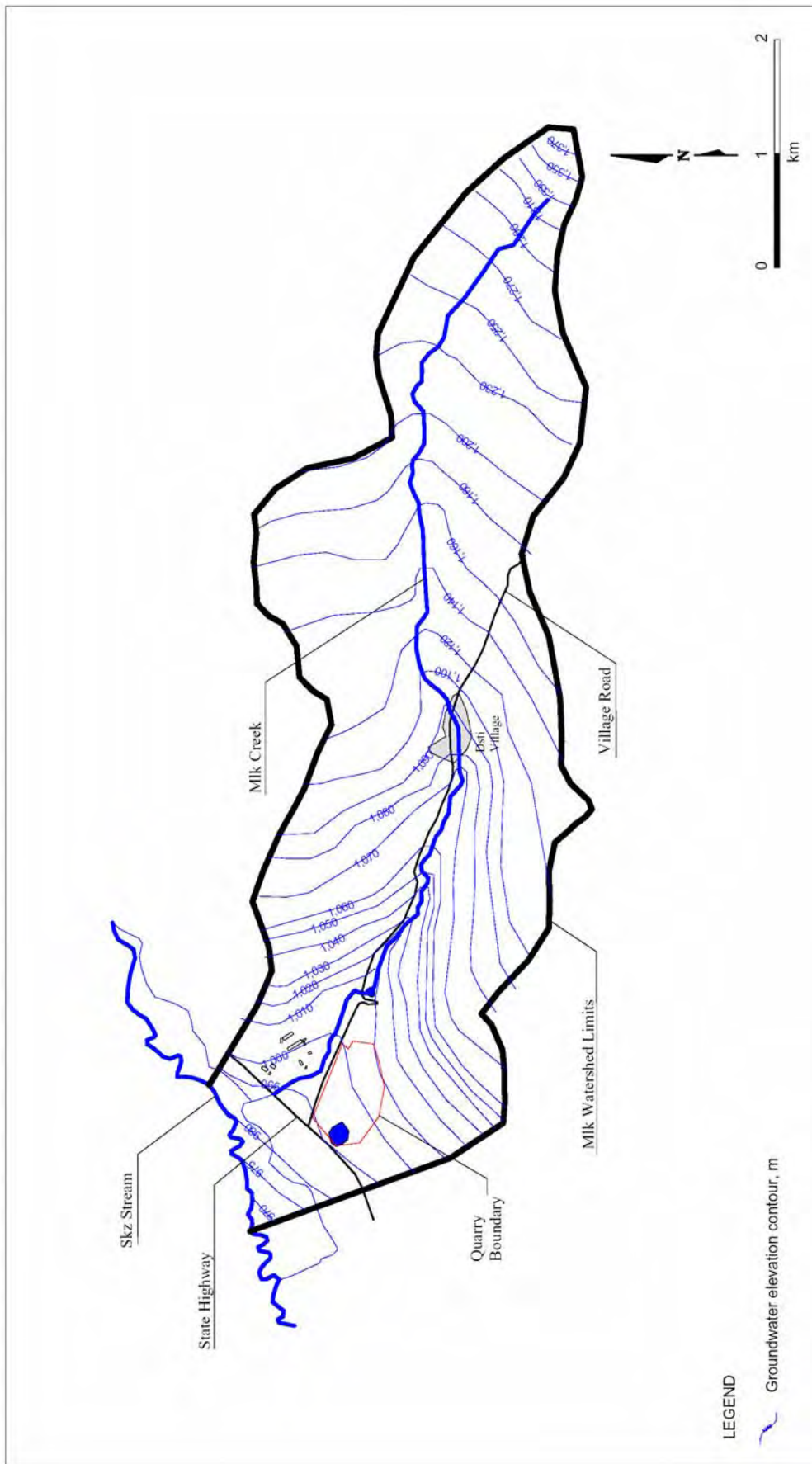


Figure 3.9. Groundwater Contour Map

3.5 Water Balance for Current Conditions

A complete water balance of the watershed area was prepared with respect to the inflows and outflows from the groundwater system. The water balance includes recharge from precipitation as the inflow component to the groundwater system. The outflow components include pumping discharge, baseflow to the Skz Stream and Mlk Creek and discharges from springs. It was assumed that the system is in equilibrium and that the total outflow is equal to total recharge. The existing water balance of the project area is summarized in Table 3.2. Precipitation is the ultimate source of recharge to the groundwater system. A percentage of precipitation will evaporate into the atmosphere as it falls. The remainder infiltrates to the groundwater system or runs off at the surface and then either infiltrates to the groundwater system or leaves the area as surface water flow. As shown in Table 3.2, total annual precipitation into the watershed area is about 10 hm³/y.

The Thornthwaite method was utilized to estimate the amount of recharge to groundwater. Most of the precipitation will run off (64%). Some of it (approximately 30%) will evaporate back into the atmosphere and the remainder (approximately 6%) will recharge the groundwater system.

The yearly recharge values in the watershed area were as follow:

- Areal recharge to the alluvium: 0.22 hm³/y (18% of the precipitation)

- Areal recharge to the marl and basement unit: 0.36 hm³/y (4% of the precipitation)

Most of the outflow from the watershed area is the baseflow to Skz Stream. The total discharge (outflow) from the watershed was calculated as 0.58 hm³/y. The baseflow constitutes 0.47 hm³/y (80%) of the outflow. This baseflow rate was calculated from the stream flow measurements during the field studies. The Skz Stream is a gaining stream in the downstream of the Mlk Creek watershed. The stream-flow difference between the upstream (SW-2) and downstream (SW-1) points was 30 l/s. Half of this amount is the base-flow from the other side of the stream. As a result the base-flow to Skz Stream from the Mlk Creek watershed was calculated as 15 l/s. The total discharge from the springs and the Mlk Creek is 3.4 l/s. Groundwater use via pumping wells in the area was minimum. There was only the AST water supply well which was located in the AST property and discharging with a flowrate of 0.2 l/s. The evapotranspiration calculated for the study area is insignificant because groundwater level is generally deeper than 2m.

Table 3.2: Water Balance for the Milk Watershed Under Current Conditions.

Hydrologic budget components			
Hydrologic Component	Annual Average (m3)	Apportionment (%)	
Precipitation	10,155,172	100	
Surface Runoff	6,498,592	64	
Evapotranspiration	3,071,772	30	
Groundwater Recharge*	584,807	6	
Surface Area of the Milk Watershed; 17942000m ²			
Groundwater budget in the Project Area			
Recharge	m³/year	Discharge	m³/year
i. Recharge from precipitation		i. Baseflow to Skz Stream	473,040
Alluvium	222,258	ii. Discharge from Springs & wells	113,427
Mam+Basement	362,549		
TOTAL RECHARGE	584,807	TOTAL DISCHARGE	586,467

CHAPTER IV

FIELD ACTIVITIES AND ANALYSES

In addition to spring and well survey and surface flow measurements, a set of tests were performed to understand the hydraulic characteristics of the Cme Formation in the quarry.

4.1 Introduction

The field observations indicated that groundwater is just below the existing pit floor. A set of in-situ tests was conducted to obtain information regarding the aquifer hydraulic parameters (hydraulic conductivity, specific yield, etc.) and boundary conditions for marl, considering the heterogeneity of the system and the existence of the structural features (fractures, etc.). A pumping well and three

observation wells were drilled. Location and layout of the wells were selected to observe the areal characteristics of the marl (Figure 4.1). Pump Well (PW) was drilled on the northeastern side of the existing pit. Observation Well-1 (OW-1) was drilled between the existing pit lake and PW, in order to monitor the probable water flow from pit lake to the pump well which has been drilled in the marl. Also another aim of locating the OW-1 monitoring well between lake and the PW boundary effect of the pit lake during the test. Observation Well-2 (OW-2) and Observation Well-3 (OW-3) were aligned with the PW in the main dip direction. This direction was also the direction of the main fracture system to see the effect of the system on pumping test response.

4.2 Drilling of Pumping and Observation Wells

The pumping well (PW) and three observation wells (OW-1, OW-2 and OW-3) were drilled and completed to various depths. PW was drilled to a depth of 41 m. OW-1 and OW-2 were drilled to a depth of 21 m and OW-3 was drilled 11 m. In addition to these wells, two previously drilled exploration boreholes were converted to groundwater monitoring wells and renamed as OW-4 and OW-5, respectively. OW-4 was completed at 50 meters and OW-5 was completed at 20 meters. Figure 4.1 shows the locations of the drilled and completed wells.

Pumping and observation wells were drilled with direct rotary technique, and during drilling water was used as drilling fluid. The pumping well borehole diameter is 12¼ inches and completed with a PVC casing of 8 inch diameter.

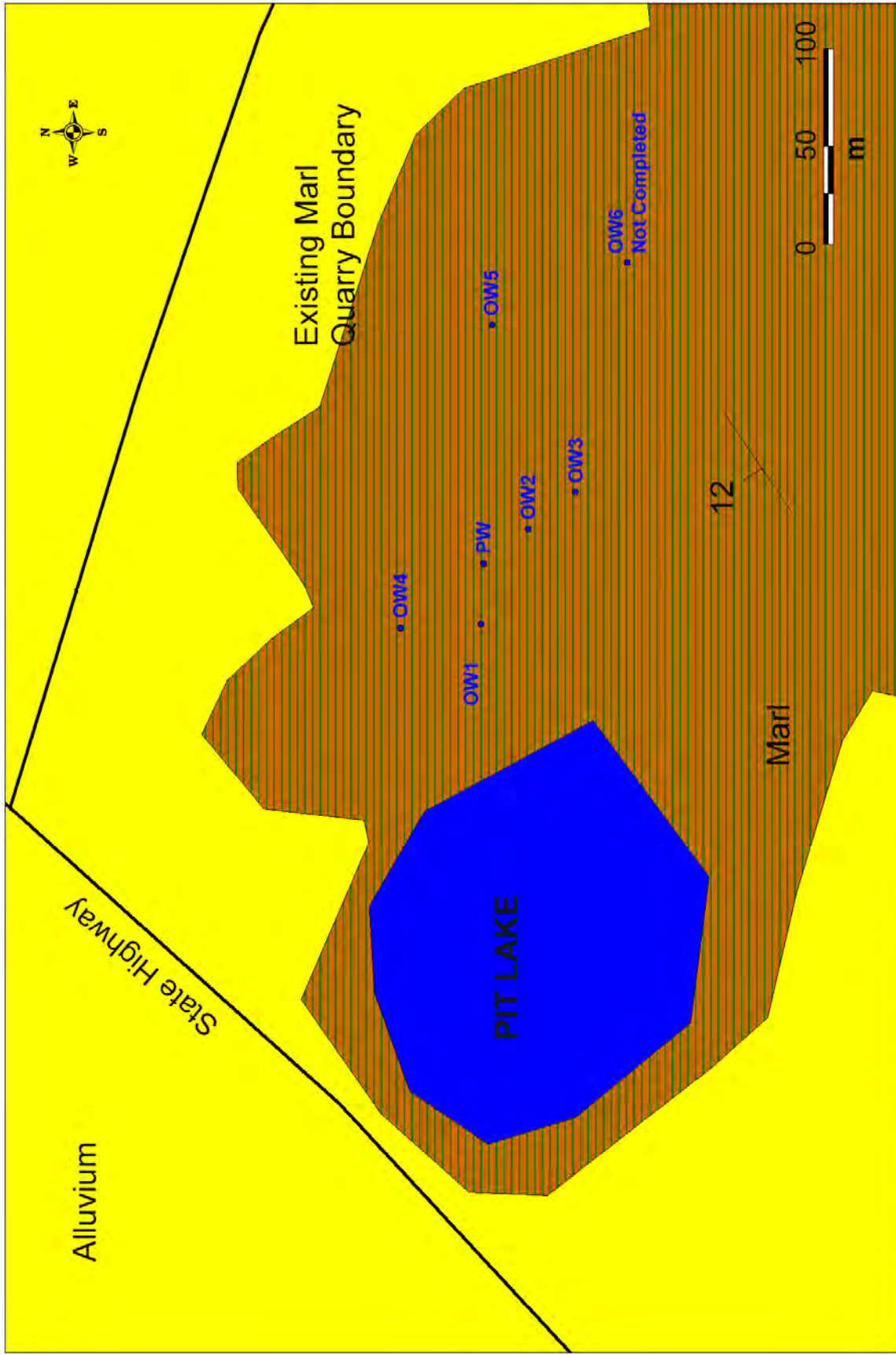


Figure 4.1. Locations of Drilled and Completed Wells

Observation wells were drilled using the same technique with a borehole diameter of 8½ inches. Each observation well was completed with 4-inch diameter PVC casing and screens. The diameter of the converted exploration wells was 10 cm and completed with 2-inch diameter PVC casing and screens. OW6 was another exploration well. It was not completed during this study but well was used during the field tests. Well logs and construction details for PW, and OW-1, 2, and 3 can be found in Appendix B. Information about the completion details of the wells is given in Table 4.1.

Table 4.1: Completion Details of Test Wells

	Elevation (m)	Depth (m)	Hole radius (m)	Casing (m)	Screen (m)	Gravel Pack (m)	Bentonite (m)	Cement (m)
PW	994.5	41	10 1/2	40---0	40---8	41---6	6---2	1---0
OW1	993.96	21	8 1/2	20---0	20---4	21---3	3---1	1---0
OW2	995.7	21	8 1/2	20---0	20---4	21---3	3---1	1---0
OW3	995.56	11	8 1/2	10---0	10---4	11---3	3---1	1---0
OW4	996.84	50	~4	50---0	50---4	50---3	3---1	1---0
OW5	997.45	30	~4	20---0	20---4	30---3	3---1	1---0
OW6	997.74	30	~4	Not Completed				

Well development is a process of maximizing the hydraulic connection between the aquifer and well screen to produce sediment-free water at maximum specific capacity. After the completion, the wells were washed with clean water and then developed with a stainless steel bailer and air lift system. More than one well volume of standing water is purged out with bailing process. PW was also developed with a 4 l/s capacity submersible pump.

4.3 Aquifer Testing

4.3.1 Introduction

A series of aquifer tests were conducted to understand the hydrogeological properties of the marl. Each aquifer test comprised of constant discharge, recovery and slug tests. During the constant discharge test the pumping rate and drawdowns were measured in the pumping well and observation wells. The slug tests are conducted separately at each well. Aquifer parameters were determined from time-drawdown data in the pumping well and the observation wells.

4.3.2 Pump Test

The production well PW used for the pump test was completed in marl and screened from a depth of 3 to 40 m, with a 4.4 horsepower submersible pump

installed at a depth of 30 m below land surface (water intake of the pump). A 50kW diesel generator powered the pump. The wellhead was equipped with a discharge valve and a totalizing and instantaneous flow meter. Discharge was routed through a 4 in. flexible hose into the 200 m North to an isolated sump. Water level in the pump well was measured with a pressure transducer, through a 1 in. diameter perforated PVC tube extending from the wellhead to the top of the pump. Also an electric water level meter was used during the pumping and recovery period to verify the data collected by the pressure transducer.

During well development it has been observed that there was not enough inflow to the well from the surrounding formation to conduct a full pump test. The emphasis of testing was therefore on the well recovery and slug testing to determine the hydraulic properties of the marls.

On December 5, 2003, a constant rate test was conducted. The flow rate was chosen as 0.13 liters per second and the groundwater level in the well reached to the top of the pump after 113 minutes (a drawdown of 25.17m). A total of 881L was pumped out from the pump well. Recovery data was collected for 1345 minutes and a recovery of 96% was reached during this period.

4.3.3 Slug Tests

After completing the pump testing, six slug tests were performed to assess the permeability of the marl. Slug tests are used to determine the hydraulic properties of subsurface formations at a relatively small scale around individual

boreholes. Slug tests are conducted by measuring the rise and/or fall of the water level in a borehole caused by a sudden introduction of a slug that displaces the water. The duration of a slug test is relatively short, and the estimated permeability determined from the test is considered to be representative only of water-bearing material close to the borehole (Ferris and others, 1962).

Slug tests were performed in PW-1, OW-1, OW-2, OW-4, OW-5 and OW-6. Since there was not enough water in OW-3, slug test was not performed in this well. Two different slugs were used during tests. Both were stainless steel, capped on both ends and having 4.6 and 1.4 liter volumes.

Drawdown-time data was collected with the electronic pressure transducers, installed 3 meters below the static groundwater level. The slug was introduced to the well and lowered down suddenly below the static level. This causes an increase in the static level. Using a water level meter, static level was checked until it has equilibrated. The rate of fall of water table gives an idea about the hydraulic conductivity of the aquifer. Slugtest data was collected from the wells for more than 4 hours.

4.3.4 Test Results

The aquifer test results were analyzed using software, AQTESOLV, which provides analytical solutions for determining aquifer properties from pumping or slug tests. The field data from each test were entered into this software. The data

points were fitted with a type-curve visually or automatically. The aquifer parameters were then calculated by the software.

Pump Test: The data collected by the pressure transducer show that drawdown was decreasing almost linearly during the pumping period of the test. This shows that groundwater inflow to the well was very small as compared to the pump discharge. This makes impossible to analyze the pumping period part of the data.

The recovery period was analyzed with Theis (1935) solution. This solution originally was developed for recovery data that has been gathered from confined aquifers. However, Neuman (1975) showed that the Theis recovery method is also applicable in unconfined aquifers, but only for the late time recovery data. At late time, the effects of elastic storage, which set in after pumping stopped, have dissipated. The residual drawdown data will then fall on a straight line in the semi log s' versus t/t' plot used in the Theis recovery method. After entering the data and fitting the straight line to the late time data, AQTESOLV estimated a transmissivity value of $1.7 \times 10^{-7} \text{ m}^2/\text{s}$. This corresponds to a hydraulic conductivity value of $4.25 \times 10^{-9} \text{ m/s}$ for a saturated thickness of 40 m.

Slug Test: The slug test data collected from OW-2, OW-5 and OW-6 was not applicable for an analysis. This may due to the clayey nature of the formation. On the other hand, slug test data from wells PW-1, OW-1 and OW-4 were analyzed with Bouwer and Rice's method using AQTESOLV. The recovery data from the pump test was also analyzed as a slug test. To determine the hydraulic

conductivity of an unconfined aquifer from a slug test, Bouwer and Rice (1976) represented a method that is based on Thiem's equation and with this equation AQTESOLV calculated the hydraulic conductivity values as shown in Table 4.2.

As can be seen from Table 4.2 slug test results gave an average hydraulic conductivity value of 8.16×10^{-8} m/s. However, the hydraulic conductivity calculated from the recovery data is smaller than the slug-test average value. The difference between the results of slug tests and the pump test may be originated from the nature of the pump test. As described above, the pump test was not successful due to the low permeable nature of the system. The results of the aquifer testing are presented in Appendix B.

Table 4.2: Calculated Hydraulic Conductivity Values

Well ID	Hydraulic conductivity Values (m/s)	
	Slug	Recovery
PW-1	1.16E-07	4.25E-09
Solved as slug	3.62E-08	-
OW-1	1.13E-07	-
OW-4	1.00E-08	-
Average	6.88E-08	

CHAPTER V

GROUNDWATER MODEL

5.1 Conceptual Model

A conceptual hydrogeological model describing the stratigraphic framework and hydraulic characteristics of the geologic units together with the hydrologic features, was developed for incorporation into the numerical model.

The conceptual model consists of three principal hydrogeological units:

1. Basement rocks, that underlies the Cme Formation,
2. Eocene Cme Formation, excluding the basalt and limestone members, and
3. Miocene Zlk Formation and Quaternary alluvium (Hereinafter Alluvium).

The conceptual hydrogeological model is shown in Figure 5.1. The boundaries of the conceptual model were selected as the Skz Stream to the north,

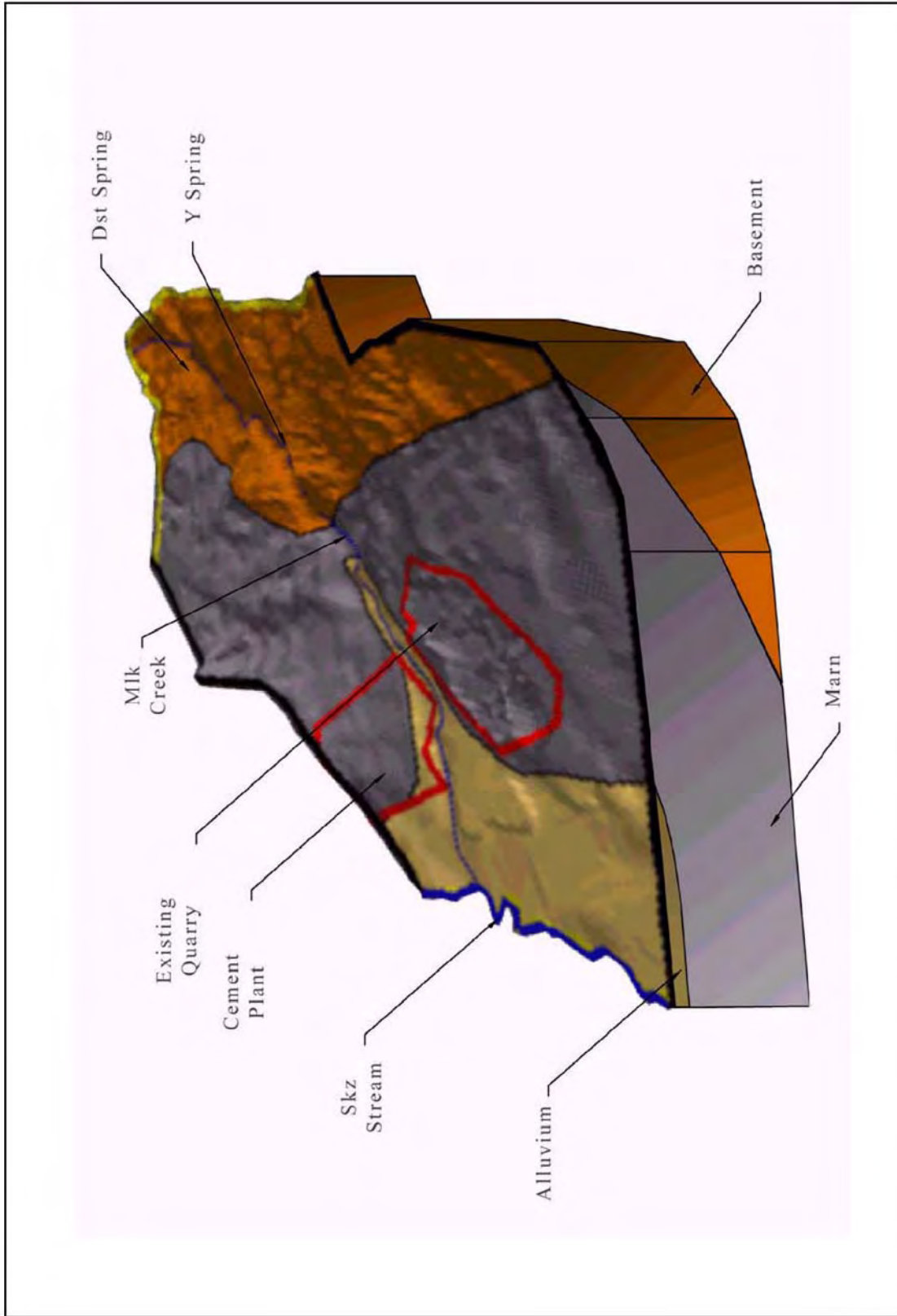


Figure 5.1: Conceptual Hydrogeological Model

watershed boundaries on the northeast and west, and 1100 m contour of the water table on the east.

