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Review

Management of urban water for domestic and industrial uses and sustainability in Anambra State, Nigeria

A. C. C. Ezeabasili[1*], O. L. Anike[2] and A.U. Okonkwo[3]

[1]Heriot Watt University, England, UK.
[2]Nnamdi Azikiwe University, Awka, Anambra State, Nigeria
[3]Anambra State University, Uli, Anambra State, Nigeria.

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Nigeria has abundant water resources. This natural endowment is evident in the yearly rainfall and large surface bodies of water-rivers, streams as well as abundant reservoirs of underground water. However, because of lack of good developmental policy, the rational use of the water resources still poses problems and challenges in most states of the country. For some time now these water problems have been left with the River Basin development authorities with little success, primarily because of lack of good management and best practices, despite huge sum of money spent. Although there are surplus surface and underground water resources within the Anambra basin, basic water supply for domestic and industrial use have been in very short supply for the teeming 5 million population despite the fact that government through international programmes like the Millennium Development Goals (MDGs), UNICEF, EU has done a lot to aid water supply. This paper is timely because of a recent introduction of a major regional water supply project earmarked to take off in Onitsha, the industrial and commercial hub of Anambra. It is expected that the quantity and quality of water would improve with this project to be financed by the government in urban areas of Onitsha, Awka and Nnewi. Published as well as unpublished secondary sources were used to present the access to drinking water in Anambra State. Reform efforts are currently going on in the state and a review of the reform reveals the effects of political and economic challenges on the existing strategies.

Key words: Urban water, water reforms, sustainability, sectors.

INTRODUCTION

Nigeria has not been able to guarantee the most important need of safe drinking water for its citizens and small population of industries. Every part of Anambra State is battling with insufficient of this basic necessity. The local government headquarters of Nnewi, Awka, Ekwuluobia, Ihiala, Ogidi, Abagana and Onitsha have been without sufficient domestic and industrial water for the past forty years since after the Nigeria Civil war. Major sources of water were rivers and stream for the first twenty years while the development of private water for individual homes and industries by the private sector, local governments and international agencies have

*Corresponding author. E-mail: accezeabasili@yahoo.com.

recently impacted on the provision of water. The problem of raising the level of drinking water from low to medium term and in cost effective ways is imperative in the state. The objective of the paper is to assess the ongoing water sector reform in Anambra State of Nigeria, to determine the key barriers to more effective reform for urban water provision with respect to water quality and water sector efficiency. Ongoing attempts by government including those of private sectors, to reform the water sector and other necessary recommendations are included in this review.

In the Urban areas of Anambra State piped water supplies does not exist in the seven (7) major cities of Onitsha, Awka and Nnewi, Abagana, Ihiala, Ogidi Ekwulubia. Virtually 95% of the population and industries receive their required water from water boreholes. It is only the rich that can afford the luxury of providing personal water boreholes for personal uses and industries. The present Administration in the state is planning major water schemes for Nnewi, Awka and Onitsha which is presently being executed. Other small town water scheme is either completed or ongoing.

THE CURRENT STATE OF URBAN WATER SUPPLY

Water availability and quality

The water supply in the urban cities was only available sparingly and the water is of questionable quality. Private and Public water borehole are common sites and supply most of the rected water. Intermittent water supply, and unpredictable service from boreholes and vendors impose both financial and health costs on the three major cities of Nnewi, Onitsha and Awka.

Many households within the urban cities were found to have undertaken long-term investments in the form of water tanks, handpumps or boreholes. Households with water tanks install booster pumps and pump water directly to water tanks. This increases the risks of contamination of the general water supply. Water tanks though popular are sources of various biological contaminants. Water pump (submersible) fixed to clevated water tanks as well. The WHO Guidelines for Drinking-water Quality (WHO 1998, 1997, 1993) are assessing the health risks posed by contaminants in drinking water. This provides a primary health requirement for a sufficient water supply, which the Government of Nigeria takes to mean about 40 L per person per day. The second requirement is that the water be microbiologically safe (FMoWR, 2006).

In most developing countries, the primary contaminant of surface and ground waters is human and animal waste (McKenzie, 2007). The WHO guidelines suggest that *Escherichia coli* should not be detectable in a 100-ml sample of water for the water to be considered to be of moderate good quality. The Government of Nigeria accepts these guidelines but has been unable to ensure that they are met. Water-borne diseases from faecal contamination are some of the biggest public health risks in the country. The incidence of typhoid fever diagnosed rightly or wrongly remains contentious till now. Monitoring of water quality in Nigeria cities is haphazard or nonexistent. Standards for drinking water that are actually enforced could have enormous positive impacts on public health, but for this to occur, the procedures for water testing and data sharing have to be made regular, standardized and public.

Water resources

Anambra State is drained principally by a number of rivers which include River Niger, Anambra River, Idemili and Mamu River Nkisi Obizi etc (Figure 1). In some areas the formation have confined aquifers and their depths depend on the nature of the formation relative high rainfall which occurs within 7 months of the year ensures the sustenance of perennial streams and rivers traversing the state including effective re-charge of groundwater acquifers (Muoghalu and Okonkwo, 1998). The majority of small water supply systems in the state are ground water based while the big water supply schemes are surface water based. The Nkissi stream is the main source of raw water for the existing Greater Onisha Water Supply Scheme, but for Awka and Nnewi, there is no scheme yet in existence. The greater Onitsha water scheme presently not functional is being considered as a major concession for the private sector. Before this when the public sector was responsible for the water works, epileptic water supply is available as provided by the state water cooperation, the water corporation was highly subsided by the Anambra State government but was finally starved of the subsidy, between 1999 and 2003 when its activities collapsed (AWSSP, 2009).

AREAS FOR URBAN REFORM

Pricing reform

Recovering at least part of the cost of a new water system or of upgrading and maintaining an existing water system is the primary rationale for pricing reform everywhere. Several studies have argued that poor people will pay for water if it is conveniently and reliably supplied and that appropriate pricing reform can promote both efficiency and equity (Boland and Dale, 2000; Mcintosh, 2003). Existing evidence suggests that many low-income households in Nigeria can afford to pay more for water, particularly if the increase in prices is accompanied by better service.