The conceptual model represents hydrogeological units to a depth of 200-250 m below ground surface. The boundary between the Cme Formation and the basement group was determined based on the data from Mineral Research Institute reports, which were prepared for metal exploration and Mineral Research Institute (MTA) exploration borehole, which was drilled at the plant site to a depth of 258 m. Surface geology and bedding plane data was also used to develop the boundary. The Eocene Cme Formation is the principal rock type found in the pit area. Two members of the Cme Formation (Apn Basalt and Dul Limestone Members) were also included to the basement group of rocks.

Alluvial deposits are found along the Skz Stream and Mlk Creek. The thickness of the alluvium is about 15 m around the Skz Stream. The thickness distribution for this unit was completed with the data from previous MTA reports and borehole data from the AST. Since there was no permeability data available for the alluvium, a representative value of 4.0×10^{-5} m/sec was assigned as the hydraulic conductivity for this unit.

Hydraulic parameters for the marl in the model area were obtained from the in-situ tests. Hydraulic conductivity values for the marl unit range from 1.1×10^{-7} m/sec to 4.3×10^{-9} m/sec. Basement was considered as a low permeability unit. Hydraulic conductivity values for the basement were chosen between 3.2×10^{-7} m/sec and 5.0×10^{-9} m/sec. Storage values for the model were

assigned after a literature search, as 0.05 for the alluvium and 0.000025 for the marl and the basement. These values are typical for these rock units.

The input to the model area consists of recharge from precipitation and subsurface inflow from the eastern boundary of the model. The outflow from this sub-basin constitutes the baseflow to the Skz Stream and the Milk Creek, and two major springs, Dst and Y.

5.2 Model Setup

A groundwater flow model was developed to evaluate the dewatering scheme and the impact of dewatering to groundwater resources and water users. The MODFLOW (McDonald and Harbaugh, 1984) computer code, running under the Groundwater Modeling System (GMS) environment, was utilized to model the groundwater regime of the study area. MODFLOW is a 3-dimensional code and is capable of simulating a groundwater flow system that takes into account discharge from wells, interactions with streams, variable recharge and a variety of boundary conditions.

The modeled area covers approximately 8 km². The area is divided into uniform 20m X 20m grids, with finer grid spacing of 10m X 10m, at the pit area. The model consists of five layers. Boundary conditions of the model area, aquifer thickness and hydraulic conductivity of the aquifer material, groundwater recharge and discharge are incorporated into the model as described in the conceptual model section.

Geological and hydrogeological characteristics of the site were considered when determining the model area and boundary conditions. The Skz Stream was treated as constant head boundary due to the perennial character of the stream. Mlk Creek were treated as a drain. Dst and Y springs were also introduced to the model as drains. The drain package is used to simulate the effect of drains in an aquifer. Drains remove water from the aquifer as long as the water table is above the elevation of the drain. If the water table falls below the elevation of drain, the drain has no effect. It is the most relevant package in MODFLOW to simulate the seasonal water flow in a creek. Northeastern and southwestern boundaries of the model area were treated as no flow boundary. The eastern boundary condition, with an 1100 m groundwater level contour, was treated as a general head boundary to simulate the subsurface inflow from the upstream section of the watershed. The general head package is similar to the drain in that flow in or out of a cell is proportional to the difference in head. If the water table elevation rises above the specified head, water flows out of the aquifer. If it falls below the specified head, water flows into the aquifer. During construction of the model, the water supply well was also considered. The discharge rate was assigned to the model accordingly with the construction of the well. Recharge from precipitation and discharge through evapotranspiration were also introduced to the model using the recharge and evapotranspiration packages in MODFLOW. The rates for these two components were derived from the analysis of the existing water balance given in Chapter III. The evapotranspiration ceases when the groundwater table is deeper than 2 m. Figure 5.2 describes the model boundary conditions.

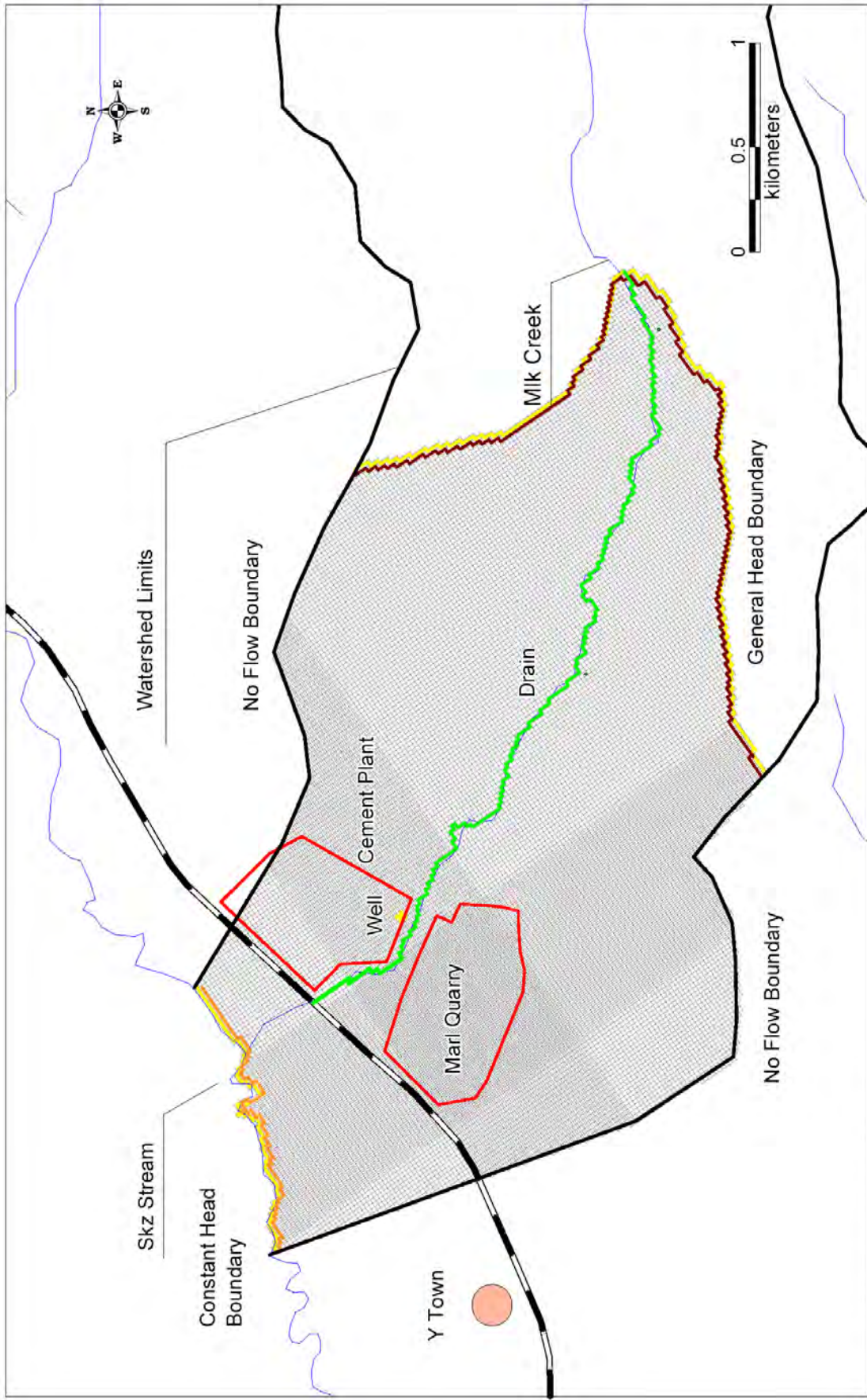


Figure 5.2. Model Boundary Conditions

Existing groundwater elevations were obtained from previously drilled exploration boreholes and recently drilled groundwater exploration/ test wells, springs and other water points. These groundwater levels were shown as a contour map in Figure 3.9.

5.3 Calibration

The model simulations were performed to match the calculated head distribution and the groundwater budget with the field observed groundwater levels (also the groundwater contour map) and discharges, which has been measured during the field studies. This process, which is known as calibration, consists of adjusting the model parameters until a best match is obtained between the observed and simulated levels. During the initial calibration, simulated groundwater levels were matched with the existing groundwater levels at the site. Also the discharge rates from springs and flow to the Skz Stream was checked during the calibration. The measured discharges for the Dst and Y springs were 1.1 l/s and 1.7l/s, respectively. The simulated value for Dst spring was 1.1 l/s and Y spring was 1.76 l/s. Baseflow to Skz stream was measured as 15 l/s and the simulations indicated that the calculated baseflow to Skz stream was 12.2 l/s. Milk creeks discharge value was measured as 0.4 l/s at the time of measurement. The simulated value for the creek was 1.12 l/s. This can be accepted as an accurate result for a steady-state simulation when the seasonal effects included. The subsurface inflow to the model from the general head boundary was simulated as

6.17 l/s. This value was checked with a simple application of Darcy's Law. With hydraulic conductivity, hydraulic gradient and cross sectional area values, subsurface inflow value was calculated as 7.53 l/s.

The match was obtained by using the hydraulic parameters listed in Table 5.1. The model results indicated an acceptable match between the field and simulated groundwater levels (Figure 5.3).

After running the steady-state simulation model, maximum and minimum differences between simulated and observed heads, mean errors, mean absolute errors and root mean squared errors were computed for each model run by using some simple Fortran programs. Error statistics for the calibrated groundwater model is as follows:

Mean Error: 0.09 m

Mean Absolute Error: 4.80 m

Root Mean Square Error: 6.52 m

As a second calibration process, existing pit lake was simulated. The groundwater model was used to predict the inflow and outflow rates for different lake stages. For each stage, a separate spreadsheet was used to do a water balance with time that includes groundwater flow, surface flow, and direct precipitation and evaporation components. The pit lake simulating spreadsheet works like a reservoir model. Further information for pit lake simulations can be found in the pit lake flooding section.

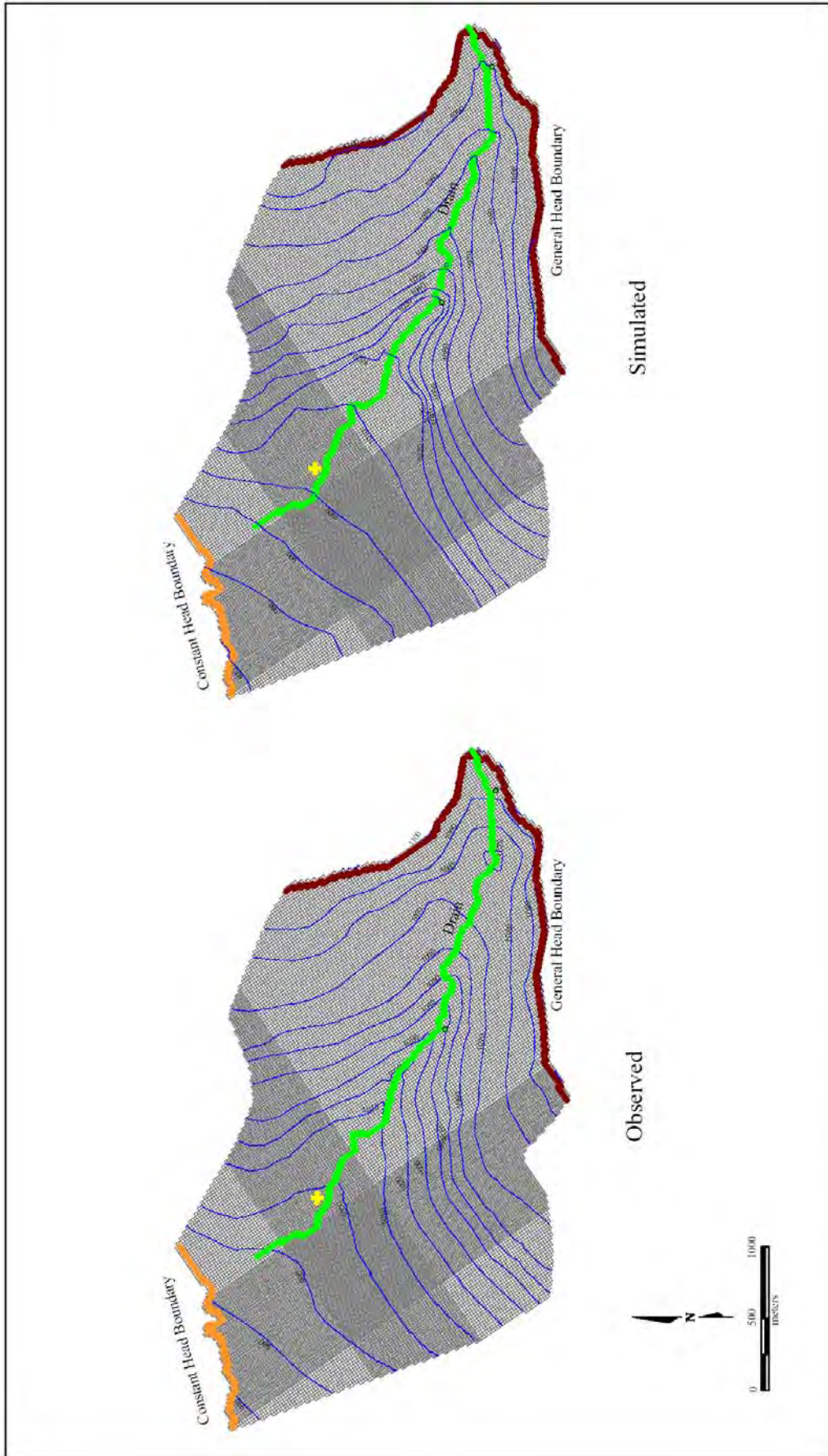


Figure 5.3. Observed vs Simulated Groundwater Levels

Table 5.1: Calibrated Hydraulic Conductivity Values for Groundwater Systems

		Kx (m/s)	Ky (m/s)	Kz (m/s)
Shallow System	Alluvium	4.E-05	4.E-05	1.6.E-05
	Weathered Marn	3.E-06	3.E-06	3.E-07
	Weathered	3.24E-07	3.24E-07	3.24E-08
	Basement	3.24E-08	3.24E-08	3.24E-09
Deep System	Marn	2.E-07	2.E-07	2.E-08
	Basement	5.E-09	5.E-09	5.E-10

Pit lake model calibration was completed with adjusting the pit wall runoff constant and actual evaporation rate from the pit lake. These simulations indicate that the existing pit lake would stabilize in 992 masl which is actually observed at the site. Therefore, the model assumed to be simulating the site conditions with acceptable accuracy. The existing pit lake is a flow through lake and the flow components are estimated to be as follows:

Groundwater inflow: 14,783 m³/yr

Groundwater outflow: 14,063 m³/yr

Surface water runoff and Direct Precipitation: 18,148 m³/yr

Evaporation: 18,761 m³/yr

CHAPTER VI

DEWATERING DESIGN

6.1 Dewatering Objectives

Because the marl quarry will be operated below the water table, dewatering will be needed to provide a safe working environment. To allow successful progression of the quarrying schedule the groundwater system will need to be dewatered by 30 m. The primary objectives of the planned pit dewatering system are as follows:

- access the marl resources below the existing water table,
- enhance the stability of the pit slopes,
- minimize the potential for any hazard to both operational personnel

and mining equipment

6.2 Dewatering Options

The methods of dewatering depend on site access, type of operation, required depth of dewatering, and aquifer/aquitard conditions. Several methods can be utilized to dewater a pit. In general, the dewatering of a pit can be achieved by dewatering wells, drain holes specifically designed for dewatering and/or seepage of the groundwater flow into the pit workings. The following methods were assessed for dewatering of the AST marl quarry:

- Vertical Wells
- Horizontal Wells
- Vertical Sumps
- Horizontal Trenches

Vertical Wells: The permeability of the marls is very low. At these permeability values large number of wells is needed to lower the groundwater table below the operating limits. Since the installation and operation costs for this system are very expensive, vertical well method was found as not applicable to the site.

Horizontal wells/ drain holes: Another method for dewatering and depressurizing of a pit wall is to drill horizontal wells/ drain holes to the wall. By this method the accumulated water can be removed from the system before it comes to the pit face. However low permeable nature of marl causes small dewatering discharge rates for horizontal wells. Furthermore, the construction

costs are reasonably high because the wells should be reconstructed with the development of open pit.

Vertical Sumps: The sump system can be applied by constructing a number of sumps in the pit and pump out the groundwater from the sumps. This system was checked for applicability by model simulations. The simulation results have shown that the sump system would not be sufficient to dewater the site. As it was stated before, the hydraulic conductivity of the material is not sufficient enough to create a cone of depression and dewater the area by using point sinks

First simulation was run with 6 sumps located 5 meters below the existing water table and it has seen that the sumps are not sufficient enough to dewater the pit. Five more simulations were run with increasing the number of the sumps for each run. Sixth simulation has shown that the pit cannot be dewatered even with 60 sumps located around the perimeter of the pit. The simulated head distribution for the sump solution is shown in Figure 6.1.

Horizontal Trenches: This system is composed of dewatering trenches that are constructed in the pit floor. Horizontal trench dewatering system is applicable to the marl quarry because it is effective to dewater the large areas for low permeability systems. Furthermore it is easy to construct and the excavated material for its construction is still a part of the quarrying operations and can be processed. The detailed information about the system is given in the next section.

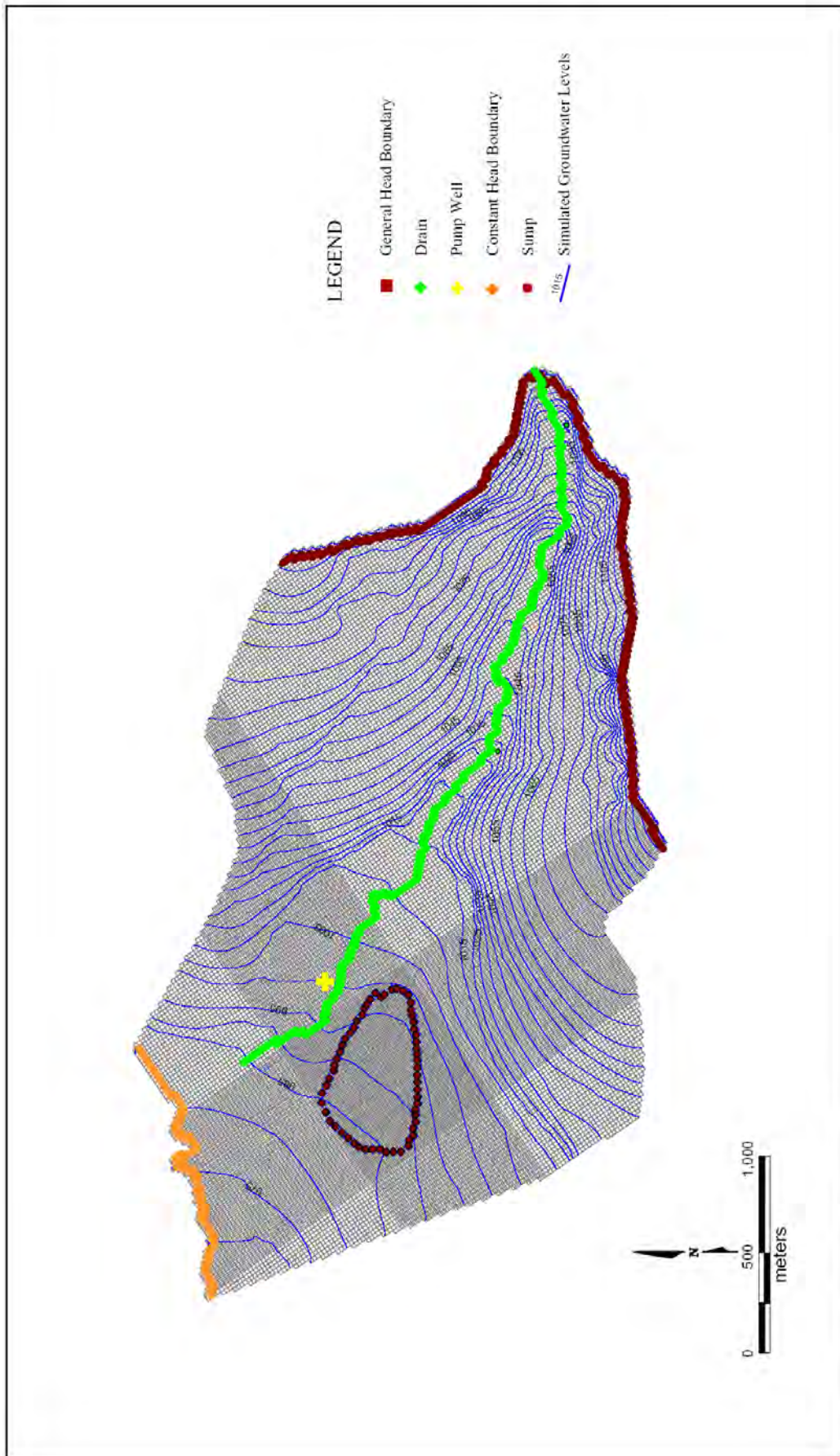


Figure 6.1. Sump Simulation - 10th Year

6.3 Proposed System and Dewatering Simulations

The proposed dewatering system for the pit consists of stage-wise construction of trenches and three sumps on the western edge of the pit. This system will be installed by excavating trenches at the perimeter of the pit. The trenches will be constructed with a slope to a main collector sump. Trenches will be constructed with the excavation of the material which lies in the pit and there will not be any protective or cover layer for the trenches. The trenches need extra attention because of the vegetation which can grow on trenches. The existing pit lake located on the western edge of the quarry can be dewatered, reshaped and can be used as the main collector sump of the trench-dewatering system.

In the absence of plans for operating the quarry in the future, during model simulations it has been assumed that one meter thickness of material would be mined out uniformly from the pit on a yearly basis. The model simulations were made as three integrated runs with the following scenario;

For the first run, starting from existing water level conditions, groundwater levels were lowered with a rate of 1 m/yr with using the time variant specified heads (TVSH) option of MODFLOW. In TVSH package, two values are assigned per stress period: a value at the beginning of the stress period and a value at the end of the stress period. This makes it possible to specify a piece-wise linear time series. TVSH is the most applicable option to simulate the stage wise construction of trenches. At the end of 10th year it has observed that the pit was dewatered for the first model layer. Groundwater levels were lowered to 980m at the west and

990m at the east. Simulation ended at the end of the 10th year and another simulation was made for the next 10 years.

After dewatering the first layer, second layer dewatering simulations were carried out with the same procedure. Since the groundwater levels were decreased to the bottom of the first layer in the previous simulation, starting heads for the TVSH (trench system) was introduced as the top elevation of the second layer. Accordingly, the final head distribution of the first simulation was introduced as the starting head for the second run. Also, the cells in the first layer that has been lying in the dewatered section of the pit were inactivated. Furthermore constant heads were assigned to the cells, which are lying at the crest of the pit. The constant head values were assigned as the bottom of the first layer to simulate the real conditions of dewatering. The model was run for the second 10 years with the same layout of the trenches. It has seen that the floor of the pit was not dewatered at the end of the 20th year. This is due to the increase in the hydraulic gradient and the recharge from the third layer. The model was run again with a different configuration of the dewatering system. Additional trenches were introduced to the model for dewatering the pit bottom and it has seen that the pit floor has been dewatered at the end of the 20th year with this new configuration.

Third simulation was made with the same configuration and procedure as the second simulation and dewatering simulations have been completed successfully with these three simulations. Figures 6.2, 6.3 and 6.4 shows the simulated head distribution with the trench system for 10th, 20th, and 30th years respectively.

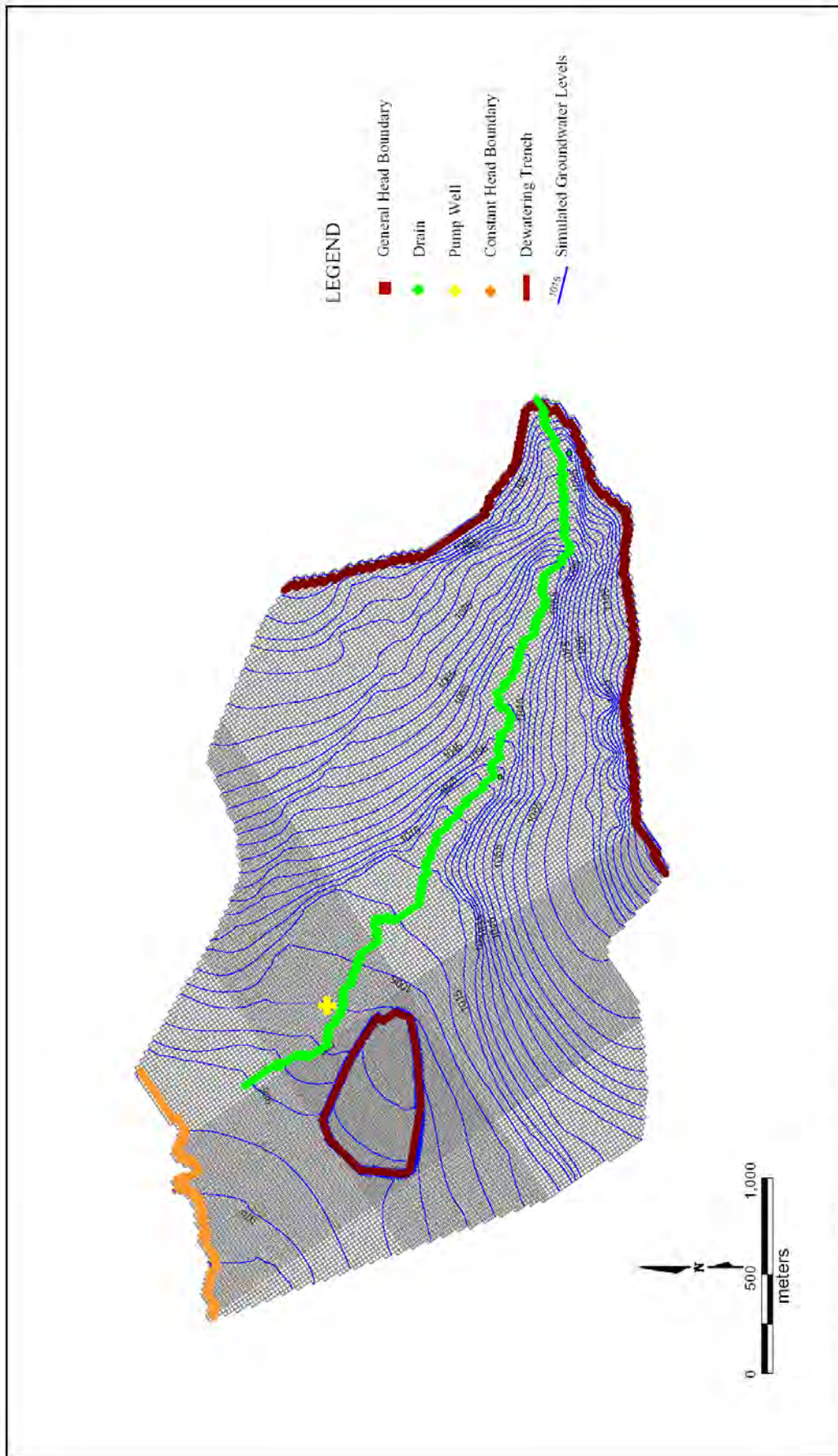


Figure 6.2. Trench Simulation - 10th Year - Layer 1

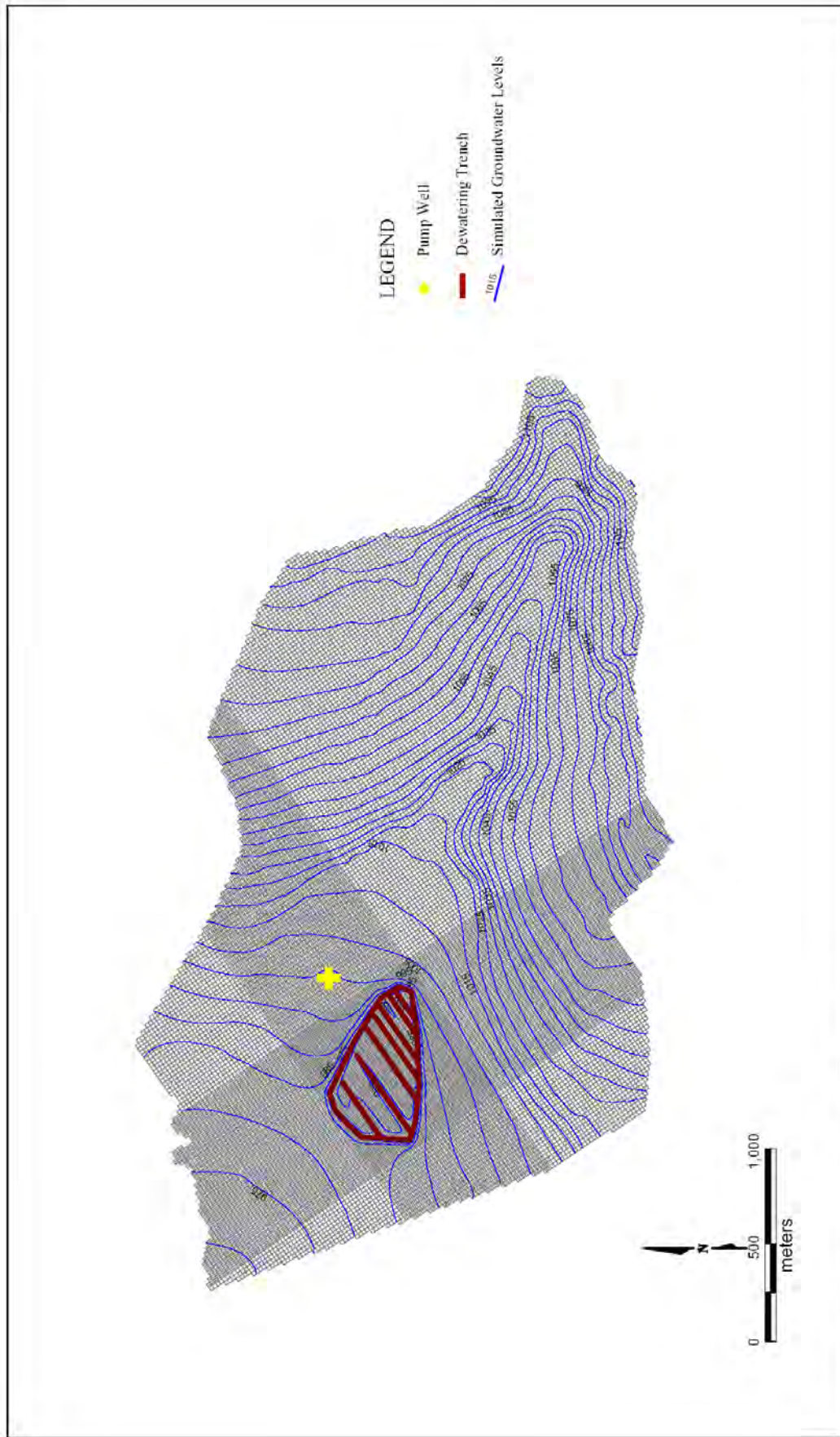


Figure 6.3. Trench Simulation - 20th Year - Layer 2

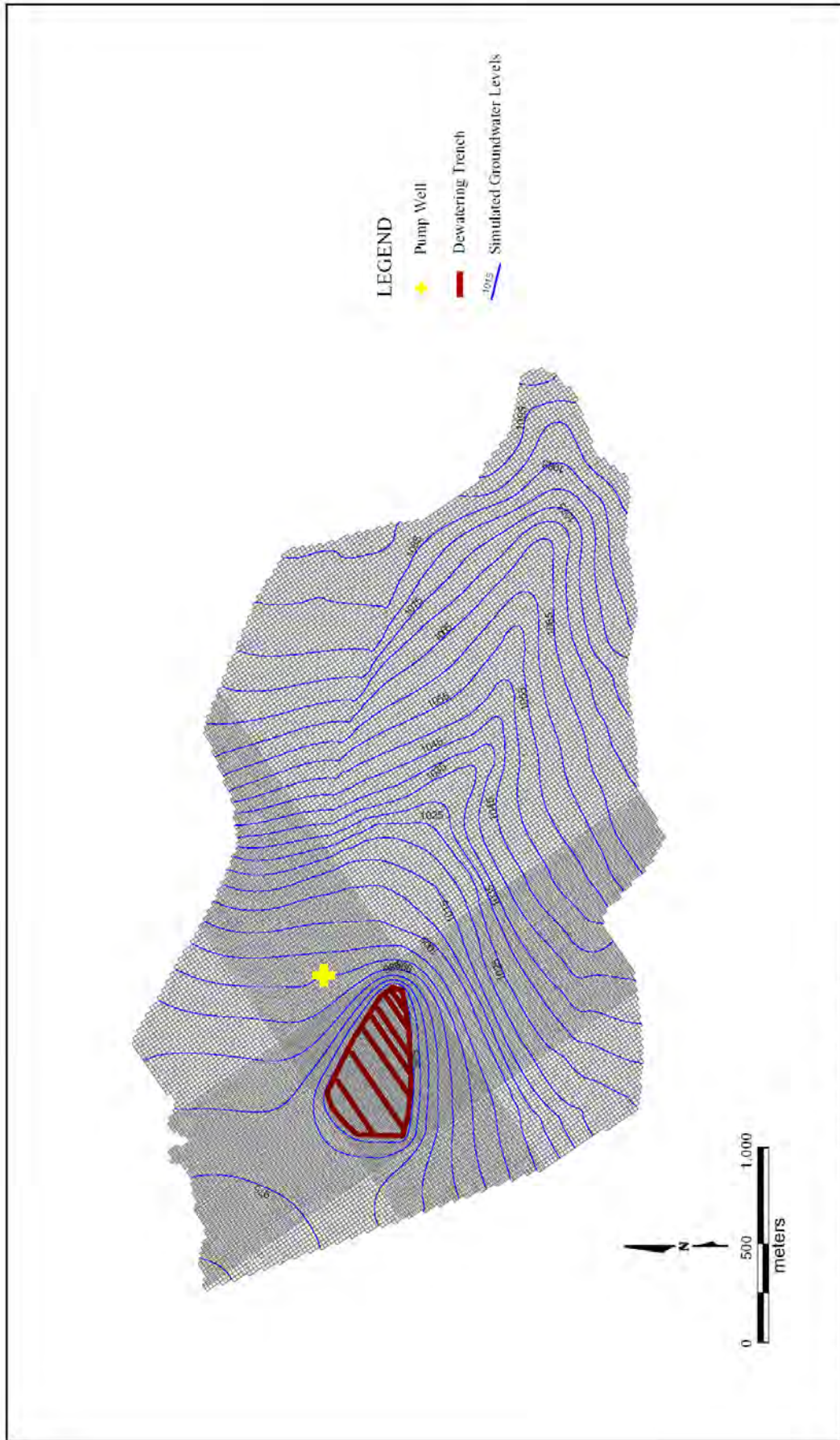


Figure 6.4. Trench Simulation- 30th Year- Layer 3

Discharge rates from the dewatering trenches were obtained from the flow budget option of the MODFLOW. Figure 6.5 shows the cumulative water production from the trench-dewatering system. The average quantity of groundwater dewatered is 3.7 l/s or 116,690 m³/y.

6.4 Dewatering Trench Details

Design and construction of dewatering trenches should be made accordingly with the nature of the operation. The proposed dewatering system is fairly simple to construct and installation is inexpensive.

The construction of trenches is the most important item in the dewatering operations. These trenches should be constructed accordingly to achieve a successful dewatering scheme. Technical details for the trench dewatering system can be summarized as follows;

- The trenches should be constructed with a specified inclination to develop a gravity flow system. Since the groundwater flow direction is from east to west, the trenches should have an inclination towards west. Proposed inclination of the trenches would be 1%.
- The pit wall side of the trenches should be constructed with the proposed slope for pit walls for final closure design and the quarry side of the trenches should be constructed with an angle of 20⁰. Constructing the quarry side of the trenches with 20⁰ slope angle would make the re-construction and deepening of trenches easier.

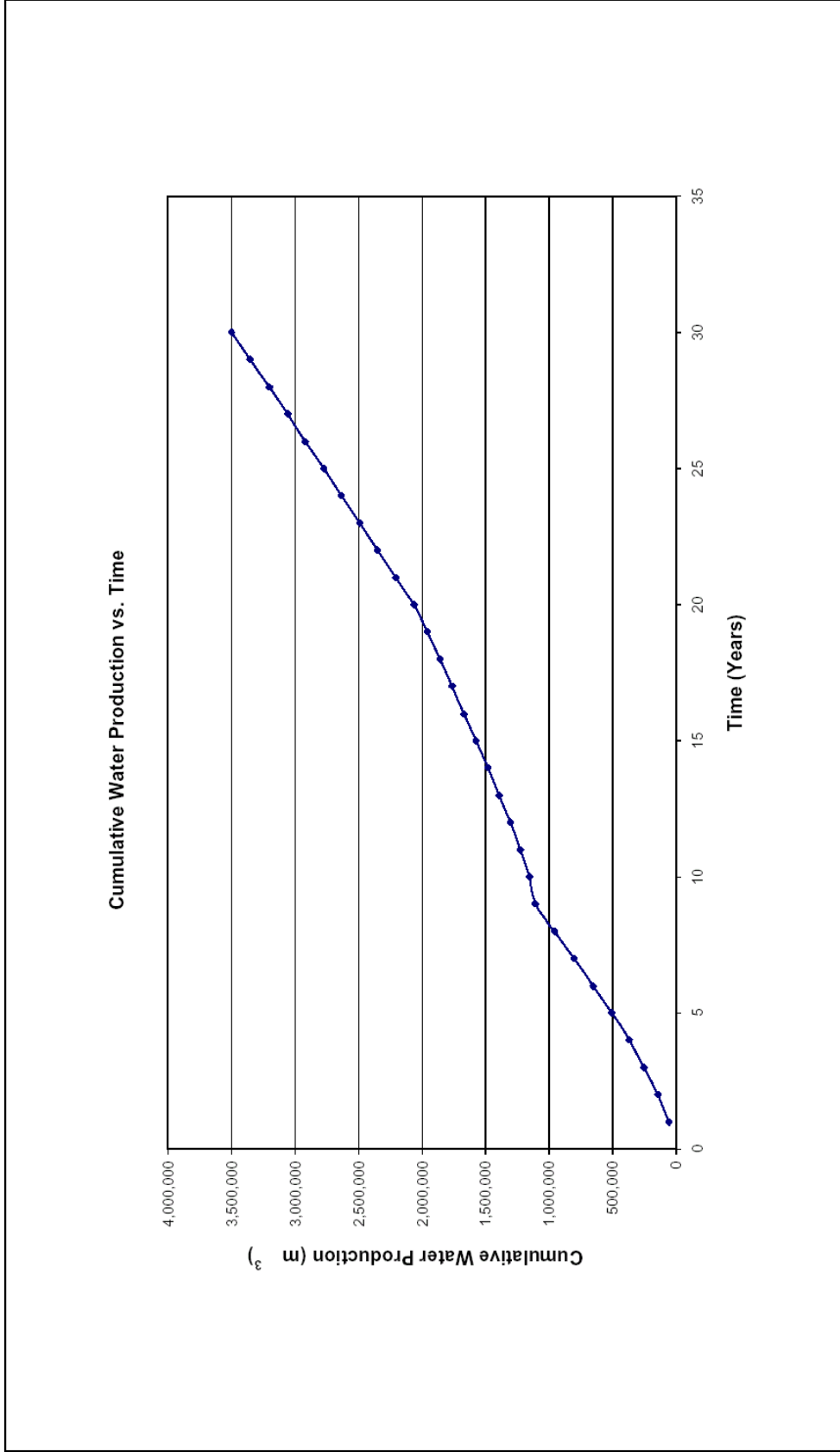


Figure 6.5. Cumulative Water Production.