One might question whether metering in very low-income areas is cost-effective for the water utilities. In general, piped connections to homes in slums were not metered; therefore we do not have the data. Metering is

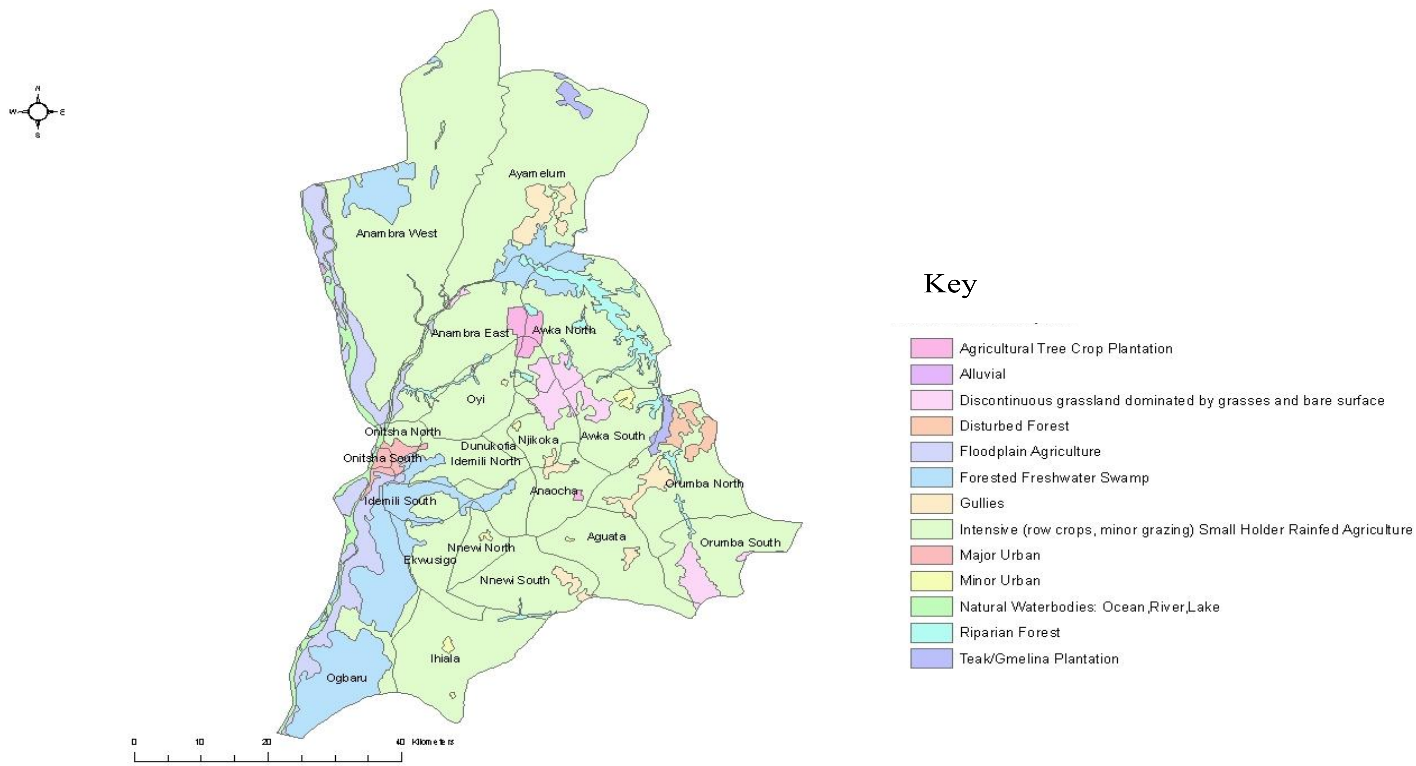

Figure 1. Anambra State Local Governments.

arbitrary done and the public does not have the ability to check the excesses of the water corporation.

However, even among low-income households there may be scope for metering with some adaptation in the future. Provision of metering devices and its functionality programme implementation in the state would likely improve with the modern water works that are envisaged for the future.

Financial reforms

A traditional mechanism for raising the capital needed for water and sewerage system expansions and upgrades is the bond, usually issued without guarantees from the federal government. Through such bonds, private credit markets lend money to state governments for a fixed period of time and at predetermined interest rates. These best practices in the developing countries have not worked well in Nigeria where state and Federal governments had been into direct participation. This practice has been variously abused in the rare instance where it was permitted by legislation in the states and the local governments especially between 1999 and 2007.

Private sector participation in urban water delivery

Over the last decade, privatization to a greater or less degree has been seen as one of the primary ways to infuse capital into the urban water sector and to overcome some of the inefficiencies of management. It has been urged upon developing countries by international lending agencies as an essential component of water sector reform.

However, the 2002 National Water Policy of the Government of Nigeria for the first time called for the encouragement of private sectors participation (PSP) in water resources. Private sector participation should be encouraged in planning, development and management of water resource projects for diverse uses, wherever feasible. Private sector participation incorporates a wide range of private sector involvement. The method of participation is coming on new in the state and would be in adapted forms. At one end lies contracting out of services to the private sector, such as mains repair, billing and collecting. Such arrangements are relatively straightforward and involve short-term (5 years) renewable contracts. More private involvement occurs under longer-term (20 to 35 years) concession and Build-own-transfer (BOT) contracts. This entails that under a concession, a private firm manages and operates the whole utility at its own commercial risk. BOT contracts are used for major investment in new facilities. At the other end of the public-private spectrum lies full divestiture, whereby the Government sells the assets of the water supply company to a private firm, who runs it on a permanent basis subject to government regulation.

Evaluation of the effects of PSP on urban water supplies has been constrained by the overall poor quality of data available and the small number of cases from which to draw conclusions. As in India, in many parts of the world the public utility does not release regular information on costs, water quality, the size of the network, etc., making it difficult to measure the pre-privatization trends in access, costs, and quality, and thus to determine what would have happened in the absence of private involvement. However, much of the criticism regarding privatization is that it will result in large increases in water tariffs, making water unaffordable to the poor (Sridhar, 2003b). But it is argued that it is better to have costly potable water than none. Overall, the experience of other countries suggests that PSP in the urban water sector may or may not, improve efficiency and provide better service to the poor. While it is too soon to evaluate the effects of private involvement in urban water in Anambra State, several projects are now underway. We comment here on three ongoing efforts. Nigeria's furthest step towards full privatization of water supply is the Build-Own-Operate-Transfer (BOOT) contract which is planned to be carried out by the Government at Onitsha.

Further private sector engagement such as concessions and leasing agreements will be difficult without prior pricing reform. Successful PSP therefore requires public awareness campaigns of the true costs of the current policies. It is equally important not to oversell the reforms possibly under private sector involvement through unrealistic goals and exaggerated public statements.