- The bottom of the trenches should be constructed 5 meters below the desired working elevation and the bottom width of the trenches should be 1 meters.
- As the material mined out and the pit dewatered, the hydraulic gradient will increase. This may cause seepage of groundwater at the pit floor. AST should construct additional trenches to the pit floor in the center of the pit to remove this water from the material. It is predicted that the need for additional trenches will show up within 10 years after starting the dewatering operation. These trenches should be constructed with an inclination of 0.5% and with 50 meters separation. The dimensioning of the trenches should be the same as the main trenching system. The construction details for the dewatering trench system are given in Figure 6.6.
- The discharge from the trenches would be collected in the main sump on the west side of the pit. The volume of the main sump should be at least 20,000 m³.
- In addition to the main sump two small secondary sumps, each having a volume of 5,000 m³ should be constructed on the right and left sides of the main sump. The construction of these secondary sumps is important to collect the sediment load that can be caused by the active mining. The sediment load from the dewatering system is not a well known issue for this stage. However during the dewatering operations the sediment load

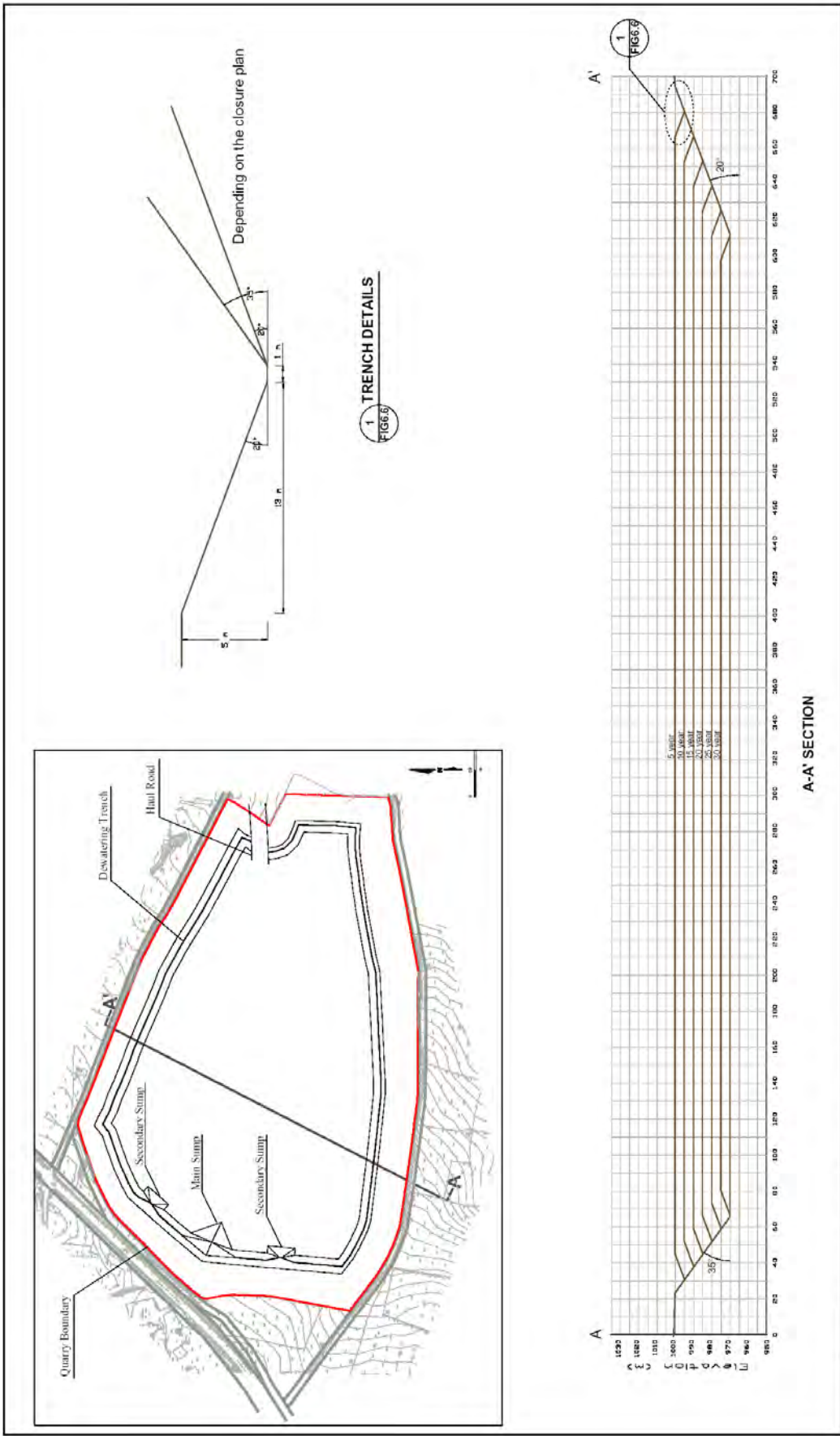


Figure 6.6 Dewatering Trench System Details

should be checked and if it is necessary, additional deposition sumps should be constructed. The secondary sumps can also be used to collect water, during the deepening of the main sump.

- Trenches will be excavated in the marn and there will not be any protective layer of material on the trenches. This may cause some vegetation problems during the spring and the summer season. If the vegetation grows rapidly and causes some problem with the flow of water, these vegetation should be removed from the trenches. Another way to stop the vegetation in the trenches is to lay gravelly material in the trenches.

Dewatering system layout, trench and sump details are given in Figures 6.7 and 6.8 for starting conditions and at the end of 10th year, respectively.

6.5 Surface Water Management

The simulation results have shown that the maximum water inflow to the pit during the dewatering operations will be about 4.9 l/s. There will be an additional input from direct precipitation to the pit dewatering trenches. The maximum monthly precipitation rate in the area is 79.6 mm in December. This will cause an increase in the discharge from the trenches. Expected average inflow from direct precipitation is about 12 l/s for the month of December. As a

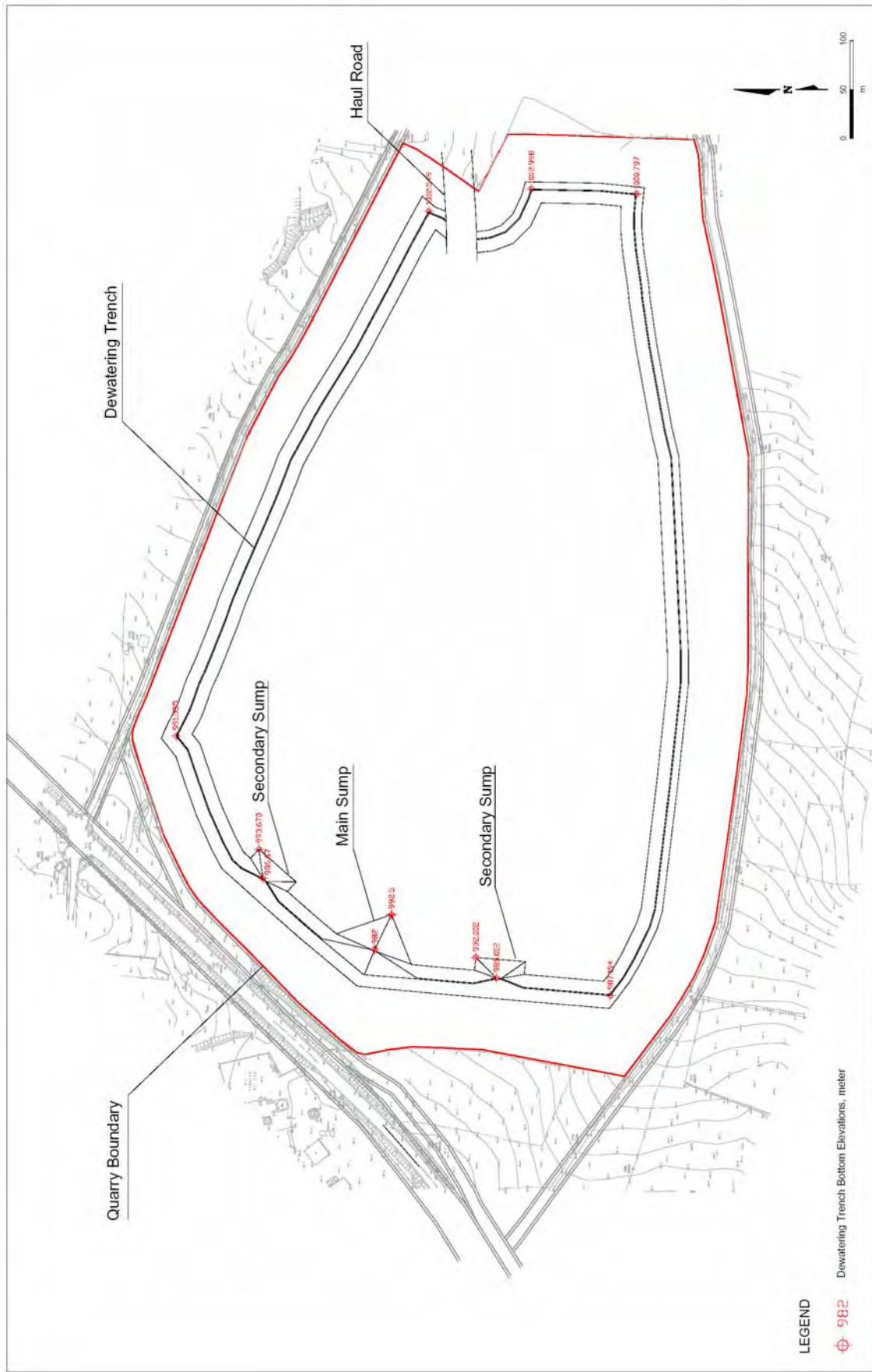


Figure 6.7 Starting Layout of Dewatering Trench

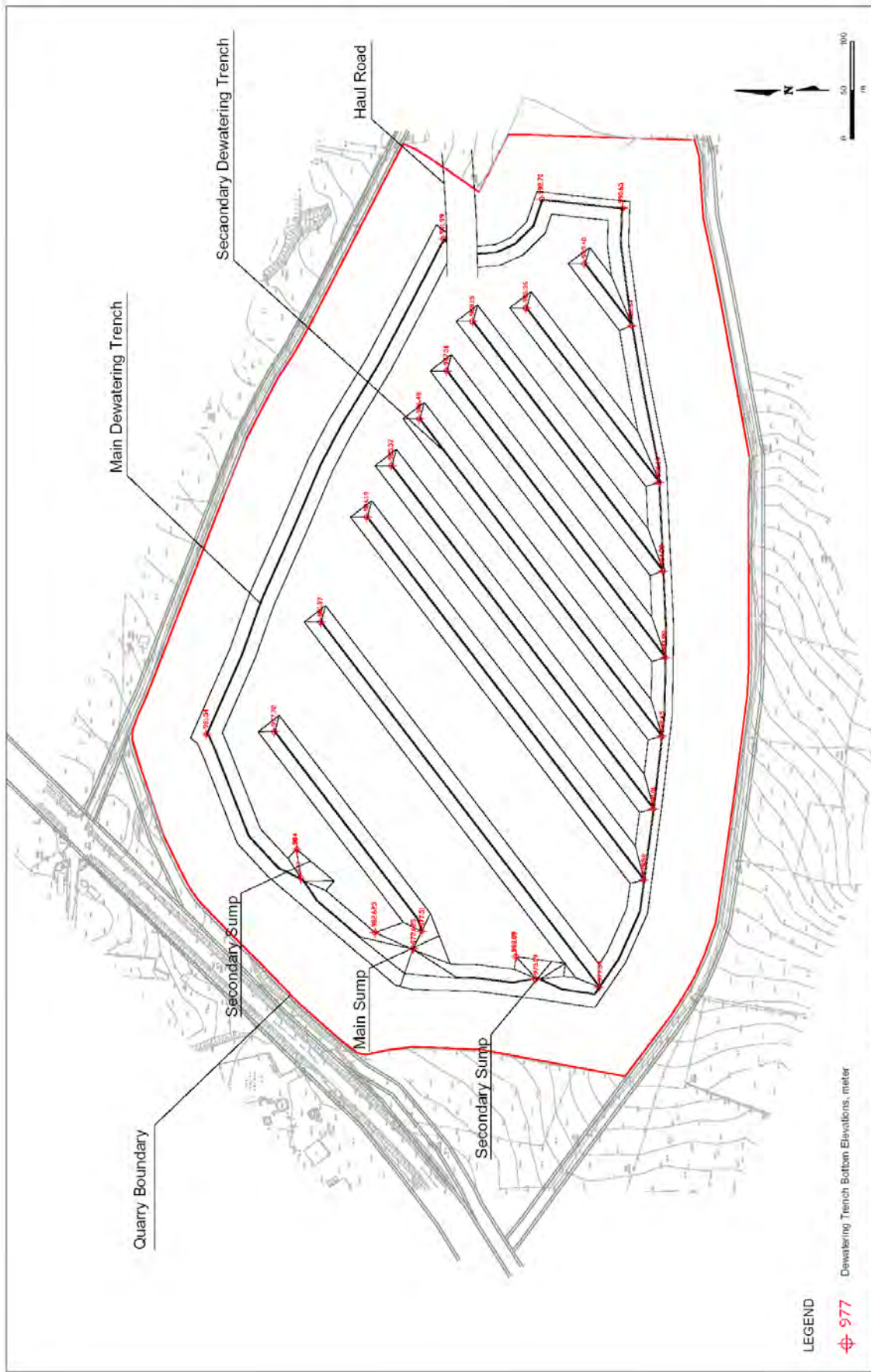


Figure 6.8 Starting Layout of Secondary Dewatering Trenches

result, the capacity of the discharge pumps should be selected by considering these two discharge values.

AST should also be prepared to the extreme storm events. Based on the climatic data from the weather station and watershed characteristics, peak discharge was estimated at Rehabilitation Report prepared for the marl quarry. Rainfall depth for a 100-year 24-hour storm event has been estimated as 69.87 mm. This amount of direct precipitation will produce a 310 l/s of discharge into the pit. Therefore, the collection sumps were also designed to store this much of water (including the dewatering inflow) for 24 hours without pumping.

The dewatering pump system should have a normal working capacity of 20-25 l/s. In addition, AST should have high capacity pumps to discharge the flow that will result from a 100-year 24-hour storm event. Furthermore, some structures should be constructed to remove the surface run off to the open pit from the basin. As it was described in detail in the closure plan of the quarry 2 m thick earthberms should be constructed on the East and South crests of the pit to divert the surface runoff water from the upstream side of the basin.

The pit floor should also be considered in the management of surface water to minimize the accumulation of precipitation on the pit floor and subsequent recharge to groundwater. Pit floor should be mined with an inclination towards the dewatering trenches to minimize the accumulation of rainwater.

6.6 Existing Pit Lake Dewatering

The existing pit lake should be dewatered in order to have access to the marl resources below the lake bottom and to construct the trench dewatering system. Two analyses were made for different seasons with the spreadsheet solution. In the first simulation, dewatering started with the beginning of the June with a discharge rate of 40 l/s (3456m³/d). According to the spreadsheet solution dewatering of the existing pit lake would be completed within 32 days. In this simulation the meteorology data of June and August were used to simulate the dry seasons meteorological conditions.

The second simulation was done for the wet season. Dewatering started with the beginning of December with the same discharge rate and was completed in 34 days. Also for this simulation the long term average meteorological data for December and January were used. The expected total inflows and outflows, total discharged water and predicted dewatering periods, during the pit lake dewatering are calculated for different seasons dewatering scheme and summarized in Table 6.1.

The pit lake could be dewatered with high capacity pumps. Discharge from the pumps should be diverted to the downstream of the basin to reduce the recharge to groundwater. The best receiving environment for the discharge of this water would be the Skz Stream. The water quality criteria for discharging the existing pit lake water to the Skz Stream are discussed in Section 6.9.

Table 6.1: Summary of Total Inflows and Outflows during the Pit Lake

Dewatering

	Dewatering Discharge Rate (lt/sec)	Dewatering Period (days)	Inflow (m ³)		Total Inflow (m ³)	Outflow (m ³)		Total Outflow (m ³)
			Groundwater Inflow (absolute)	Inflow from precipitation		Evaporation	Dewatering	
DRY Season	40	32	1966	484	2450	2270	111680	113950
WET Season	40	34	2000	3240	5240	0	116740	116740

It would be better to start the dewatering in dry season in order to save energy and time. Furthermore, working in dry season would minimize the recharge to the pit lake from precipitation and maximize the evaporation from the lake surface. However, dewatering in dry season would disturb the frogs in the vicinity of the pit lake and may cause the frogs to jump on the highway and produce slippery road conditions. Therefore, it is safer to conduct the dewatering of the existing pit lake in winter season.

6.7 Dewatering Performance Monitoring

The performance of the dewatering system should be monitored in order to achieve a successful dewatering. It will include the collection of data pertaining to water quantity and quality monitoring and groundwater level monitoring.

6.7.1 Discharge water monitoring

The quantity of the discharged water should be monitored and recorded on a daily basis to check the performance of the dewatering system. The quantity monitoring can be done with an inline flowmeter connected to the water discharge pipe from the main sump.

The quality of discharging water from the system should be monitored on a monthly basis. The parameters according to the related regulation (10/03/1995-22223-Official Gazette), which should be monitored, are listed in Table 6.2.

6.7.2 Groundwater Level Monitoring

AST should drill two groundwater level monitoring wells to check the dewatering system performance. One well should be drilled on the downstream and the other should be drilled to the upstream of the quarry. Wells should be completed with a PVC casing to a depth of 60 m. Groundwater levels should be monitored on a weekly basis to check the performance of the dewatering operations. The recommended locations of groundwater monitoring wells and the expected groundwater level changes for these wells during 30 years of dewatering operations is shown in Figure 6.9. Construction details for the groundwater monitoring wells are shown in Appendix D.

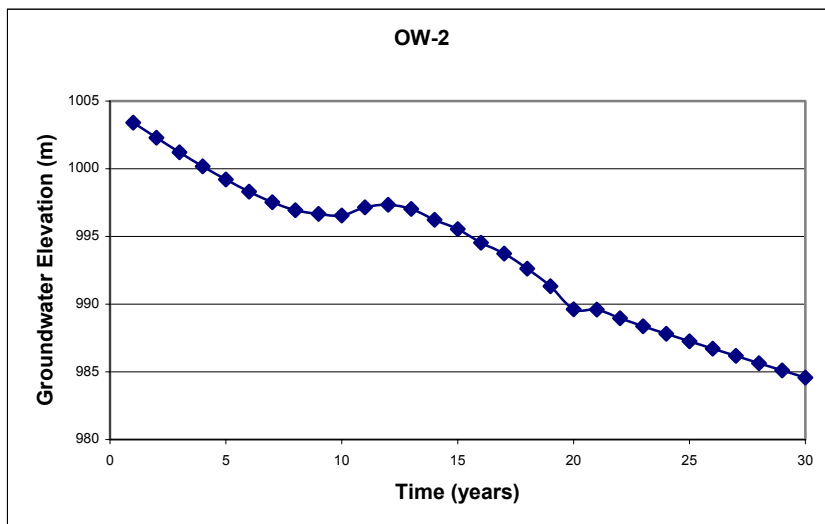
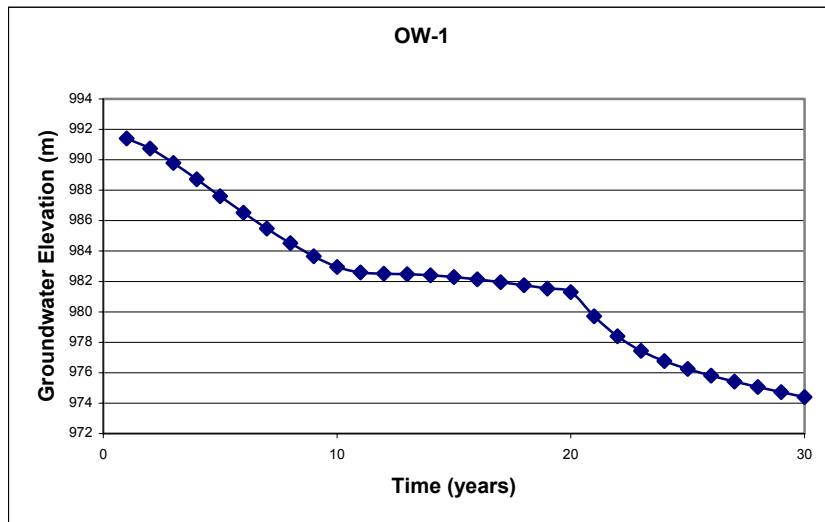
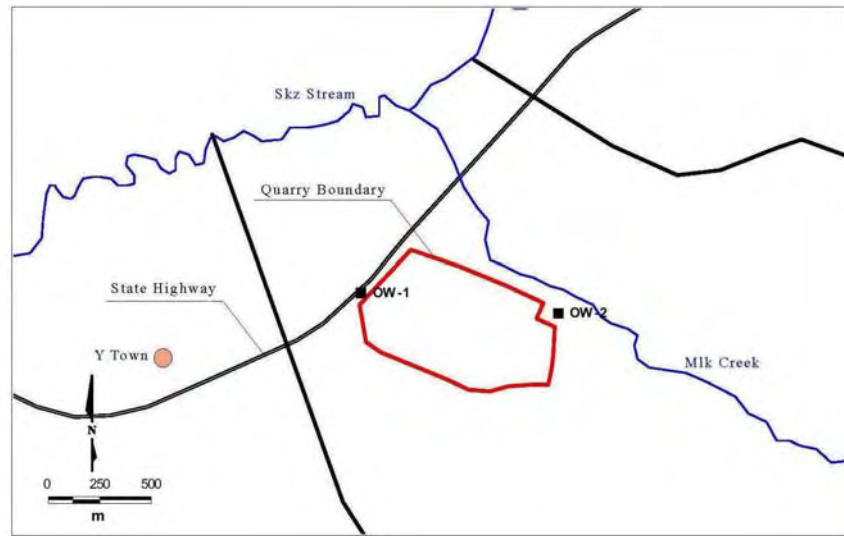


Figure 6.9. Proposed Locations of Performance Monitoring Wells and Predicted Decrease in the Groundwater Levels.

Table 6.2: Discharge Parameters and Acceptable Values for discharge.

Parameter	Acceptable value for discharge (mg/l)
BOD	50
COD	170
TSS	200
Oil and grease	10
Phenols	5
Free Cyanide	0.06
Total Cyanide	0.3
Free Chlorine	0.5
Total Sulphur	1
Nitrate-N	5
pH	5--9
Total Phosporus	0.02
Ammonium-N	0.2
Flourine	20
Mercury	0.01
Cadmium	0.05
Lead	0.5
Arsenic	0.5
Cromium (Total)	0.5
Copper	0.5
Nickel	0.5
Zinc	2

6.8 Predicted Water Balance

The water balance for the dewatering simulations was prepared to assess the changes in the overall groundwater regime from existing conditions. The predicted groundwater budget for the 30-year simulation period is given in Table 6.3. In the table total inflow was calculated from subsurface inflow and recharge values. According to the predicted budget, total pumped volume from the dewatering system is varying between 1.35 l/s and 4.9 l/s and this discharge from the groundwater system causes some decrease in the discharge rates of other groundwater features.