With the unattractiveness of peri-urban areas to the formal private sector (Cairncross, 2003), more city governments should consider recognizing, contracting with, and regulating local water entrepreneurs as mainstream rather than interim delivery mechanisms. In the absence of official recognition, water vendors will continue operating anyway, but without quality controls, price monitoring or accountability.

Political economy of reform

In light of the widespread inefficiencies in the water sector in developing countries, there is clearly ample scope for reform. However, as in most democratic nations, any major reform needs to survive the political process, while even small changes in prices require public approval. As Noll et al. (2000) argued, several features of urban water systems make reform difficult.

The political benefits of water reform are often low, as reform may affect employment and investment in the public enterprise. Changes in prices and staff layoffs are more visible to the public than improved operating efficiency, reductions in state subsidies, and small improvements in quality.

The longer reform is delayed, the more difficult it becomes. In particular, critics viewed the price rises as laying the ground for privatization of water, with little apparent recognition that even an efficient public utility would require prices substantially higher than those prevailing pre-reform.

Promoting changes in the water sector therefore requires finding a way to raise the political benefits of reform efforts. Public awareness could be further increased by reporting the results of water quality tests, along with information on hours of service. This information could be coupled with accessible data on how much water subsidies take from the state budget, and price increases could be explicitly linked to targeted improvements in key sectors. This would work for the informed members of the society and those with different positive political inclination.

EXISTING SITUATION OF THE WATER SUPPLY AND ENVIRONMENT IN ANAMBRA STATE

Access to water supply services

There is very little data available to inform the drafting of the policy and its implementation strategy. However, the Federal Ministry of Agriculture and Water Resources (FMAWR) appointed a consultant to carry out a baseline survey of Water Supply and Sanitation (WSS) in the state. The findings of these study shows that access to safe water from public water supply in all the Local government of the state ranges from as low 0.2% to above 40%. It is estimated that on the average, access to potable water is not more than 20% across the state.

Condition of the existing government water supply infrastructures in Anambra State

Recent study by Water Supply and Sanitation Program (WSSP) shows that the State water supply infrastructures are in a very poor state. Out of 62 systems owned by the State Water Corporation only three of them are functional. Water supply infrastructure identified under the Federal Government baseline survey for Anambra State comprised 497 motorized boreholes, 72 hand pump boreholes and 7 surface water schemes. The total installed capacity excluding the Nkissi surface Water Scheme Which Serve Onitsha North, Onitsha South, Idemili North, Idemili South and Ogbaru LGAs is 30,795 m^3/day.

Greater Onitsha Water Supply Scheme (Nkissi Water Works) which was destroyed by massive siltation of the intake facilities is being considered for concession and rehabilitation. The State Government had entered into concession with a Jordanian Firm for the provision of water and an independent power distribution at Onitsha.

The take off is still awaited. This was abandoned for a South African firm with the mode of procurement changing to the traditional procurement route. The Greater Onitsha Water Scheme was the live wire of Anambra State Water Corporation generating over 90% of its revenue. The corporation has been out of operation since the past 15 years due to inefficient operation, corruption and other ills associated with public installations in Nigeria.

Capital funding and development of the sector

The sector has been poorly funded. According to the Anambra Sate Economic Empowerment Strategy (SEEDS) document, it was estimated that over N8.0 billion was required to improve the state water supply between the year 2004 and 2007. In reality less that 20% of that was actually spent in the sector during the period. Meanwhile water demand continued to increase annually due to increase in population and economic activities in the state, thus the water coverage has actually continued to decline during the period.

Supply-demand gap

It has been estimated that the demand for water in Anambra State in the year 2005 was 213,952 m^3 per day. This will rise to 27,313 m^3/day in 2015. The percentage of water available for the state or the theoretical percentage of water supply against demand was also found to be 7.2% (Anambra State WSSSP, 2009). This means that on a state wide basis only 7.2% of safe water requirement is met in 2005 or is available in Anambra State and this is very poor. Clearly then, extensive work would have to be embarked upon to find new water supply sources and establish new water schemes to close up the demand gap. This has been done and is found in the creater Onitsha water scheme, proposed greater Nnewi and Awka water scheme, the small town water schemes and water boreholes projects scattered over the state.

Water supply promoters in the Anambra State

According to the recent baseline survey on WSS conducted by the Federal Government under the Urban Water Sector Reform, Anambra State has 102 small water scheme or 17.7% of the federal government promoted water. And 28 of these schemes or 27.5% are functional.
The State Government promotes 52 of the schemes which is 9% of the total and only 13 of them or 25% are functional. Local Governments in Anambra State promoted 8 or 1.4% of all the schemes in the state and 3 of these or 37.5% are functional. Donors promoted 148 schemes or 25.6% of the total number of schemes of which 122 or 82.4% are functional. Members of the various communities promote 257 schemes or 44.5% of all schemes out of which 131 or 89.9% are functional. Together Donors and members of the various communities provided 405 schemes or 70.2% of all schemes out of which 353 or 87.2% are functional. Donors and Communities constitute the backbone of potable water supply in Anambra State and the functionality of these schemes is superior to other providers (AWSSR, 2009; WSP, 2001).

The Local Government Areas (LGAs) appear now to be stepping up involvement in water supply in this state. Clearly then, the most sustainable way forward is for the government to take the path of developing water supply in the state through community managed systems, this will best be done through government strong financial support to community water supply projects.

Private sector participation on water supply

Over 80% of water supply services provision in the State is in private hands and the service providers charge more than ten times the expected public sector charging rates.

Water pricing challenges

Bottled water: *Prices for bottled water are set in the market. Retail prices vary widely between countries, brands, bottle sizes and place of sale (supermarket, restaurant). They can be as high as ₦600 per cubic meter.*

Tanker trucks: Prices for water sold by tanker trucks in bulk, which is common in cities, for households without access to piped water supply. Prices for trucked water vary between about ₦1, 500,000 per cubic meter.

Utility tariffs: *Prices for piped water supply provided by utilities, be they publicly or privately managed, are determined administratively. They vary from ₦100 when it was available per cubic meter.*

Irrigation: *Prices for irrigation water that is being provided by a public agency are also typically determined administratively.*

The water rate charges by the water corporation were for the following:

1. Metered Supply: M_1 -domestic; M_2 -industrial/commercial.
2. Big Hotels; Bakeries, breweries; block moulding, car washing; laundry services, banks, filling stations.
3. Small hospitals; maternities.