The most significant changes are observed in the change in base-flow to the Skz Stream values. Baseflow will decrease from 12.2 l/s to 8.1 l/s. Also there will be change in the groundwater reserves as the system is not under steady-state conditions. However, this change is insignificant as compared to the other components of the water balance. The discharge rates of the spring will decrease 3-6 % at the end of thirty years.

6.9 Discharge Options

The extracted water from the dewatering system can be disposed in several ways. Most convenient options are:

- Use as drinking or irrigation water
- Discharge to Skz Stream

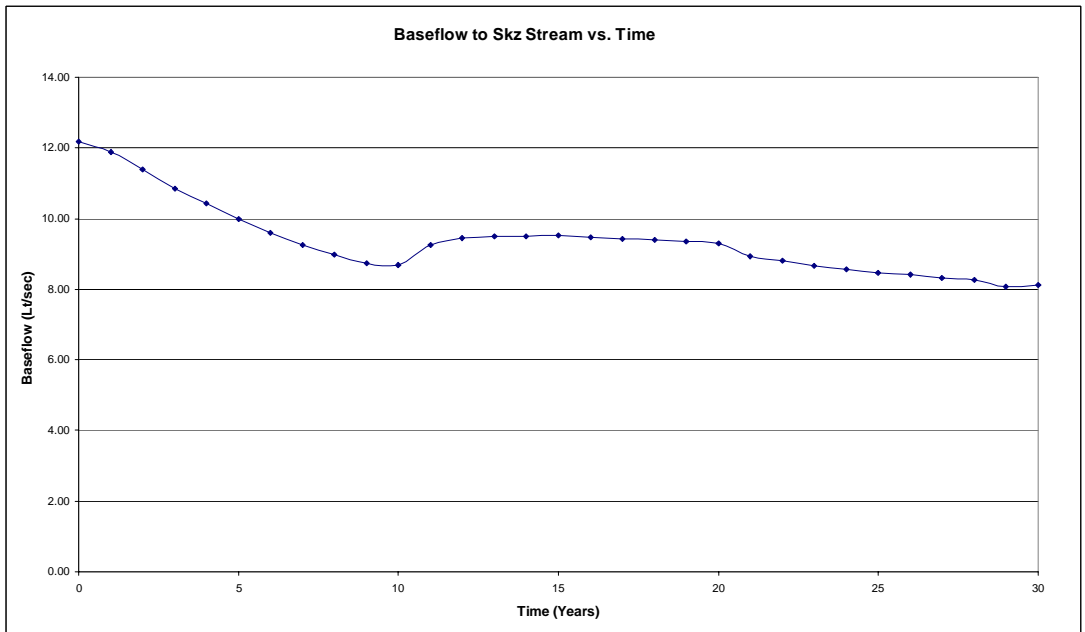
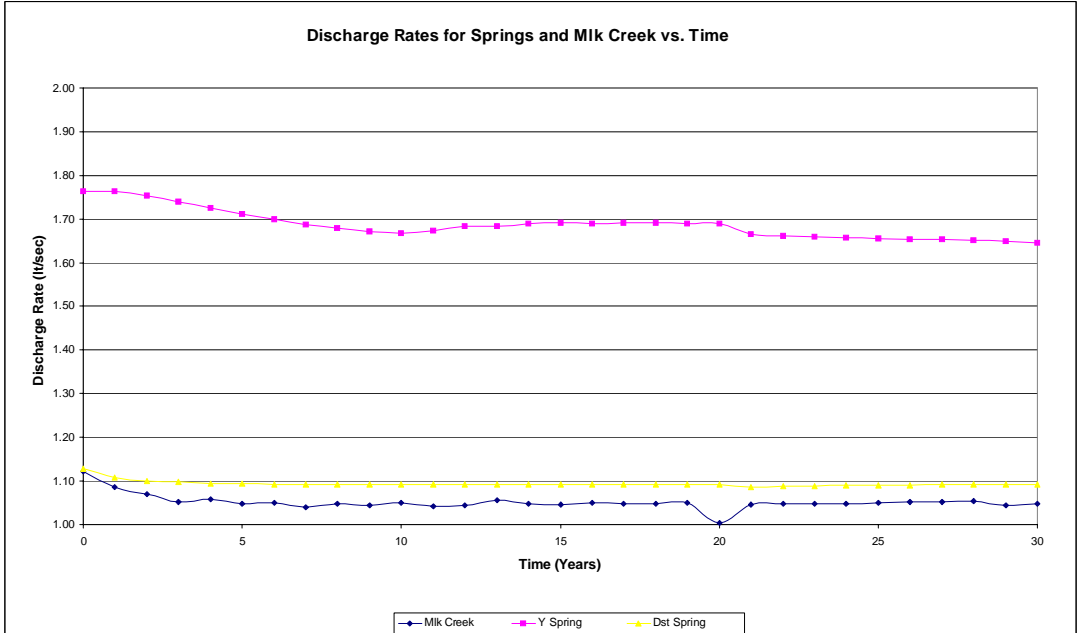


Figure 7.1. Decrease in Groundwater Levels

- Use as process water

Use as drinking or irrigation water: Water from the dewatering system can be used for drinking or irrigation purposes based on its quality. The results of chemical analyses were compared with the drinking and irrigation water quality standards. The detected concentrations of boron, calcium, EC, magnesium, sodium and sulphate exceed the drinking water quality standards specified by the European Union, The Ministry of Health, and World Health Organization. Also the electrical conductivity (EC) value and sulphate concentration exceed the irrigation water quality standards. These results have shown that the water from the dewatering system cannot be used for drinking or irrigation purposes (Table 6.4).

Discharge to the Skz Stream: The discharge rate from the dewatering system is not expected to be more than 5 l/s. Discharge of this much water to the Skz Stream would minimize the impact of dewatering on the Skz Stream, by making up the reduced base-flow quantities. A new pipeline should be constructed to discharge the water to the Skz Stream. Discharge end of the pipeline should be buried below the stream bed, discharge should be diverted to downstream and pipeline in the stream bed should be covered with gravel to minimize turbidity effect. The most important item in this option is the quality of the discharged water. Discharge limits according to the related regulation (10/03/1995-22223-Official Gazette) have to be followed to discharge the water directly to the Skz Stream. During the field studies groundwater from the pump

well was sampled and analyzed. The required limits (according to the regulation) and the analyzed results are shown in Table 6.4. It should be pointed out that the water sample from the groundwater well is filtrated as it flows through the surrounding gravel pack, a situation that will not occur in dewatering trenches. Consequently, the quality of the discharged water from dewatering trenches may be quite different than what is presented in Table 6.4.

Use as process water: Currently, AST obtains water pumped from a well drilled at the plant site and from two wells on the north side of the watershed. The average discharge rate for the well at the plant site is 0.2 l/sec. AST can utilize the water from the dewatering operations in the plant.

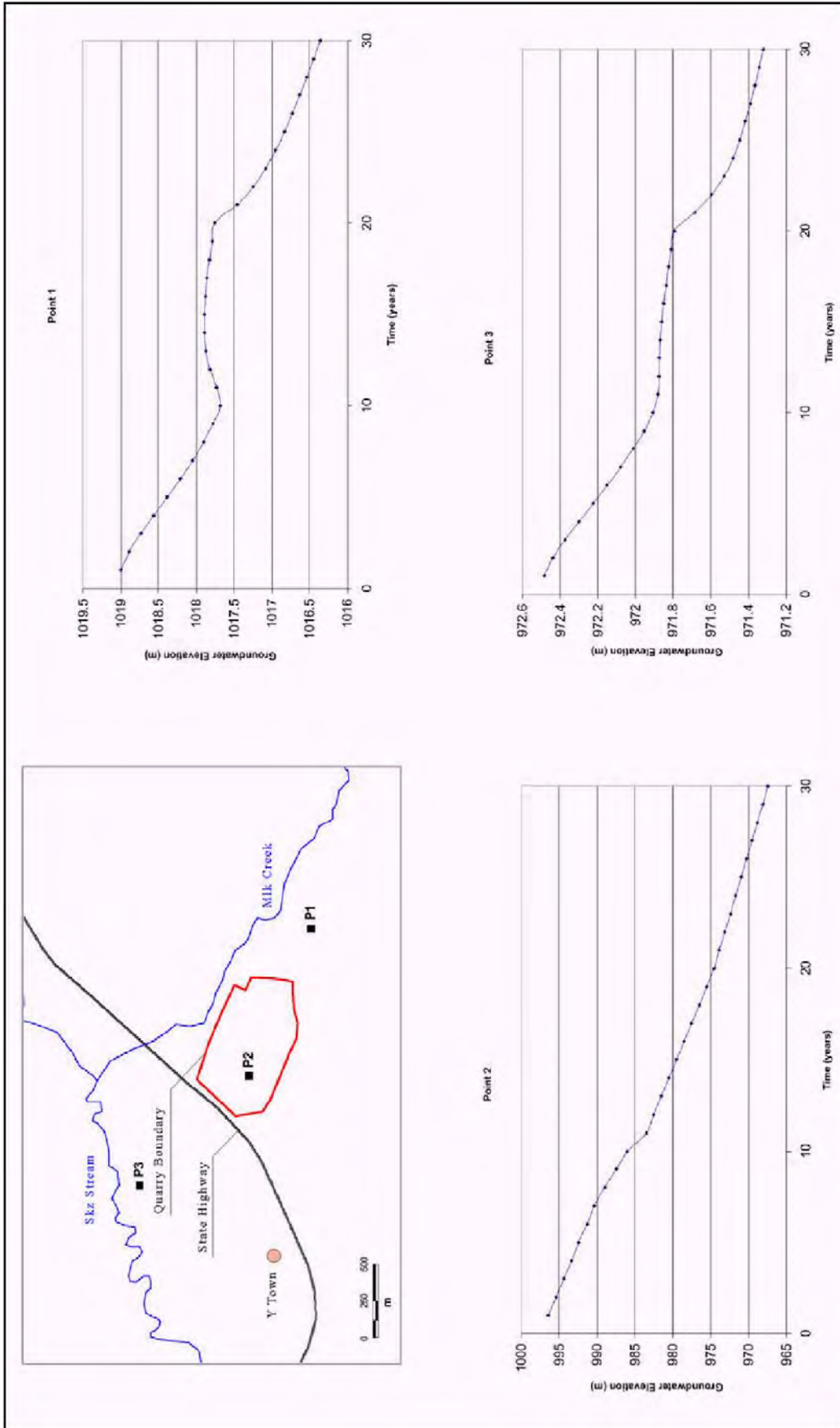


Figure 7.2. Decrease in Groundwater Levels

CHAPTER VII

DEWATERING IMPACTS

Groundwater modeling has indicated that pit dewatering will have an impact on water resources in the vicinity of the pit. The continuous dewatering from the system will cause a decrease in the groundwater levels. The examination of the simulated water levels during the first year of operation of the dewatering scheme supports this conclusion. However, because the discharge rate is small relative to the catchment recharge rate, the dewatering scheme is likely to have little impact beyond the immediate vicinity.

The primary issues that should be considered during the analysis of dewatering impacts on the other resources include:

- Baseflow to Skz Stream
- Major Springs located in the water catchment
- Groundwater levels.

Baseflow to Skz Stream: The most significant impact of dewatering is the expected decrease in baseflow rate to the Skz Stream. As it was mentioned before, Skz Stream is located at the downstream part of the pit. Since the dewatering will discharge water from the upstream part of the system, there will be a decrease in the baseflow. As it can be seen from the predicted groundwater budget, the maximum decrease will be about 30% (4 l/s). Figure 7.1 shows the predicted decrease in the baseflow to Skz Stream during 30 years of dewatering. This can be considered as an insignificant impact when it is compared to the flowrate of Skz Stream. Because water from dewatering trenches will be discharged into Skz Stream, the impact would be minimized in the quantity wise. However, quality of the water in the Skz Stream may have been impacted.

Major Springs located in the water catchment: There are two major springs located in the catchment area and both are located at the upstream of the dewatering area. Y spring is located closer to the pit area. From the predicted groundwater budget it can be concluded that there will be a 6% decrease in the flowrate of the spring at the end of 30 years. Similarly there will be a 3% decrease in the flowrate of Dst Spring (Figure 7.1). The decrease in the flowrates of these springs can be assumed as insignificant when seasonal effects are considered.

Groundwater levels: There will be a decrease in the groundwater levels with the beginning of the dewatering. It is predicted from the model simulations that the dewatering system will cause a cone of depression that extends almost to the borders of the modelled area. Drawdown contour of 0.5m extends to 600 m (to Skz Stream) on the west and approximately 1800m to the east. During the

Table 6.3: Predicted Water Balance

Year	Total In		Total	Total Out							Total	Groundwater Reserv Change
	Subsurface Inflow	Recharge		AST Well	Milk Creek Y Spring	Dst Spring	Baseflow to Skz Stream	Dewatering System	ET			
0	6.17	11.27	17.44	0.17	1.12	1.76	1.13	12.19	0.00	0.32	16.70	-0.74
1	6.52	11.27	17.79	0.17	1.09	1.76	1.11	11.89	1.86	0.94	18.82	1.03
2	6.55	11.27	17.82	0.17	1.07	1.75	1.10	11.40	2.70	0.91	19.11	1.29
3	6.56	11.27	17.84	0.17	1.05	1.74	1.10	10.86	3.40	0.89	19.21	1.38
4	6.57	11.27	17.85	0.17	1.06	1.72	1.09	10.43	3.87	0.88	19.24	1.39
5	6.57	11.27	17.85	0.17	1.05	1.71	1.09	9.99	4.28	0.88	19.18	1.33
6	6.59	11.27	17.86	0.17	1.05	1.70	1.09	9.59	4.59	0.87	19.07	1.22
7	6.59	11.27	17.86	0.17	1.04	1.69	1.09	9.25	4.80	0.87	18.91	1.05
8	6.60	11.27	17.87	0.17	1.05	1.68	1.09	8.97	4.87	0.87	18.70	0.83
9	6.60	11.27	17.87	0.17	1.04	1.67	1.09	8.73	4.77	0.86	18.34	0.47
10	6.60	11.27	17.87	0.17	1.05	1.67	1.09	8.69	1.35	0.86	14.88	-2.99
11	6.60	11.23	17.82	0.17	1.04	1.67	1.09	9.25	2.42	0.86	16.51	-1.32
12	6.60	11.23	17.82	0.17	1.04	1.68	1.09	9.44	2.58	0.86	16.88	-0.95
13	6.60	11.23	17.82	0.17	1.06	1.68	1.09	9.49	2.71	0.86	17.07	-0.76
14	6.60	11.23	17.82	0.17	1.05	1.69	1.09	9.49	2.80	0.86	17.16	-0.67
15	6.60	11.23	17.82	0.17	1.05	1.69	1.09	9.52	2.87	0.86	17.25	-0.57
16	6.60	11.23	17.82	0.17	1.05	1.69	1.09	9.48	2.95	0.86	17.30	-0.53
17	6.60	11.23	17.82	0.17	1.05	1.69	1.09	9.43	3.03	0.86	17.33	-0.49
18	6.60	11.23	17.82	0.17	1.05	1.69	1.09	9.40	3.09	0.86	17.35	-0.47
19	6.60	11.23	17.82	0.17	1.05	1.69	1.09	9.35	3.16	0.86	17.38	-0.45
20	6.63	11.23	17.86	0.17	1.00	1.69	1.09	9.29	3.22	0.84	17.31	-0.55
21	6.62	11.19	17.81	0.17	1.05	1.66	1.09	8.94	4.72	0.84	18.47	0.66
22	6.62	11.19	17.81	0.17	1.05	1.66	1.09	8.82	4.53	0.85	18.16	0.35
23	6.61	11.19	17.80	0.17	1.05	1.66	1.09	8.67	4.49	0.85	17.98	0.18
24	6.61	11.19	17.80	0.17	1.05	1.66	1.09	8.56	4.46	0.85	17.84	0.04
25	6.61	11.19	17.80	0.17	1.05	1.66	1.09	8.48	4.45	0.85	17.75	-0.05
26	6.61	11.19	17.80	0.17	1.05	1.65	1.09	8.40	4.48	0.85	17.70	-0.10
27	6.61	11.19	17.80	0.17	1.05	1.65	1.09	8.32	4.51	0.85	17.65	-0.15
28	6.61	11.19	17.80	0.17	1.05	1.65	1.09	8.26	4.55	0.85	17.63	-0.17
29	6.61	11.19	17.80	0.17	1.04	1.65	1.09	8.07	4.71	0.85	17.58	-0.22
30	6.61	11.19	17.80	0.17	1.05	1.65	1.09	8.12	4.65	0.85	17.57	-0.23

* All units are in Lt / sec
For Groundwater Reserv Change Values

dewatering simulations three points were chosen to monitor the groundwater level changes. These points were located at the downstream of the pit, in the pit and upstream of the pit. Figure 7.2 shows the location of the points and the change in groundwater levels during 30 year period.

Table 6.4: Water Quality Standards

Parameters	Unit	Drinking water quality standards				Irrigation					Discharge Limits To Skz Stream	AST Marl Quarry	
		EU, 1998	SB, 1997	WHO, 1998	Min.*	I. Class (very good)	II. Class (good)	III. Class (can be used)	IV. Class (used with caution)	V. Class (Harmfull, not suitable)		GW	Pit Lake
TSS	mg/l					20	30	45	60	>100	200	129	10
Alkalinity-Bicarbonate CaCO3	mg/l											84.4	79.1
Alkalinity-Carbonate CaCO3	mg/l											<1.0	<1.0
Alkalinity-Hydroxide CaCO3	mg/l											<1.0	<1.0
Alkalinity-Total CaCO3	mg/l											84.4	79.1
Aluminium	mg/l	0.2	0.2	0.2	0.2							<0.050	<0.050
Ammonia	mg/l		0.05	1.5	0.05								
Ammonia Nitrogen N	mg/l											0.195	0.0267
Ammonium	mg/l	0.5			0.5								
Ammonium-N	mg/l										0.2		
Antimony	mg/l	0.01	0.005	0.005	0.005							<0.0050	<0.0050
Arsenic	mg/l	0.01	0.01	0.01	0.01						0.5	0.0069	<0.0050
Barium	mg/l			0.7	0.7							0.081	0.034
Beryllium	mg/l											<0.010	<0.010
Bismuth	mg/l											<0.20	<0.20
BOD ₅	mg/l					0-25	25-50	50-100	100-200	>200	50		
Boron	mg/l	1	0.3	0.5	0.3	0-0.5	0.5-1.12	1.12-2	2			1.92	1.86
Cadmium	mg/l	0.005	0.003	0.003	0.003						0.05	<0.00050	<0.00050
Calcium	mg/l		100		100							157	238
Changeable Na per. (%Na)						<20	20-40	40-60	60-80				
Chlorine	mg/l	250	250	250	250	0-142	142-249	249-426	426-710	>710		281	203
Chromium	mg/l											<0.010	<0.010
Cobalt	mg/l											<0.0030	<0.0030
COD	mg/l										170		
Color	Pt-Co		10	15	10								
Copper	mg/l										0.5	<0.010	<0.010
Electrical Conductivity (Ec)	mS/cm	2500			2500	0-250	250-750	750-2000	2000-3000	>3000		3960	5520
Fluorine	mg/l	1.5	1.5	1.5	1.5						20		
Free Chlorine	mg/l										0.5		
Free Cyanide	mg/l										0.06		
Hardness CaCO3	mg/l											885	1700
Iron	mg/l	0.2	0.3	0.3	0.2							<0.030	<0.030
Lead	mg/l	0.01	0.01	0.01	0.01						0.5	<0.0050	<0.0050
Lithium	mg/l											0.114	0.197
Magnesium	mg/l	0.05	0.05	0.1-0.5	0.05							120	268
Manganese	mg/l		50		50							0.0109	<0.0030
Mercury	mg/l	0.001	0.001	0.001	0.001						0.01	<0.000050	<0.000050
Molybdenum	mg/l			0.07	0.07							<0.010	0.021
Nickel	mg/l	0.02	0.02	0.02	0.02						0.5	<0.010	<0.010
Nitrate	mg/l	50	45	50	45								
Nitrite	mg/l	0.5	0.05	0.2	0.05								
Nitrite/Nitrate Nitrogen N	mg/l											0.197	0.0583
NO ₃ or NH ₄ ⁺	mg/l										5		
Oil and grease	mg/l										10		
pH		6.5-9.5	5.5-8.5	6.5-9.5	0						5 - 9	7.78	8.18
Phenols	mg/l										5		
Potassium	mg/l		12		12							8.1	19.9
Selenium	mg/l	0.01	0.01	0.01	0.01							0.034	<0.010
Silicon	mg/l											4.79	1.87
Silver	mg/l											<0.00020	<0.00020
Sodium	mg/l	200	175	200	175							600	749
SAR						<10	43374	18-26	>26				
RSC						<66	66-133	>133					
Strontium	mg/l											8.56	12.2
Sulfate	mg/l	250	250	250	250	0-192	192-336	336-575	576-960	>960		1790	3210
Thallium	mg/l											<0.0020	<0.0020
Tin	mg/l											<0.0050	<0.0050
Titanium	mg/l											<0.010	<0.010
Total Crom	mg/l	0.05	0.05	0.05	0.05						0.5		
Total Cyanide	mg/l	0.05	0.01	0.07	0.01						0.3		
Total dissolved solids	mg/l			1000	1000								
Total Phosphorus	mg/l										1	0.0708	0.0059
Total Salinity	mg/l					0-179	179-525	525-1400	1400-2100	>2100			
Uranium	mg/l			0.002	0.002							0.0027	0.0075
Vanadium	mg/l											<0.030	<0.030
Zinc	mg/l		3	3	3						2	<0.050	<0.050

CHAPTER VIII

PIT LAKE FLOODING

The pit will start to fill with water immediately after the dewatering operations stopped. In order to predict the pit lake flooding period and final lake elevation, pit lake hydrologic model (spreadsheet model) was developed. Figure 8.1 shows the conceptual water balance developed for the future pit lake.

In order to evaluate the inflow-outflow data some volumetric calculations were completed to find out the elevation/volume/surface area relationship. This relation was introduced as an input into the spreadsheet model.

The evaporation and precipitation components of the pit lake water balance were estimated from yearly average values multiplied by the lake surface area. Since the pit lake surface area changes with every change in storage, for every single stage these components were calculated by the model and the net precipitation value was obtained at the end of every time step.

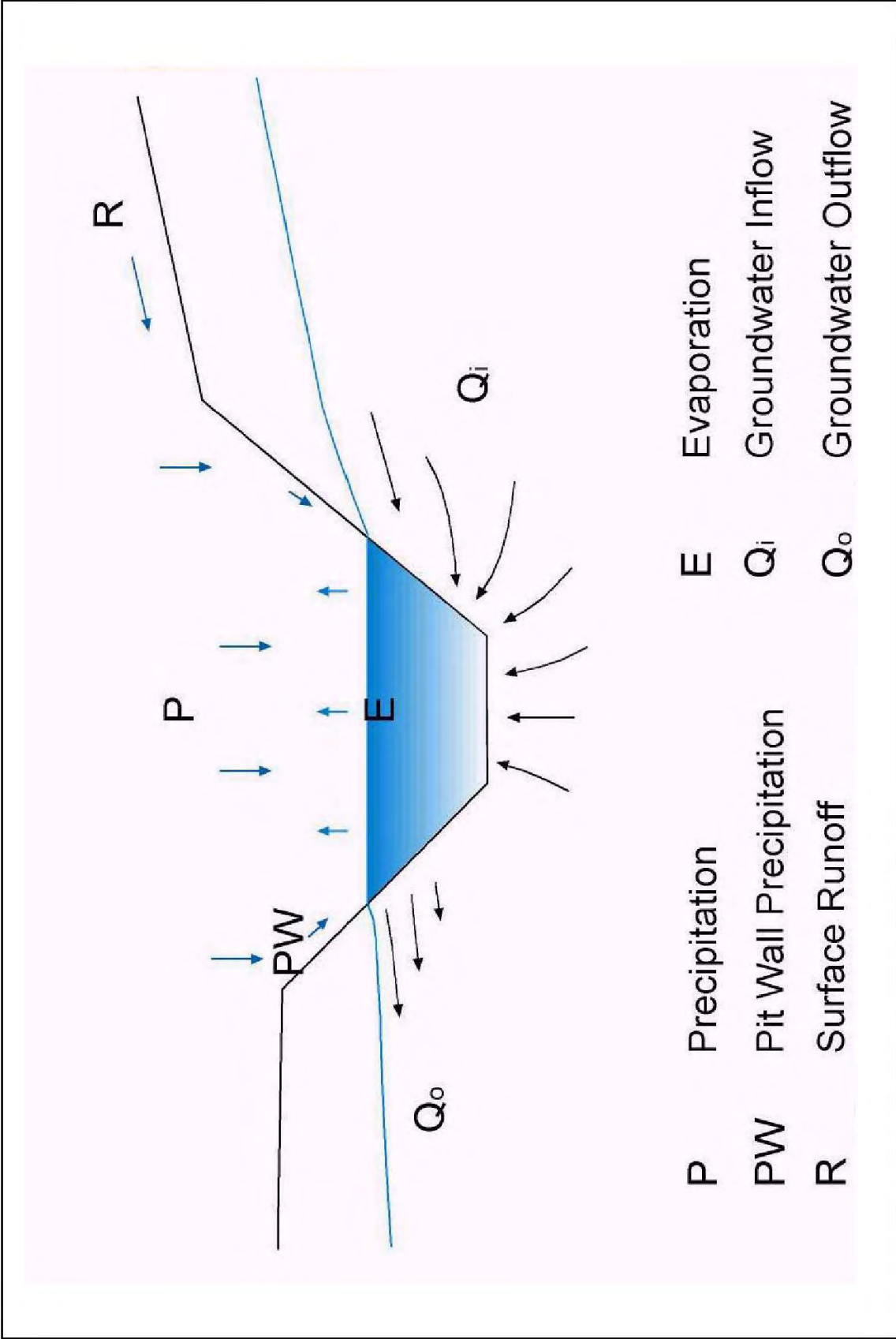


Figure 8.1. Conceptual Water Balance Model-Pit Lake

Precipitation falling on the pit walls may either evaporate from the benches or reach the pit lake through surface water runoff or recharge groundwater. There is some uncertainty associated with the fraction of pit wall precipitation that actually enters the pit lake. Based on the existing pit lake simulations, the pit wall run off constant was calibrated and used as an input to the final pit lake model.

Constructed groundwater model was used to predict the groundwater inflow and outflow rates to the pit lake. Through the use of constant head cells, the model predicted these rates into the pit at different stages.

Total inflow and total outflow were calculated separately and the net change in storage was calculated from these figures at the end of each time step. Then total inflow is added to the lake volume as the initial pit lake volume of next time step. This procedure was repeated several times until steady state conditions were attained.

The model was run with an annual precipitation value of 564.8mm and an evaporation value of 775.95mm. The simulations predict that the final pit lake elevation would be at 991 masl. The pit lake will rise to this level at approximately 72 years after closure. The water balance components when the pit lake reaches to its equilibrium conditions are estimated to contain the following components:

- Groundwater inflow: 77,544 m³/year
- Groundwater outflow: 60,766 m³/year
- Surface water Runoff and Direct Precipitation: 207,000 m³/year
- Evaporation: 224,150 m³/year

Two additional analyses were completed to simulate the long term extreme meteorological conditions. Long term precipitation data was analyzed for the extreme dry and wet periods.

In order to estimate the filling events for wet and dry periods meteorology data was analyzed again. Effective precipitation rate was calculated for each year and highest and lowest five years average values were determined to simulate wet and dry periods.

Wet period's precipitation and evaporation data was determined as 650.2mm and 962.8mm respectively. Then the Modflow model was run to determine the inflow and outflow rates for the wet period. Results were imported to the lake model and the simulation, which was run with these values, indicated that the flooding will complete in 83 years and the pit lake will be flooded to a level of 997m. Slowest period's precipitation and evaporation data was determined as 478.1mm and 1121mm, respectively and flooding period was calculated as 90 years and the level will rise to 985m. This simulation was also complete with the inflow and outflow rates determined from the Modflow model with the dry periods data. Figure 8.2 shows the wet, dry and average effective precipitation periods filling simulations.

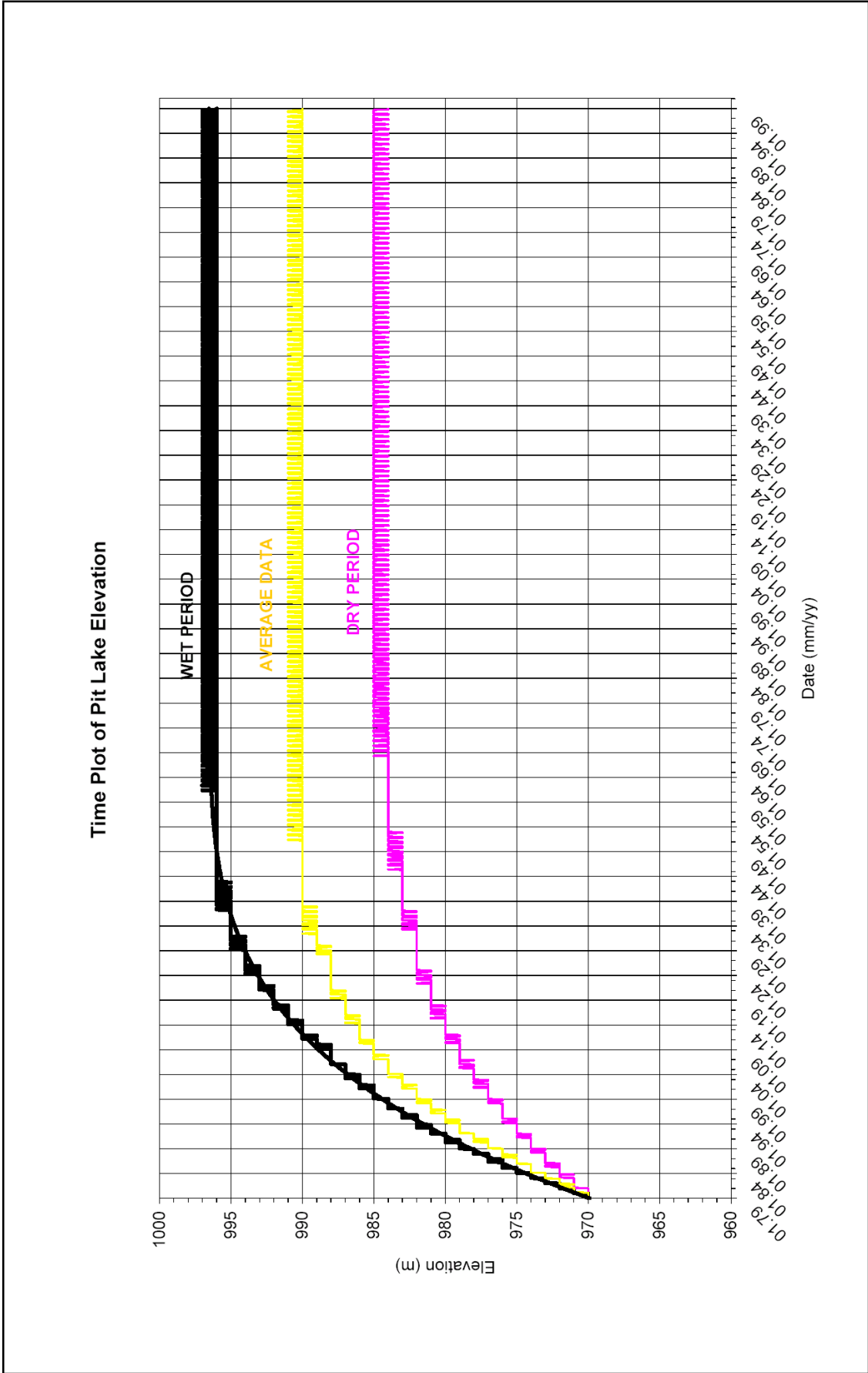


Figure 8.2. Final Pit Flooding with different precipitation and inflow data.

CHAPTER IX

CONCLUSIONS

In this study, various site specific data was collected and analyzed to prepare a dewatering plan for marl quarry. A numerical finite difference groundwater model was constructed and calibrated to site hydrogeological conditions. The calibrated model was subsequently used to test the performance of the proposed dewatering schemes and to analyze their impact on water resources. The response of the finite difference groundwater model was introduced to the pit lake model which simulates the pit lake flooding with the periodic budget calculations. The following can be concluded:

- Among various dewatering techniques that could be applied to the site, trench dewatering system is found to be highly efficient, least cost and compatible with the site hydrogeological characteristics and the mining methodology.

- Progressive dewatering is proposed to dewater the quarry.
- The rate of groundwater dewatered from the trench system varies from about 1.4 l/s to 4.9 l/s, the average rate being 3.7 l/s or 11,690 m³/yr. Because the expected discharge rate from the system is small, impact to water resources would be insignificant.
 - The dewatering pump should have a normal working capacity of 20-25 l/s to dewater the system during the wet season. In addition, high capacity pumps to discharge the flow (310 l/s) that will result from a 100-yr 24 –hr storm event should be available on site.
 - Excess water from the dewatering system can be discharged to Skz Stream. Elevated uranium and boron values may have an impact on the stream. Therefore, sampling and analysis of the Skz Stream is necessary.
 - The performance of the proposed trench dewatering system should be checked by discharge and groundwater level monitoring and if necessary secondary trenches should be constructed.
 - Existing pit lake can be dewatered in 34 days, with a continuous dewatering discharge rate of 40 l/s.
 - At the end of quarrying operations the pit will be flooded to the 991 masl. This filling period was predicted as 72 years with the average precipitation and evaporation rates.

REFERENCES

- Aguado, E., and Remson, I., 1980. *Groundwater Management with Fixed Charges*. Journal of Water Resources Planning and Management, ASCE,106(2), 375-382.
- BNG Mühendislik, 2003. *S. Sahasının Hammadde Olanakları (Jeoloji ve Sondaj)*. Ankara
- Bouwer, H. and Rice, R. C. 1976. *A slug test for determining hydraulic conductivity of unconfined aquifers with completely or partially penetrating wells*. Water Resources Res. Vol. 12, pp 423-428.
- Carelli, J., Lowe, A., Pressly, N.1998. *Horizontal Wells Installed For Groundwater Extraction at Brookhaven National Laboratory*. Horizontal News, Vol. 4/ Nb.1.
- Çevre ve Orman Bakanlığı, 1988. *Su Kirliliği Kontrol Yönetmeliği Kıta İçi Su Kaynakları Sınıflandırma Sınırları, 419/1988*. Ankara.
- Çevre ve Orman Bakanlığı, 1991. *Su Kirliliği Kontrol Yönetmeliği Teknik Usüller Tebliği, Resmi Gazete Sayı: 20748, Tarih 07/01/1991*. Ankara.
- Çevre ve Orman Bakanlığı, 2003. *Çevre Etki Değerlendirme Yönetmeliği*. Ankara

- Danskin, W. R., and Gorelick, S. M., 1985. *A Policy Evaluation Tool: Management of a Multiaquifer System Using Controlled Stream Recharge*. Water Resources Research, Vol.21, No.11, pp. 1731-1747.
- Dennen W. H. and Moore B.R., *Geology and Engineering*, Wm. C. Brown Publ., 1986.
- Driscoll, F. G., 1986. *Groundwater and Wells*. Johnson Filtration Systems Inc., St. Paul, Minnesota.
- Ferris, J. G., Knowless, D. B. , Brown,R. H., Stallman, R. W. 1962: *Theory of aquifer tests*.- US Geological Survey, Water supply paper 1536E, 174 pp.
- Glesson, B., 2002. *The Waters of The Nile*. Research Penn State, Vol 23, Issue 3.
- Kara, H., 1991. *1 / 100,000 Ölçekli Açınsama Nitelikli Türkiye Jeoloji Haritaları serisi*. MTA Yayını, Ankara.
- Mathewson C.C., *Engineering Geology*, C.E. Merril Publ. 1981.
- McDonald, M. G., and Harbaugh, A. W.,1984. *A Modular Three-Dimensional Finite-Difference Groundwater Flow Model*. Scientific Publications Co., Washington.
- Odling, N. E. et al, 1999. *Variations in a fracture system geometry and their implications for fluid flow in fractured hydrocarbon reservoirs*. Petroleum Geoscience, Vol. 5, pp. 373-384.
- Remson, I., Hornberger, G.M., and Molz, F.J., 1971. *Numerical Methods in Subsurface Hydrology*. John Wiley, New York.
- Neuman, S. P., 1975. *Analysis of pumping test data from anisotropic unconfined aquifers considering delayed gravity response*. Water Resources Res. Vol. 11, pp 329-342.

- Niccoli, W., Marinelli, F., Fairbanks, T. and Dancause, R. 1998. *Latin Hybercube Sampling: Application to Pit Lake Hydrologic Modeling Study*. Proceedings of the 1998 Conference on Hazardous Waste Research.
- Odling, N. E. et al, 1999. *Variations in a fracture system geometry and their implications for fluid flow in fractured hydrocarbon reservoirs*. Petroleum Geoscience, Vol. 5, pp. 373-384.
- SRK Consulting, 2004. *Rehabilitation Plan, AST Marl Quarry*. Ankara
- Sumer, S. M., Elton, J. J., Tapics, J. A., 1988. *Dewatering Optimization Using a Groundwater Flow Model at the Whitewood Open Pit Coal Mine, Alberta*. Canadian Geotech., J. 25, 684-693.
- Stone, D.B. and Fontaine, R.C., 1988. *Simulation of Groundwater Fluxes During Open Pit Filling and Under Steady State Pit Lake Conditions*. Proceedings of the 1998 Conference on Hazardous Waste Research.
- Tarbutck J.E. and Luthens F.K., *The Earth. An Introduction to Physical Geology*, Merril Publ., 1987.
- Tahir, Y., 1987. *X ve Yöresi Uranyum Aramaları Raporu*, MTA Yayını, Ankara.
- The Canadian Geotechnical Society, 1987. *A Symposium on Dewatering- Theory and Practice*, Ontario.
- Theis, C. V. 1935. *The relation between the lowering of piezometric surface and the rate of duration of discharge of a well using groundwater storage*. Trans American Geophys. Union, Vol. 16, pp 519-524.
- Tokgoz et. al, 2002. *Optimal Aquifer Dewatering Schemes for Excavation of Collector Line*. Journal of Water Resources Planning and Management, Vol. 128, No. 4, July/August 2002, pp. 248-261
- Wang, H.F., and Anderson, M.P., 1982. *Introduction to Groundwater Modeling*. W.H. Freeman and Company, San Francisco.

http://www.blm.gov/education/mining/mining_poster.html (Last Access 5 April 2004)

<http://www.civil.port.ac.uk/projects/geotech/geo4.html> (Last Access 5 April 2004)

<http://www.geoforum.com/knowledge/texts/broms/viewpage.asp?ID=29> (Last Access 5 April 2004)

http://www.readymix-zement.de/pdf/Rohst_e.pdf (Last Access 5 April 2004)

<http://www.unr.edu/mines/adti/ElkoPresentation/PitLake.pdf> (Last Access 5 April 2004)

APPENDIX A-1

Surface Water Measurements

SKZ STREAM FLOW MEASUREMENTS						
Station Name:		SW -2				
Coordinates:		3214/ 6285				
Date:		14.12.2003				
Time:		08:40				
Width of channel (m):		3.80				
Weather Conditions:		Sunny				
Distance(m)	Depth (cm)	Velocity Reading (m/sec) X10			Average Velocity (m/sec)	Notes
0.175	18	0.57			0.06	
0.555	26	3.11	3.87	3.84	0.36	
0.935	25.5	6.34	6.56	6.21	0.64	
1.315	24.5	5.67	5.57	5.93	0.57	
1.695	21	3.46	3.95	3.59	0.37	
2.075	17.5	3.82	3.93	3.94	0.39	
2.455	10.5	2.65	2.69	2.6	0.26	
2.835	8.5	3.5			0.35	
3.215	4	2			0.20	
3.595	2	2			0.20	
Total Q (m³/s)					0.23614	
Measured by:		Uygar DURU				

SKZ STREAM FLOW MEASUREMENTS						
Station Name:		SW -1				
Coordinates:		1761/ 5783				
Date:		14.12.2003				
Time:		09:30				
Width of channel (m):		3.60				
Weather Conditions:		Sunny				
Distance(m)	Depth (cm)	Velocity Reading (m/sec) X10			Average Velocity (m/sec)	Notes
0.18	18.5	1.24	1.29	1.15	0.12	
0.54	28	4.42	4.28	4.28	0.43	
0.9	27	4.99	4.97	4.97	0.50	
1.26	25	4.32	4.72	4.92	0.47	
1.62	21.5	5.1	5.56	5.19	0.53	
1.98	21	4.83	4.99	4.79	0.49	
2.34	17	3.68	3.69	3.69	0.37	
2.7	13	3.65	3.51	3.57	0.36	
3.06	11.5	1.22	1.85	1.92	0.17	
3.42	4	0.57			0.06	
Total Q (m³/s)					0.266754	
Measured by:		Uygar DURU				

APPENDIX A-2

Laboratory Results

RESULTS OF ANALYSIS

Sample ID	Pump Well	Pit Lake
Date Sampled	03.12.2003	03.12.2003
Time Sampled		
Sample ID	1	2
Nature	Water	Water

Physical Tests

Conductivity (uS/cm)	3960	5520
Total Dissolved Solids	3000	4900
Hardness CaCO ₃	885	1700
pH	7.78	8.18
Total Suspended Solids	129	10

Dissolved Anions

Alkalinity-Total CaCO ₃	84.4	79.1
Alkalinity-Bicarbonate CaCO ₃	84.4	79.1
Alkalinity-Carbonate CaCO ₃	<1.0	<1.0
Alkalinity-Hydroxide CaCO ₃	<1.0	<1.0
Chloride Cl	281	203
Sulphate SO ₄	1790	3210

Nutrients

Ammonia Nitrogen N	0.195	0.0267
Nitrite/Nitrate Nitrogen N	0.197	0.0583
Total Phosphate P	0.0708	0.0059