One does not foresee what the new rate would come to and the success of selling these rates to the public since some individual and community schemes have made some organization forget the public water works.

Cost recovery and sustainability

From the tariff approved for the state water corporation, it is evident that the domestic connection charges are lower than the production cost by almost 50% while the Anambra State Government is required to produce and sell water in a way as to cover its operational cost. When a utility earns less revenue than its expenses in a particular year, it run short of funds to properly run its services in the following year and this leads to lower water production with the resultant low sales due to the low quality of service. The resultant effect is the dearth of the consumer's willingness to pay. The utility will then have even less revenue to operate in the following year. The vicious cycle will continue leading to the continued deterioration or service delay. This policy is addressing this by suggesting sweeping reforms.

Subsidy and access to WSS services

Common argument usually attributed to the application of flat subsidy to domestic consumers is to say that water is being made affordable as part of the government commitment to its citizens. Unfortunately the opposite is being achieved in this case. The reality is that majority of people in urban areas of Anambra State do not have water connections in their houses. Those connected are mostly those that can afford to pay higher price. Instead, they are benefiting from government subvention originally intended to assist the poor gain access to service. Furthermore the poor are the ones that purchase water from vendors and tanker operators at the exorbitant rates. The alternative is for them to get water from contaminated sources that are unhealthy. Sickness leads to low human productivity which in turn increases poverty.

Public/private sector participation

The private water supply operators are in business because the consumers are ready to pay for their services. This leads to the conclusion that in spite of the reported high level of poverty, large section of the population is willing and ready to pay higher prices for water supply if the service is reliable. This finding opens a whole new opportunity to carry out two sweeping reforms; the first is to produce a level playing field for both government and private companies. This would release latent potentials to compete favourably so that both can generate sufficient revenue to expand the services.

The second is the opportunity to redefine roles of all the actors in the provision of services especially in separating policy, service delivery and regulations. Experience has shown that reforms like this brought tremendous improvement in water supply in both developed and developing countries.

Design and application of appropriate tariff in Anambra State

Sufficient data on the production cost of water supply for urban areas is not available to enable proper costing of water production. This is perhaps because the major water treatment plant has been out of operation for a number of years. It is not clear what the guiding principles for setting tariff lower than production cost without giving details on how the shortfall in the revenue will be financed. The tariff structure should take into account the following:

1. To allow the water supply system to have resource and liquidity maintenance that consumers are able and willing to pay as failure to do so will mean to them spending more money to get potable water from other sources.
2. Put into consideration the poor who may be denied access to clean water if the rates are too high.
3. Be assured that it will not cause social unrest and is politically acceptable to the Government.

Unaccounted-for water

Unaccounted for water refers to water losses in the form of system leakages in the distribution network (which includes physical losses due to pipe bursts and leakage, and water theft through illegal connection) and losses in revenue due to poor billing and collection system and weak tariff structure. Unaccounted for water has not been assessed due non-availability of water in the network. Unaccounted for water can be taken as the difference between the account of water produced or purchased and the amount of water sold to all customers. Unaccounted for water includes underground leakage, unauthorized use, unavailable leakages, inaccurate master, industrial, commercial and domestic water and unusual causes. Underground leakage is caused by age of the pipe, soil conditions, traffic loading, pipe movement, poor installation practices and electrolysis.

Listening surveys and water audit was used in checking this in the past. Improved various types of the surveys and audit processes are imperative for the new water works scheme expected.

Other problems facing the state WSS sector in urban areas

The following problems are found to be the major causes of inadequate water supply and sanitation in the state:

1. There is no coordination and focus and because of this, planning, activities were “reactive” rather than “proactive” until the setting up of the new Ministry of Public Utilities, Water Resources and Community Development in August 2008, there was no central agency

to develop policy direction of Government."
2. There is lack of a philosophy, objectives and strategy guiding government activities in this sector.
3. Approach to plans for service delivery were "top bottom supply driven" rather than "bottom up demand driven."
4. The very strong community structure and vibrant private sectors, for which the state is well known, was not exploited in developing an efficient water delivery system for the people.

ANAMBRA STATE WATER CORPORATION (ANSWC)

Anambra State Water Corporation (ANSWC) established by Edict No 3 of 7th May 1999 has as its main function; to supply and mange water supply in the whole of the State (including urban, semi-urban and rural water supply). It was established to develop, manage, control, provide, conserve and distribute water in the state for public, domestic and industrial purposes. It had power to enter into contract or other agreements for the purpose of expedient performance of its function.

As at the year 2006, ANSWC had 12 Zonal Offices covering the twenty one LGAs of Anambra State for managing their water scheme of the 62 water schemes operated by ANSWC, 58 (93.5%) were non functional while only 4 (6.5%) are functional. The record is indeed an issue of great concern. Even more disastrous for ANSWC is the collapse of the Greater Onitsha Water Supply Scheme since 2001. When operational, the scheme provided 90% of ANSWC's internally generated funds. With so many public water boreholes and private ones in private building the people of the state would not be said to be short of water supply but the portability remains the question.

Water policy

The existing level of service of water supply and sanitation in the state is far from satisfactorily improving priorities of Anambra State Government. There was need for a comprehensive WSS services in a sustainable way. The institutional arrangement for Water Supply and Sanitation were not properly streamlined to have proper focus in a coherent and sustainable way. In a situation like this, one finds gaps and overlaps and lack of clarity on the responsibilities of the state actors. An institutional framework defines strategies and programmes that the sector must implement the organizations to implement these policies, and regulatory framework needed to support the policies as well as the institutional actions. The institutional framework looks beyond organizations and their functioning and takes into account the policy, legal and regulatory environment that supports the delivery of effective and efficient services which is the ultimate objective of any reform process. One of the primary impacts of the policy is the creation of the enabling environment for the implementation of the principles of the policy. This conducive working environment is created by the policy, legislature and financing structure Policy development gives an opportunity for setting objectives for managing water resources and water service delivery within a framework of overall development objectives.

Legislative framework sets the rules to follow in order to active policy objectives and goals. One of the main duty here is the development of the Water Law. The law covers the ownership of water schemes, the use of permit operation and maintenance and the regulatory norms. Also necessary is the need to repeal and reform the existing legislatures, edicts with the development of the State Water and Sanitation Policy. A component of the enabling environment is the financing and incentive structures that deals with the allocation of financial resources to meet the water needs. The financing needs of the water sectors are huge, and water projects tend to be capital intensive and require investment policy with financing options.

The fundamental objective of the policy is to bring reform to institutional roles and responsibilities for better water governance. The policy document defines the roles of resource managers, service providers, regulators, the roles of private sector, local authorities, civil society organization (CSOs) community based organizations (CBOs) and other water sector stakeholders. This forms part of the overall institutional framework.

Capacity building or human resources development involves the upgrading of the skills and understanding of all levels of capacities: Public decision makers, water professionals, regulatory bodies and other bodies like civil society groups.

An important aspect in the policy development is the overall application of management instruments. A primary aspect of this is a good understanding of resources and the needs in water resources assessment. This assessment starts with the collection of hydrological, physiological, demographic, and social-economic data. This will enable development in the sector to be modeled in line with the integrated water resources management (IWRM) principles. A very important concept here is the demand management. It is a process of using water more efficiently by balancing supply and demand and requires the following:

1. Improved efficiency of use.
2. Recycling and use.
3. Improved efficiency of supply.

Demand management application will work with good social change supply and sanitation systems as a matter of priority, the government is already planning for subsequent challenges with regards to expansion of the water supply throughout the state, there is a need to accompany any rehabilitation works with a good structure of operation and maintenance system. This is strongly emphasized in the policy. Lack of operation and maintenance was the bane of the dysfunctional water schemes. This aspect of the reform and indeed all aspect of the policy are necessary to ensure sustainable growth and development of the service delivery in the State.

CONCLUSIONS AND RECOMMENDATION

The literature on drinking water in Nigeria is characterized by an overall sense of policy failure and barriers to access, private sector participation, of fiscal reform, willingness to pay, of civil society participation. It remains a challenge to answer the key question for designing an affordable and sustainable urban drinking water program. With respect to private sector participation, relative to the efficiency or prices, analyses of the kinds of contracts, regulatory regimes and citizen oversight that can ensure accountability and the inclusion of low-income communities, are less common. It is agreed that governments should not be in the water provision business, but should ensure that private providers are regulated with respect to price structures and water quality, and should provide incentives for these providers to serve the poor. This new role for government translates to developing partnerships with the private sector and with civil society for water delivery.

A major barrier to the design of appropriate policies is the lack of reliable, up-to-date and publicly accessible information on many aspects of the Anambra water system. Baseline information is necessary in order to evaluate various reforms in progress, and in order to allow for benchmarking against government targets, that is, other states and nearby countries. A lack of transparency over the true costs of under-priced and inefficient municipal systems dampens public support for major reforms that may be needed.

Scanty information on groundwater withdrawals make urban and Semi-Urban drinking water interventions unsustainable. While efforts are underway to carry out some benchmarking of financial performance of several large utilities, regular and comparable data need to be made available on, water quality, subsidization, metering, groundwater levels, and infrastructure maintenance. The abundance of groundwater resources in Anambra State remains a big boast to solving the problems of domestic and industrial water with ease since exploration and exploitation is easily feasible through the state.

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Transient numerical approach to estimate groundwater dewatering flow rates for a large construction site: a case study from the Middle East

GUHA Hillol*, DAY Garrett and CLEMENTE José

Bechtel Power Corporation, Geotechnical and Hydraulic Engineering Services, U.S.A.

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A three-dimensional, transient groundwater model was developed to determine the rate, volume, and number of pumping wells required to estimate the dewatering of three deep excavations at a large coastal construction site in the Middle East with a shallow groundwater table. There was limited site-specific hydrogeologic information for the site. A calibrated MODFLOW-based groundwater model was developed using the "the model-independent parameter estimation and uncertainty analysis" (PEST) software with pilot points and regularization mathematical techniques. Simulated heads were fitted against the monitoring well heads along extrapolated site groundwater head contours by estimating the hydraulic conductivity at each pilot point. Model-calibrated hydraulic conductivities obtained were within the range of medium to fine sand with silt values and matched closely with the subsurface material descriptions obtained through site geotechnical investigations. Dewatering of the three pits, each with approximate dimensions of 10 by 8 m and a depth of 20 m, was simulated through a series of sensitivity analyses to determine the number of wells, discharge rate, time duration to dewater the pits, and the volume of discharge water per pit to be diverted. Conclusions from the dewatering simulation estimations were as follows: (1) sensitivity analysis showed that the range of dewatering from each pit was dependent on the selected hydraulic conductivity and storage values, (2) storage was most sensitive to achieve the dewatered groundwater elevation depths, and (3) a one order-of-magnitude decrease of storage resulted in a shorter duration to dewater a pit. In summary, model simulations showed that site-specific pumping tests should be performed to optimize the design of a dewatering well system, specifically in low hydraulic conductivity soils where using large capacity wells is not feasible. The use of a numerical transient groundwater model is warranted for dewatering estimations as site-specific conditions are complex.

Key words: Middle East, construction dewatering, groundwater modeling, MODFLOW, PEST.

INTRODUCTION

Groundwater dewatering designs are often required prior to undertaking any subsurface geotechnical constructions (Ergun and Naicakan, 1993; Powers et al., 2007). The amount of dewatering is a function of the depth of the

*Corresponding author. E-mail: hguha@bechtel.com.

groundwater as it relates to the depth and size of the construction area (Powers et al., 2007; Preene, 2012). Understanding the geology, hydrogeology, and heterogeneity of the subsurface prior to undertaking dewatering is critical in successful geotechnical construction designs. Dewatering in a heterogeneous system requires development of a numerical groundwater model to better predict the number of well points or pumping wells required and also the time-variant nature of the dewatering process (Boak et al., 2007). We present a case study of a dewatering prediction rate estimate with the development of a numerical groundwater model of a large coastal construction site in the Middle East. The main objectives of this paper are to:

1. Develop a calibrated numerical groundwater model at and around three pits to include the site groundwater elevation shown in Figure 1 (determined from geotechnical boring logs and a limited number of groundwater observation wells) and
2. Estimate the rate, amount, and number of pumping wells needed to lower the groundwater elevation by approximately 20 m for each of the three pits with plan dimensions of approximately 10 by 8 m.