Dissolved Metals

Aluminum D-Al	<0.050	<0.050
Antimony D-Sb	<0.0050	<0.0050

Arsenic	D-As	0.0069	<0.0050
Barium	D-Ba	0.081	0.034
Beryllium	D-Be	<0.010	<0.010
Bismuth	D-Bi	<0.20	<0.20
Boron	D-B	1.92	1.86
Cadmium	D-Cd	<0.00050	<0.00050
Calcium	D-Ca	157	238
Chromium	D-Cr	<0.010	<0.010
Cobalt	D-Co	<0.0030	<0.0030
Copper	D-Cu	<0.010	<0.010
Iron	D-Fe	<0.030	<0.030
Lead	D-Pb	<0.0050	<0.0050
Lithium	D-Li	0.114	0.197
Magnesium	D-Mg	120	268
Manganese	D-Mn	0.0109	<0.0030
Mercury	D-Hg	<0.000050	<0.000050
Molybdenum	D-Mo	<0.010	0.021
Nickel	D-Ni	<0.010	<0.010
Potassium	D-K	8.1	19.9
Selenium	D-Se	0.034	<0.010
Silicon	D-Si	4.79	1.87
Silver	D-Ag	<0.00020	<0.00020
Sodium	D-Na	600	749
Strontium	D-Sr	8.56	12.2
Thallium	D-Tl	<0.0020	<0.0020
Tin	D-Sn	<0.0050	<0.0050
Titanium	D-Ti	<0.010	<0.010
Uranium	D-U	0.0027	0.0075
Vanadium	D-V	<0.030	<0.030
Zinc	D-Zn	<0.050	<0.050

Footnotes

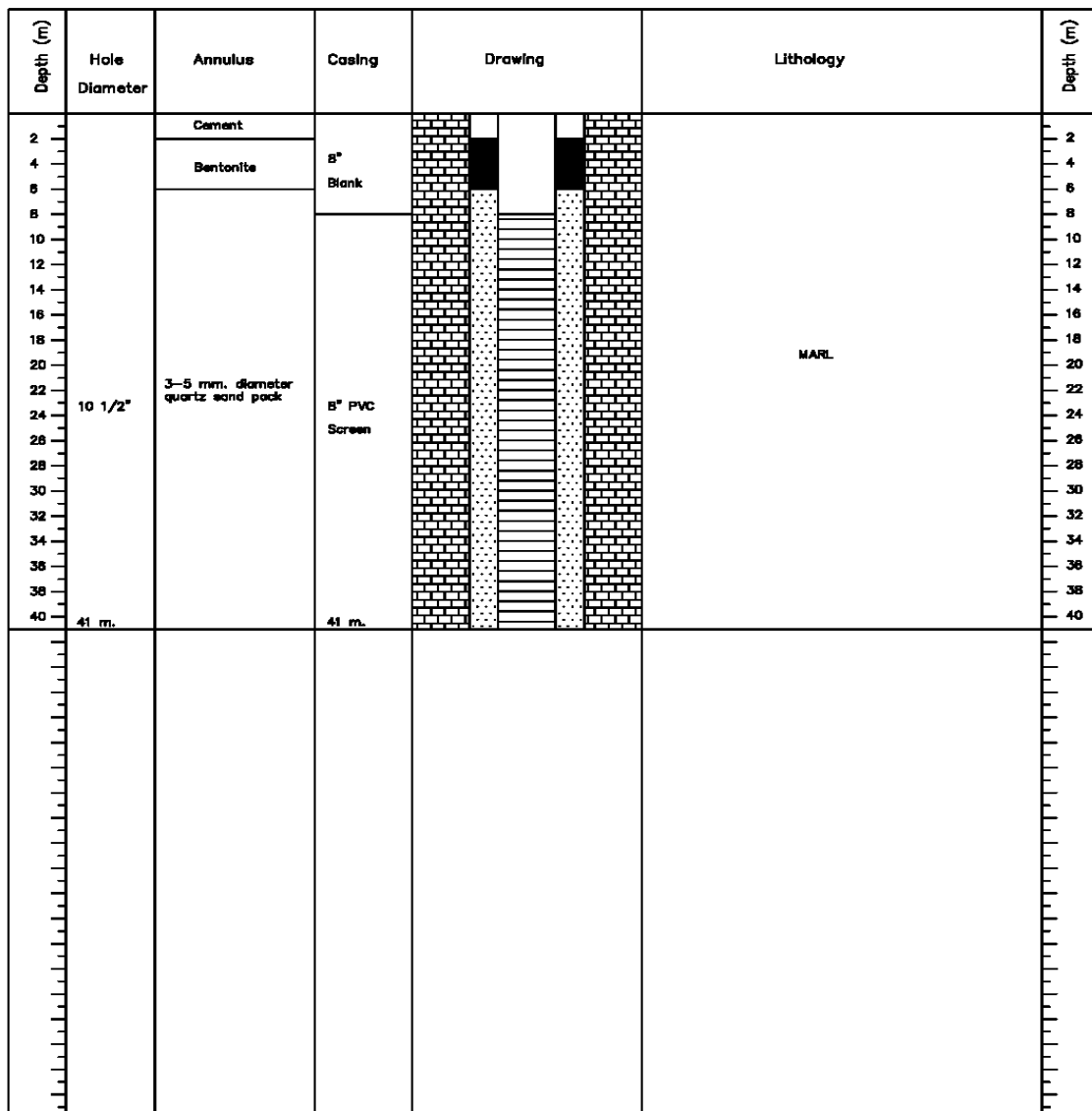
Results are expressed as milligrams per litre except where noted.

< = Less than the detection limit indicated

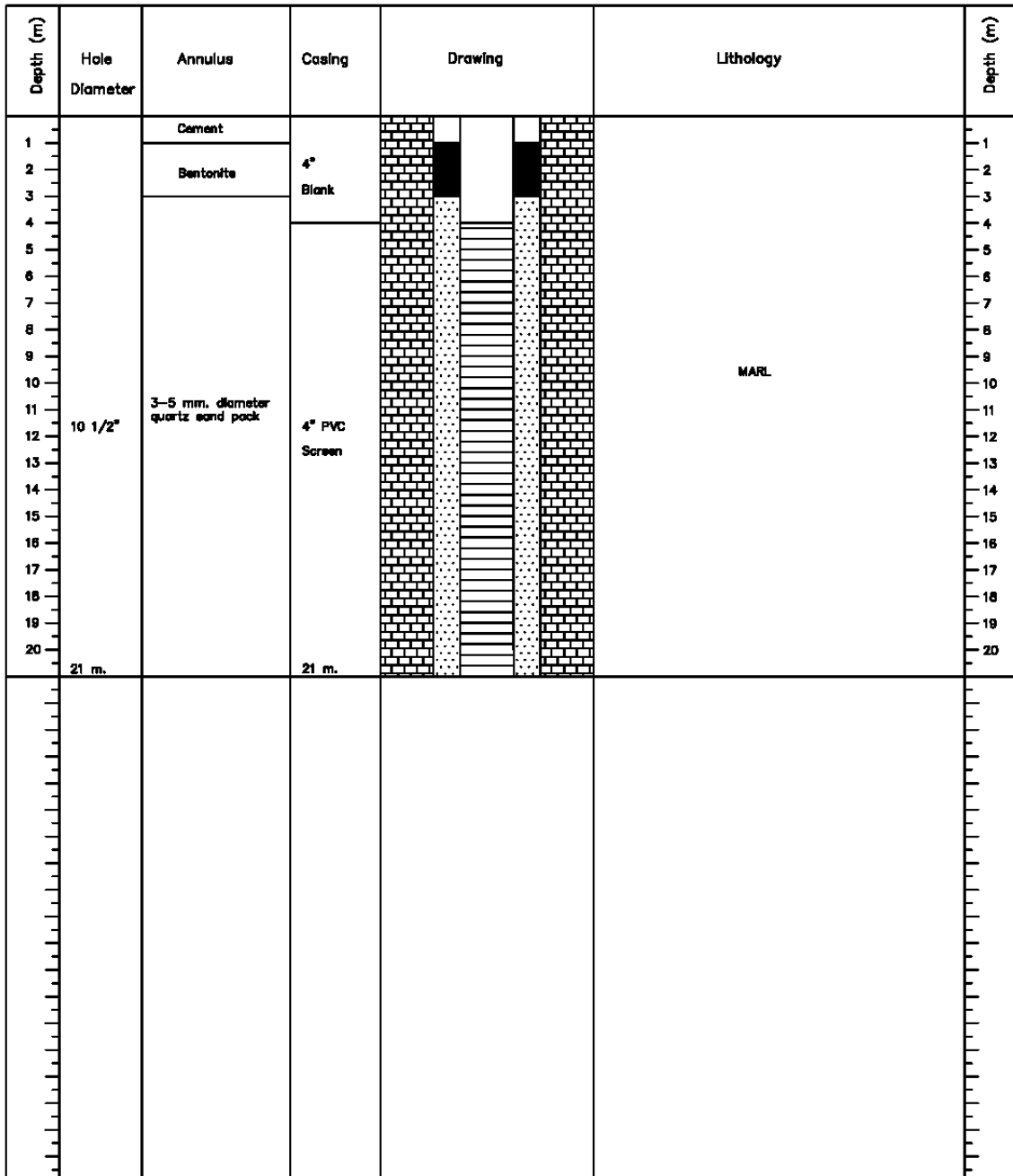
APPENDIX B-1

Well Construction Details

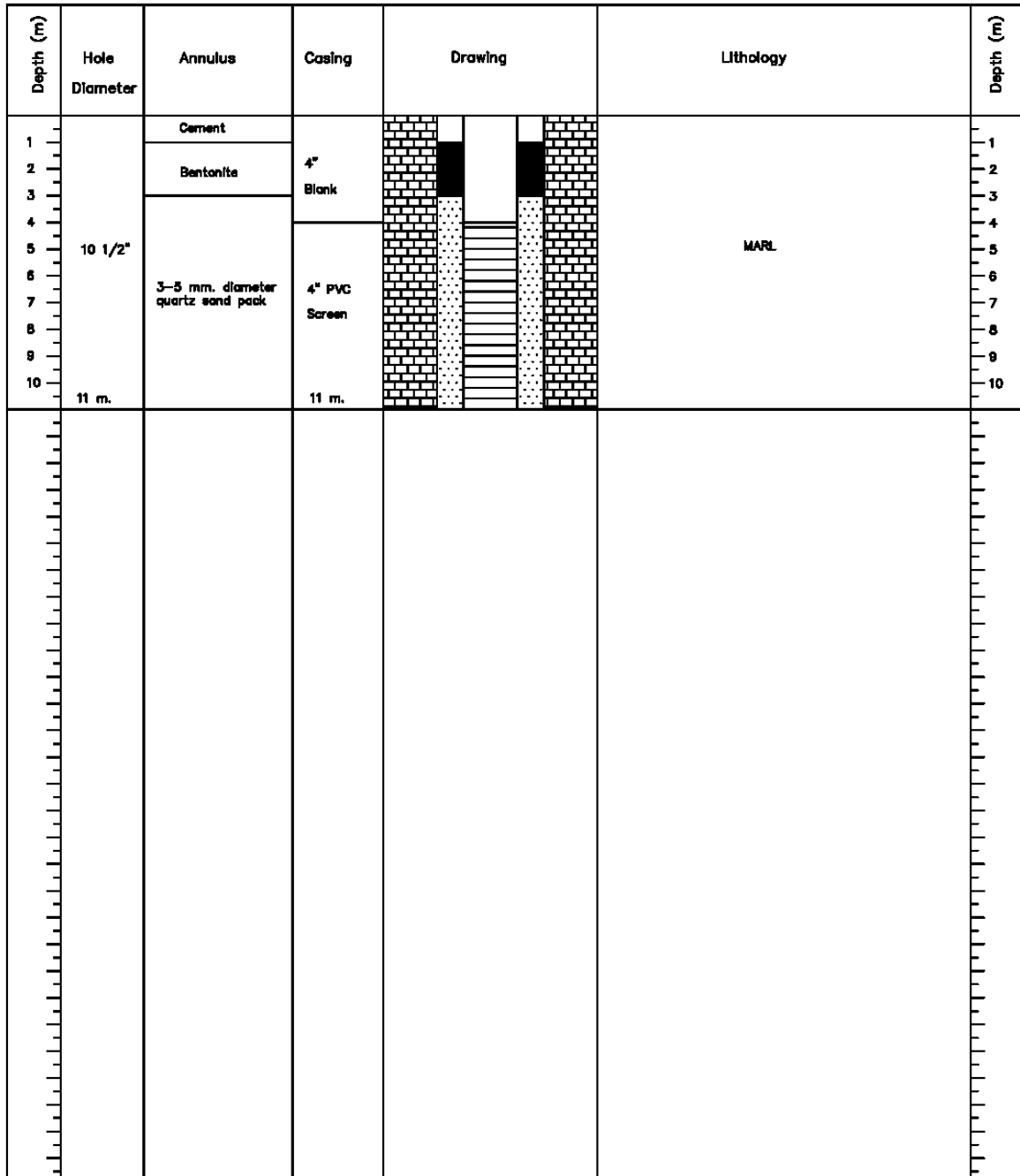
Well ID	PW			
Coordinates			Elevation	994.5
City			Province	
Casing	PVC, 8" diameter, from 0 m. to 41 m.		Drill Method	Rotary, 10 1/2" diameter tricone bit
Screen	2mm slot opening, from 4 m. to 41 m.		Drilling Fluid	Water
Gravel Pack	3-5 mm diameter, Quartz sand, from 3 m. to 41 m.		Hole	10 1/2 diameter,
Sealing	Bentonite seal from 1m. to 3 m.		Date Started	
Surface Seal	Cement injection, from 0 m. to 1 m.		Date Finished	



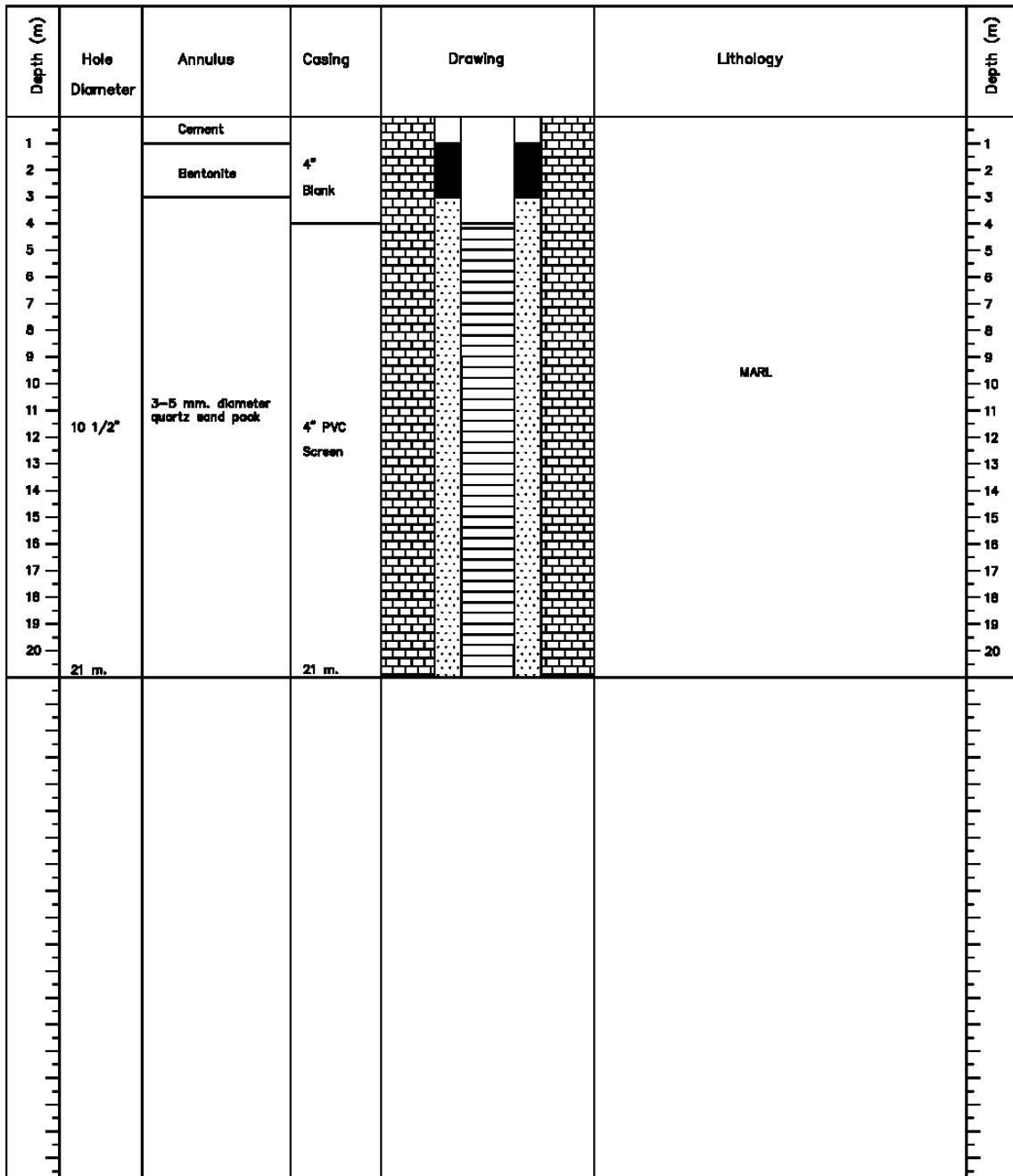
Well ID	OW-1		Elevation	993.960
Coordinates			Province	
City			Province	
Casing	PVC, 4" diameter, from 0 m. to 21 m.		Drill Method	Rotary, 10 1/2" diameter tricone bit
Screen	2mm slot opening, from 4 m. to 21 m.		Drilling Fluid	Water
Gravel Pack	3-5 mm diameter, Quartz sand, from 3 m. to 21 m.		Hole	8 1/2 diameter,
Sealing	Bentonite seal from 1m. to 3 m.		Date Started	
Surface Seal	Cement Injection, from 0 m. to 1 m.		Date Finished	



Well ID	OW-2		
Coordinates		Elevation	995.56
City		Province	
Casing	PVC, 4" diameter, from 0 m. to 11 m.	Drill Method	Rotary, 10 1/2" diameter tricone bit
Screen	2mm slot opening, from 4 m. to 11 m.	Drilling Fluid	Water
Gravel Pack	3-5 mm diameter, Quartz sand, from 3 m. to 11 m.	Hole	8 1/2 diameter,
Sealing	Bentonite seal from 1m. to 3 m.	Date Started	
Surface Seal	Cement Injection, from 0 m. to 1 m.	Date Finished	

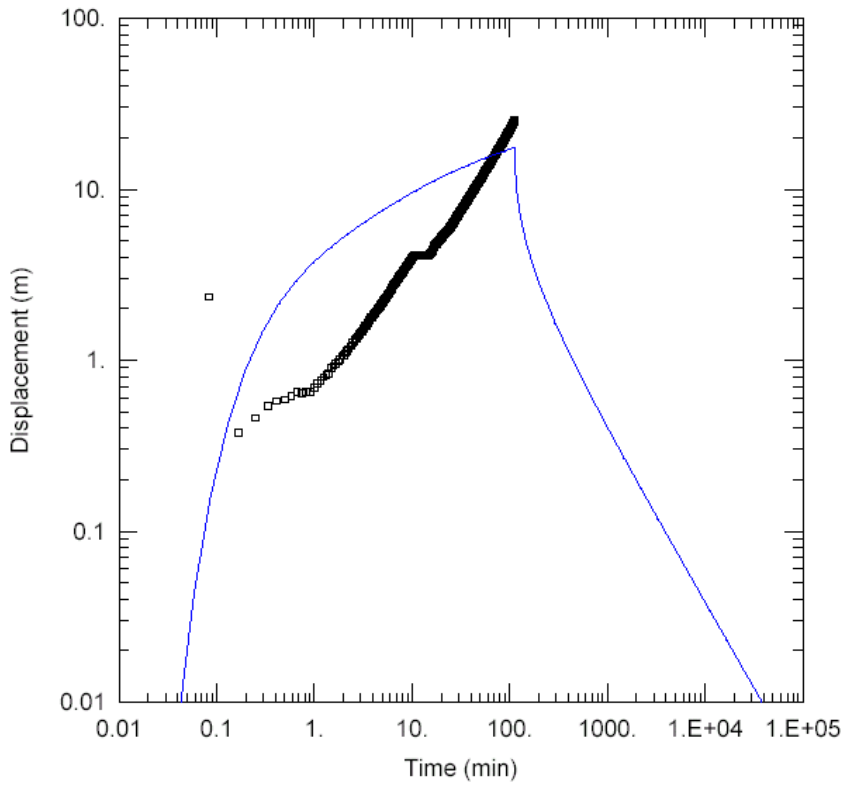


Well ID	OW-3		
Coordinates		Elevation	995.7
City		Province	
Casing	PVC, 4" diameter, from 0 m. to 21 m.	Drill Method	Rotary, 10 1/2" diameter tricone bit
Screen	2mm slot opening, from 4 m. to 21 m.	Drilling Fluid	Water
Gravel Pack	3-5 mm diameter, Quartz sand, from 3 m. to 21 m.	Hole	8 1/2 diameter,
Sealing	Bentonite seal from 1m. to 3 m.	Date Started	
Surface Seal	Cement Injection, from 0 m. to 1 m.	Date Finished	