METHODOLOGY

Hydrogeological conditions

The study area is composed of Miocene and Pliocene sandy limestone, marl, gypsum, and beachrock formations. Evaporatic and low supratidal flats (Sabkha) are predominant and close to the study area. In the immediate study area geotechnical boring logs show interlayered medium to coarse sand with silt followed by thick fat clay layer at a depth of -21.5 m below ground surface. Groundwater elevation at the proposed dewatering area varied between 5 to 10 m below ground surface and is under unconfined condition, however, at greater depths groundwater is under confined conditions. Annual precipitation rates are very low along with high evaporation throughout the year result in a permanent water deficit. Although recharge is nearly zero due to high evaporation than precipitation; however, there are occasions when short heavy showers result in some recharge to the groundwater in similar hydrologic conditions (Memon et al., 1986; de Vries and Simmers, 2002; Kalbus et al., 2011).

Groundwater model development

A three-dimensional numerical groundwater model was developed using the pre-processor Groundwater Vistas, version 5 (ESI, 2007), utilizing the U.S. geological survey MODFLOW 2000 numerical model code for groundwater flow. The model included two layers, a horizontal grid dimension of 10 by 10 m aligned in East–West and North–South directions, resulting in a total of 328 and 200 columns. The model dimension encompassed an area greater than that of the pit so as to minimize any modeled boundary effects near the pit dewatering area. The length and width of the model domain was 1,990 by 3,267 m, respectively. The average model vertical depth was approximately 27.6 m (elevation depth of -21.5 m) at the three dewatering pits. The two layers in the model were simulated as an unconfined aquifer as there were no confining lithologic units separating the two layers. The four boundaries of the model were depicted using the general head boundary (GHB) conditions (Figure 2). The GHB conductance was determined by multiplying the hydraulic conductivity of the layer with the area of the finite difference grid dimension of each cell and dividing by the thickness of the layer at each of the cell nodes corresponding to the GHB. As there was no surface water features or water level monitoring well outside the model domain, the immediate groundwater head assumed outside the model domain was based on the surface elevation (assuming that groundwater was at the surface).

The model was divided into two layers to differentiate between the two distinct lithologies as determined from the geotechnical boring logs. Based on the boring logs, the top layer was denoted as medium dense to dense sand with silt, and the lower layer was denoted very dense sand with silt. The boring logs indicated that at a depth of approximately -21.5 m elevation, hard, fat clay exists at the pit area. As there were limited boring logs within the model domain, the existence and depth of the clay layer beyond the pit areas was not certain. For the purpose of model development, the clay layer was assumed to exist within the model domain. The clay layer was assumed as no-flow boundary within the model domain. The surface elevation of the model varied between 0.35 to 13 m, whereas, in the immediate project area, the range of surface elevation was 4.5 to 7 m. The thicknesses of model layer 1 varied between 0.5 to 9 m, and for layer 2, the thickness varied between 18 and 26 m. The model did not include any net recharge as it was assumed that groundwater infiltration was minimal and did not significantly impact the site.

Groundwater model calibration

The numerical groundwater model was calibrated under steady-state conditions, using the groundwater head contour distribution, as shown in Figure 1, as the target heads. The target heads were distributed along the contours in Figure 1. MODFLOW 2000 code was used to simulate the groundwater heads (Harbaugh et al., 2000). Prior to performing the calibration run, a total of 426 hydraulic conductivity pilot points were distributed within the model domain (Figure 2). These pilot points were distributed uniformly outside the immediate area of the project area but were increased in the area where the groundwater head contours are shown in Figure 1. The model-independent parameter estimation and uncertainty analysis (PEST) software with pilot point and regularizations (Doherty, 2010; Watermark Numerical Computing, 2010) was used to calibrate the model by fitting the simulated head against the monitoring well heads by estimating the hydraulic conductivity at each pilot point. Hydraulic conductivity was distributed among the pilot points using the Kriging interpolation method of geostatistics (Watermark Numerical Computing, 2010).

The regularization approach provides an advantage in estimating more parameters than there are observations to calibrate against. It is also a technique in minimizing the global objective function (ϕ_g). The global objective function can be defined as:

$$\phi_g = \phi_m + \mu\phi_r \quad (1)$$

where ϕ_m is the measurement objective function, ϕ_r is the regularized objective function, and μ is similar to a Lagrange multiplier, which in PEST is estimated through the Gauss-Marquadt-Levenberg optimization routine (Doherty, 2010).

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The measurement objective function, ϕ_m, is defined as

$$\phi_m = (d - M(p))^t Q_1 (d - M(p)) \quad (2)$$

where d represents vector of field measurement, M is the modeled simulated values, Q_1 is the weight of the observation points, and

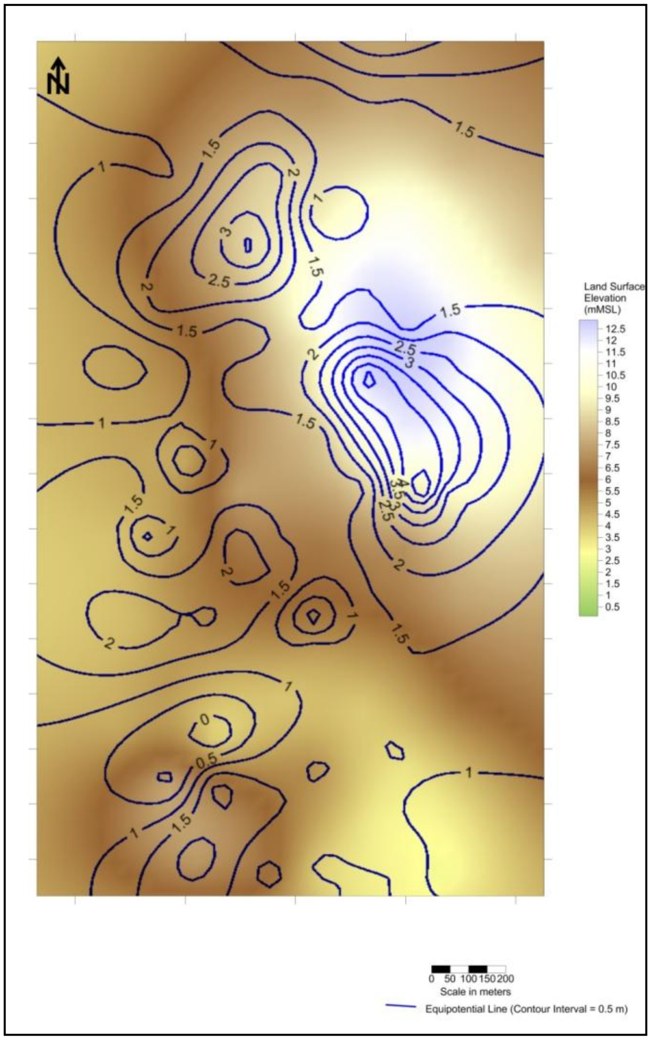

Figure 1. Site Observed Groundwater Levels

d–M(p) is the difference between field and modeled data acting on parameter vector p.

The regularized objective function, ϕ_r, is defined as

$$\phi_r = (e - R(p))^t Q_2 (e - R(p)) \quad (3)$$

where *e* is the regularized observation values, *R* is a regularized operator acting on the parameter vector *p*, Q_2 is the weights assigned to the regularized observations, and *e–R(p)* is the difference between regularization observation and regularized parameter vector values.

Figure 3 shows the simulated groundwater head distributions within the model domain under steady-state conditions. Figure 4 shows the simulated groundwater head distribution at and around the three-pit area. The distribution of the simulated groundwater heads was within 0.1 m of the measured heads (Figure 1) at the dewatering area and outside; that is, in areas to the northeast of the model domain, it varied between 0.5 and 1 m. The greater

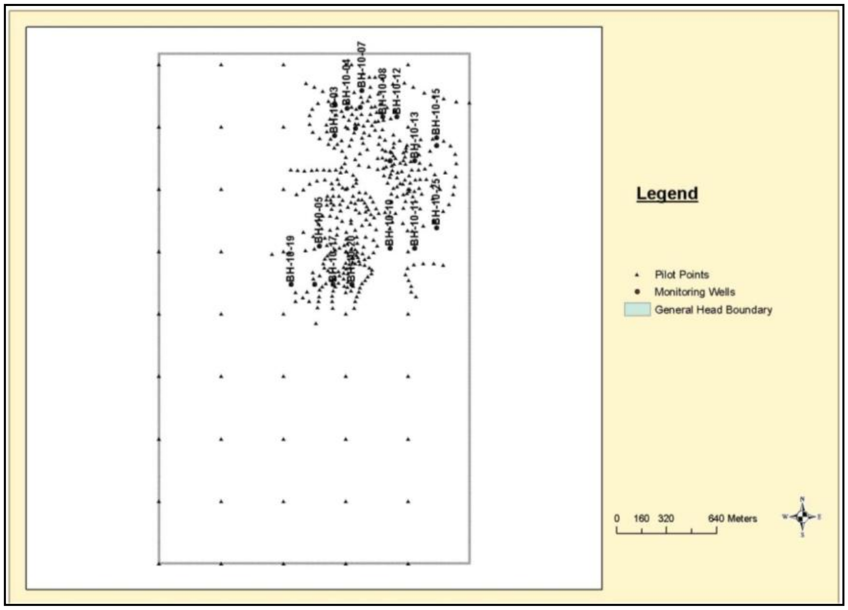

Figure 2. Model domain and pilot points distributions.

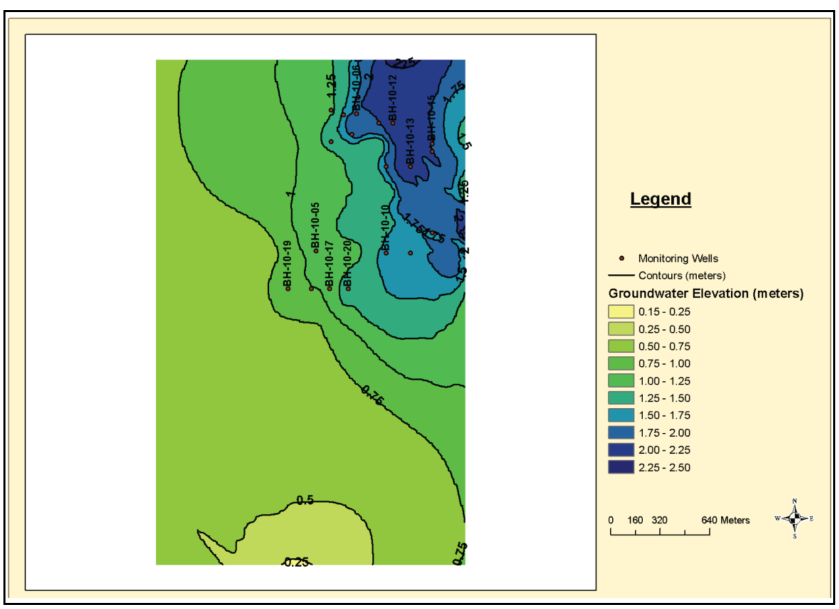

Figure 3. Groundwater head contours in the Calibrated Groundwater Model.

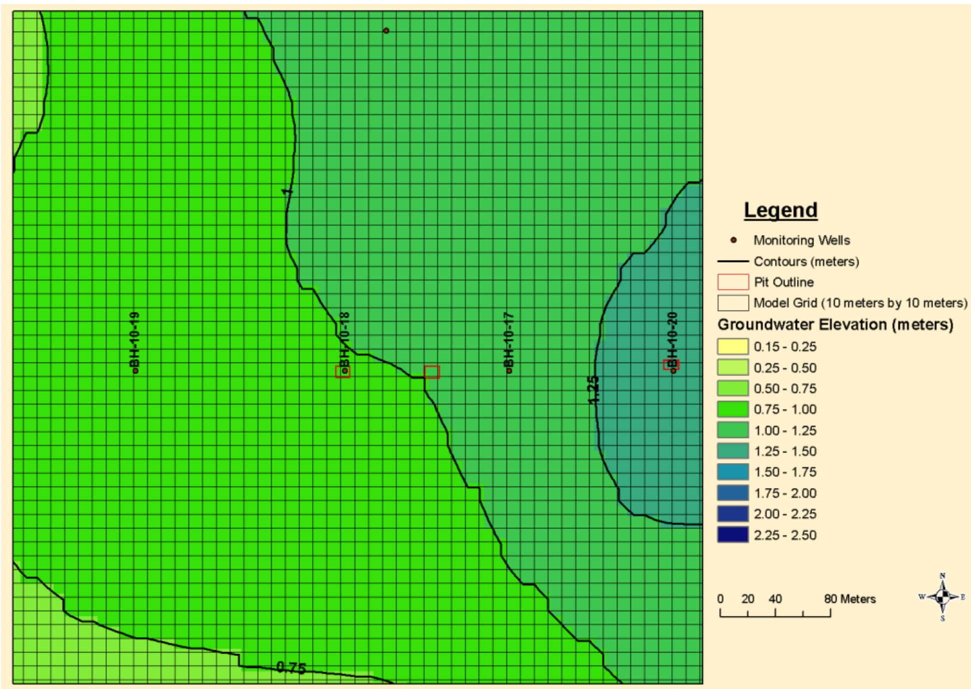

Figure 4. Groundwater head contours in the Calibrated Groundwater Model in the pit areas.

difference in heads between contoured heads (Figure 1) with that of the simulated head in the northeastern portion of the model domain was attributed to hydrogeologic conditions that may not be correctly conceptualized in the current groundwater model. However, the model-simulated heads did capture the mounding effects to the northeast. Some boundary effect is also responsible for the difference in the heads in the northeast portion of the model; however, the effects of the boundary did not have any impact at and around the three-pit area.