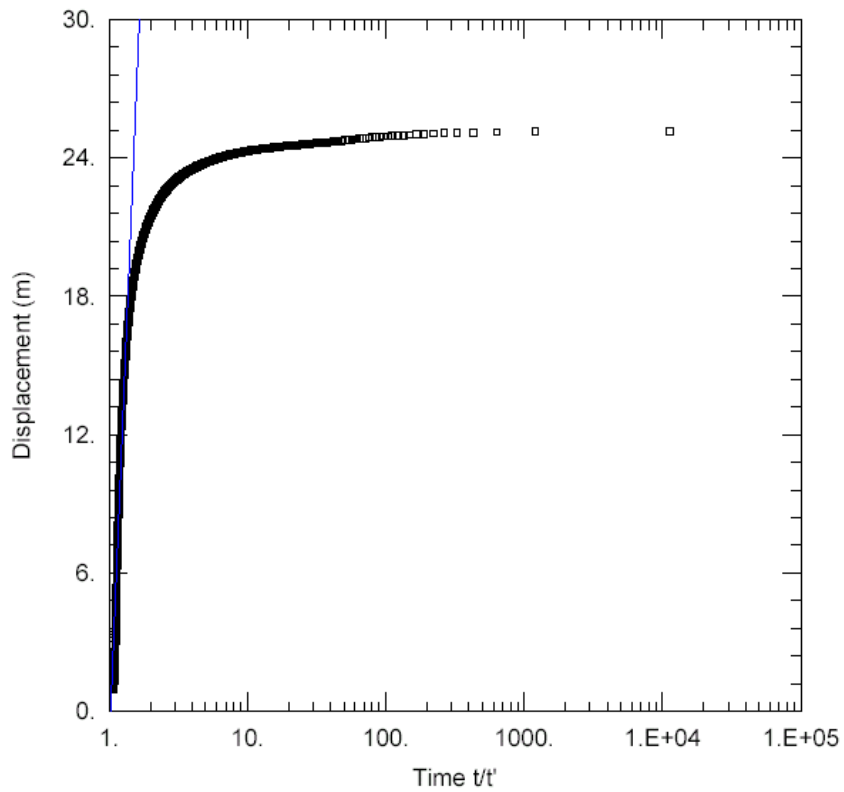


APPENDIX B-2

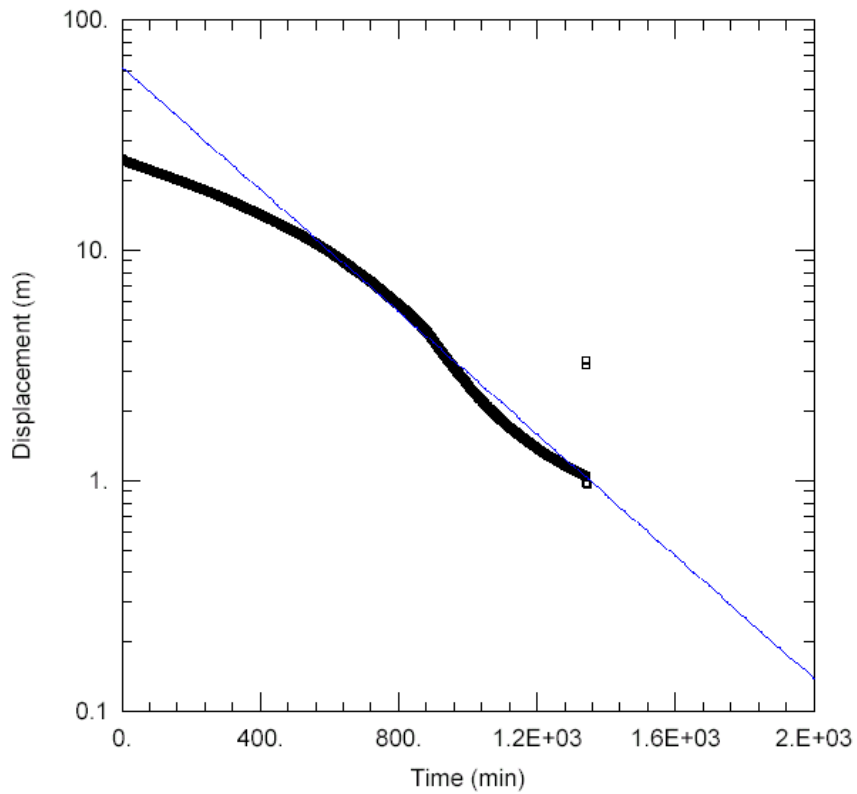
Test Results



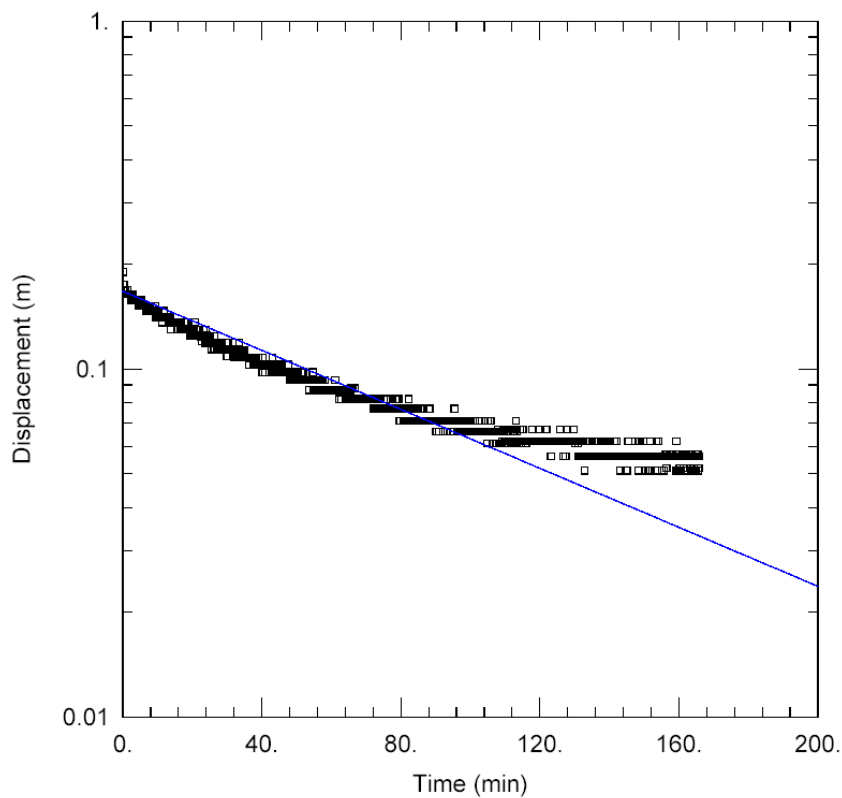
<u>WELL TEST ANALYSIS</u>					
Data Set: <u>PUMPTEST</u>		Time: <u>19:27:27</u>			
Date: <u>03/08/04</u>					
<u>AQUIFER DATA</u>					
Saturated Thickness: <u>48.17</u> m					
<u>WELL DATA</u>					
Pumping Wells			Observation Wells		
Well Name	X (m)	Y (m)	Well Name	X (m)	Y (m)
PW 1	0	0	PW 1	0	0
<u>SOLUTION</u>					
Aquifer Model: <u>Unconfined</u>			Solution Method: <u>Neuman</u>		
T = <u>3.178E-06</u> m ² /sec			S = <u>0.01655</u>		
Sy = <u>0.01769</u>			β = <u>0.1</u>		



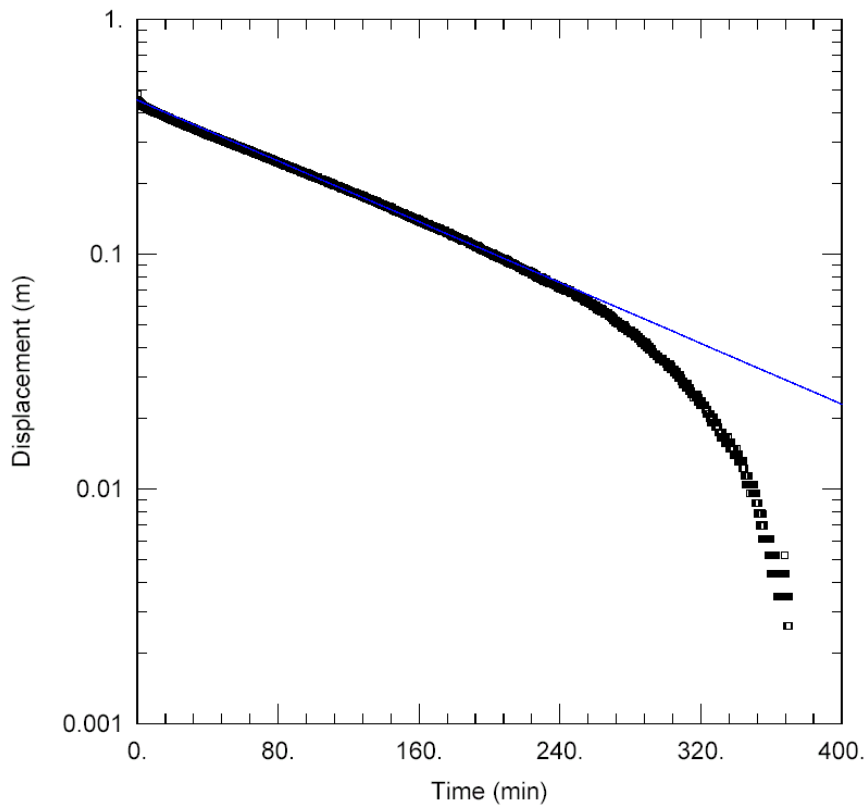
<u>WELL TEST ANALYSIS</u>					
Data Set: <u>PUMPTEST-RECOVERY</u>					
Date: <u>03/08/04</u>			Time: <u>19:26:50</u>		
<u>AQUIFER DATA</u>					
Saturated Thickness: <u>48.17</u> m			Anisotropy Ratio (Kz/Kr): <u>1.</u>		
<u>WELL DATA</u>					
Pumping Wells			Observation Wells		
Well Name	X (m)	Y (m)	Well Name	X (m)	Y (m)
PW 1	0	0	□ PW 1	0	0
<u>SOLUTION</u>					
Aquifer Model: <u>Confined</u>			Solution Method: <u>Theis (Recovery)</u>		
T = <u>1.709E-07</u> m ² /sec			S' = <u>1.033</u>		



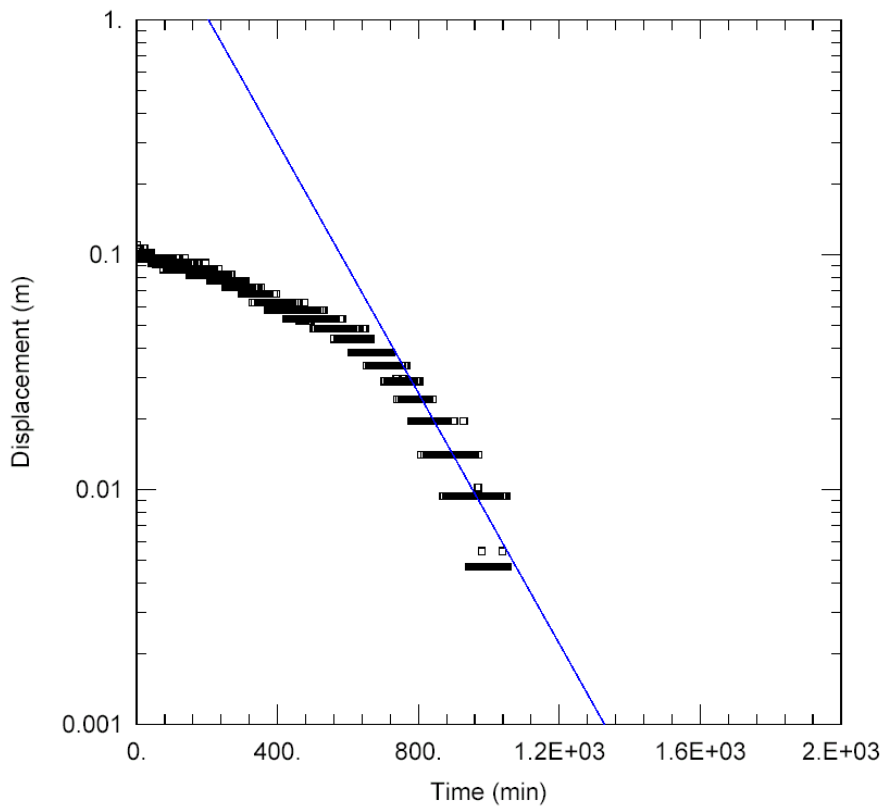
<u>WELL TEST ANALYSIS</u>	
Data Set: <u>PW RECOVERY SOLVED AS SLUG TEST</u>	
Date: <u>03/08/04</u>	Time: <u>19:26:29</u>
<u>AQUIFER DATA</u>	
Saturated Thickness: <u>49.17 m</u>	Anisotropy Ratio (Kz/Kr): <u>1.</u>
<u>WELL DATA (PW 1)</u>	
Initial Displacement: <u>25.16 m</u>	Water Column Height: <u>39.17 m</u>
Casing Radius: <u>0.0925 m</u>	Wellbore Radius: <u>0.15 m</u>
Screen Length: <u>36. m</u>	Gravel Pack Porosity: <u>0.3</u>
<u>SOLUTION</u>	
Aquifer Model: <u>Unconfined</u>	Solution Method: <u>Bouwer-Rice</u>
K = <u>3.626E-08 m/sec</u>	y0 = <u>61.96 m</u>



<u>WELL TEST ANALYSIS</u>	
Data Set: <u>PW SLUG TEST</u>	Time: <u>19:26:03</u>
Date: <u>03/08/04</u>	
<u>AQUIFER DATA</u>	
Saturated Thickness: <u>49.17 m</u>	Anisotropy Ratio (Kz/Kr): <u>1.</u>
<u>WELL DATA (PW 1)</u>	
Initial Displacement: <u>0.19 m</u>	Water Column Height: <u>39.17 m</u>
Casing Radius: <u>0.0925 m</u>	Wellbore Radius: <u>0.15 m</u>
Screen Length: <u>36. m</u>	Gravel Pack Porosity: <u>0.3</u>
<u>SOLUTION</u>	
Aquifer Model: <u>Unconfined</u>	Solution Method: <u>Bouwer-Rice</u>
K = <u>1.161E-07 m/sec</u>	y0 = <u>0.1673 m</u>



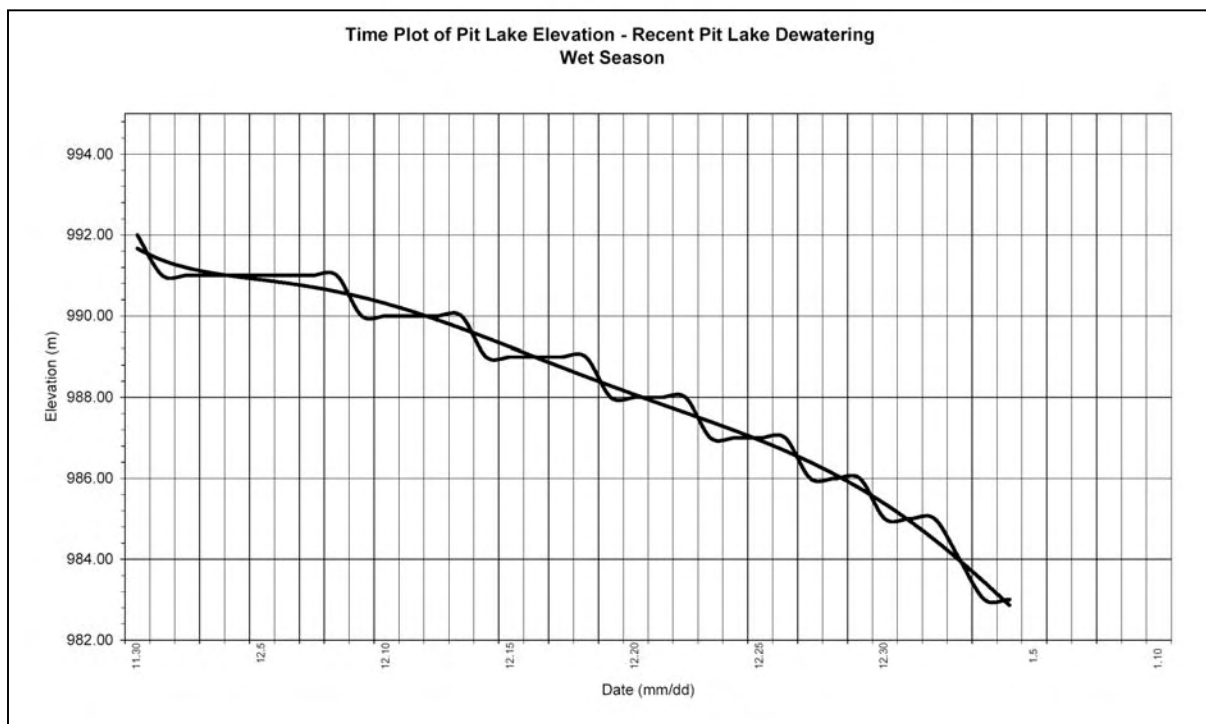
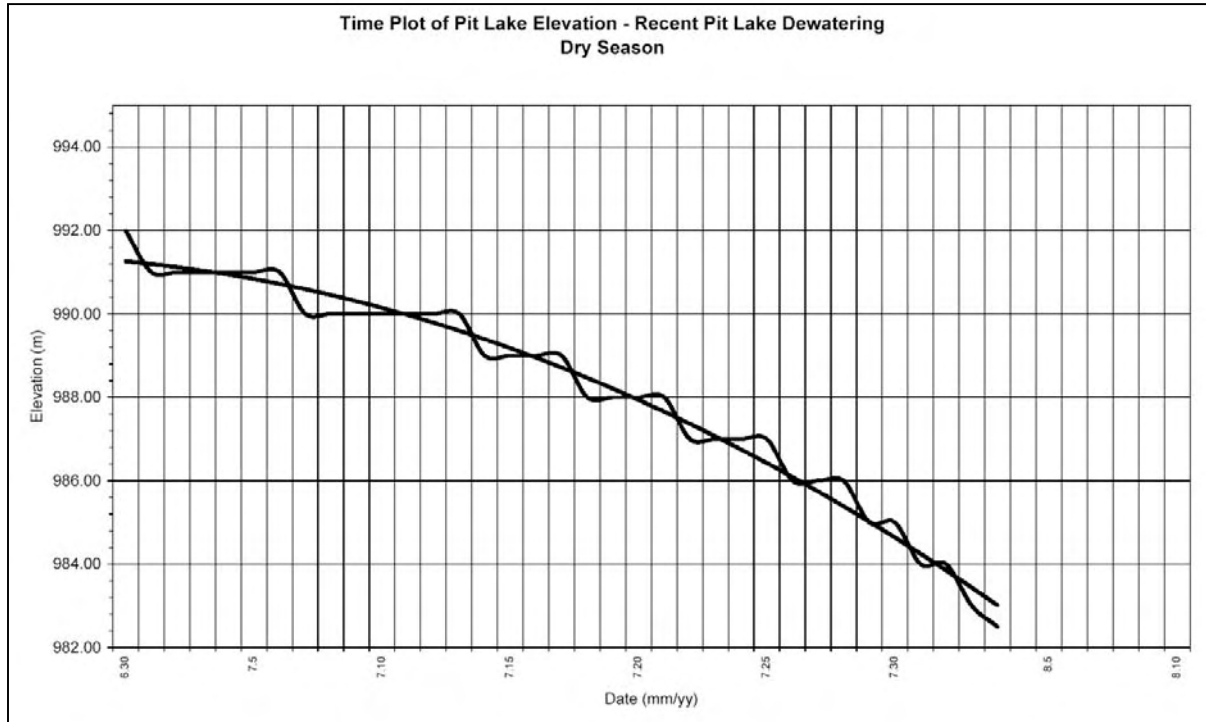
<u>WELL TEST ANALYSIS</u>	
Data Set: <u>OW 1 SLUG TEST</u>	Time: <u>19:28:01</u>
Date: <u>03/08/04</u>	
<u>AQUIFER DATA</u>	
Saturated Thickness: <u>50. m</u>	Anisotropy Ratio (Kz/Kr): <u>1.</u>
<u>WELL DATA (OW 1)</u>	
Initial Displacement: <u>0.48 m</u>	Water Column Height: <u>20. m</u>
Casing Radius: <u>0.055 m</u>	Wellbore Radius: <u>0.15 m</u>
Screen Length: <u>16. m</u>	Gravel Pack Porosity: <u>0.3</u>
<u>SOLUTION</u>	
Aquifer Model: <u>Unconfined</u>	Solution Method: <u>Bouwer-Rice</u>
K = <u>1.129E-07 m/sec</u>	y0 = <u>0.4513 m</u>



<u>WELL TEST ANALYSIS</u>	
Data Set: <u>OW 4 SLUG TEST</u>	Time: <u>19:28:13</u>
Date: <u>03/08/04</u>	
<u>AQUIFER DATA</u>	
Saturated Thickness: <u>46.74</u> m	Anisotropy Ratio (Kz/Kr): <u>1.</u>
<u>WELL DATA (OW 4)</u>	
Initial Displacement: <u>0.11</u> m	Water Column Height: <u>46.74</u> m
Casing Radius: <u>0.035</u> m	Wellbore Radius: <u>0.05</u> m
Screen Length: <u>47.</u> m	Gravel Pack Porosity: <u>0.3</u>
<u>SOLUTION</u>	
Aquifer Model: <u>Unconfined</u>	Solution Method: <u>Bouwer-Rice</u>
K = <u>1.007E-08</u> m/sec	y0 = <u>3.51</u> m

APPENDIX C

Pit Lake Dewatering Simulations



APPENDIX D

Construction Details For Dewatering Performance Monitoring Wells

Well ID	OW-2 (PROPOSED)			
Coordinates			Elevation	
City			Province	
Casing	PVC, 4" diameter, from 0 m. to 21 m.		Drill Method	Rotary, 8 1/2" diameter tricone bit
Screen	2mm slot opening, from 4 m. to 21 m.		Drilling Fluid	Water
Gravel Pack	3-5 mm diameter, Quartz sand, from 3 m. to 21 m.		Hole	8 1/2" diameter,
Sealing	Bentonite seal from 3 m. to 4 m.		Date Started	
Surface Seal	Cement injection, from 0 m. to 3 m.		Date Finished	