Figures 5 and 6 show the calibrated horizontal and vertical hydraulic conductivity distributions in layer 1 at the proposed dewatering zone. Similarly, Figure 7 depicts the calibrated horizontal hydraulic conductivity distributions in layer 2. The vertical hydraulic conductivity of layer 2 was 1.2×10^{-5} cm/s. The values of the horizontal hydraulic conductivity distributions were within the range of the medium to fine sand with silt values as provided in Table 4.6 of Fetter (1994). The estimated hydraulic conductivity values were determined inversely (no trial-and-error approach) by the model-independent parameter estimation and uncertainty analysis (PEST) suite of algorithms and the values are within the reported values of the lithologic units. The vertical hydraulic conductivity values were, on average, an order of magnitude smaller than the horizontal hydraulic conductivity values. Layer 1, which was described as medium dense to dense sand with silt, was characterized by horizontal hydraulic conductivities ranging from 1.1×10^{-4} cm/s to 1.5×10^{-3} cm/s at and around the three pits (Figure 5). The higher values of horizontal hydraulic conductivities than the vertical hydraulic conductivities is attributed to the anisotropy of the soil types, where the dominant flow is along the horizontal direction which is typical in this kind of hydrogeological settings. In layer 2, the horizontal hydraulic conductivity values at and around the three pit areas were mostly within the range of 1.1×10^{-4} cm/s to 4.4×10^{-4} cm/s; in the immediate area of Pit 3, however, the value ranged from 1.3×10^{-3} cm/s to 4.6×10^{-3} cm/s (Figure 7).

RESULTS AND DISCUSSION

Pits dewatering estimate

Once the groundwater model was calibrated against the groundwater heads, the next step was to estimate dewatering rates for the three pits (Figure 4). Pits 1 and 2 were defined by a surface perimeter of approximately 10 by 8 m, and Pit 3 was approximately 10 by 7 m. Because the grid dimensions of the calibrated model were 10 by 10 m, finer grid discretization of 0.5 by 0.5 m was utilized at and around the three pits to numerically estimate the required dewatering at the pits. The grid was also expanded at selected areas beyond the vicinity of the pit. The maximum grid dimensions in the finer resolution model were 14 by 14 m. The total number of rows and

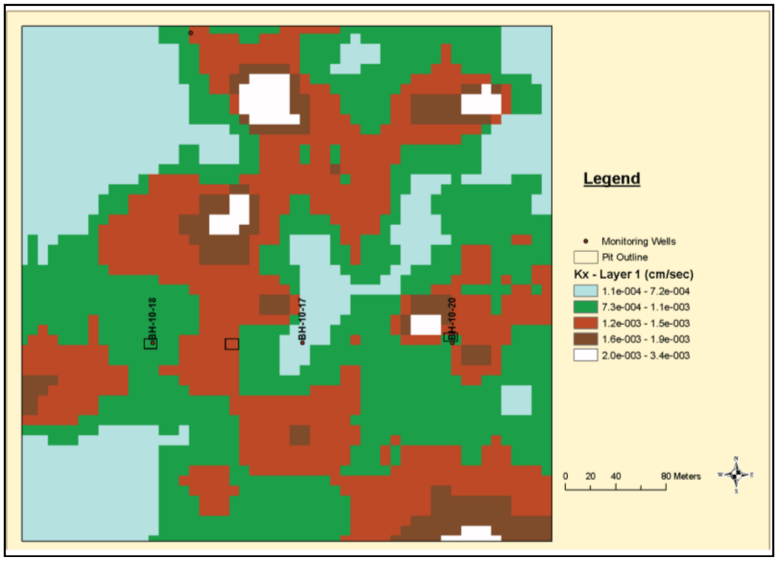

Figure 5. Distribution of Horizontal Hydraulic Conductivity Values in Layer 1 of the Groundwater Model at the pit areas.

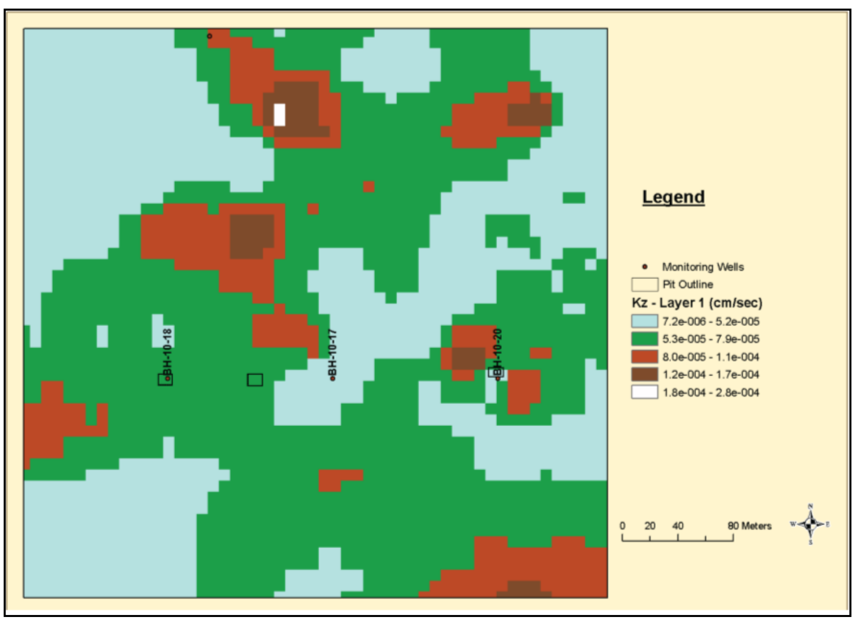

Figure 6. Distribution of Vertical Hydraulic Conductivity Values in Layer 1 of the Groundwater Model at the pit areas.

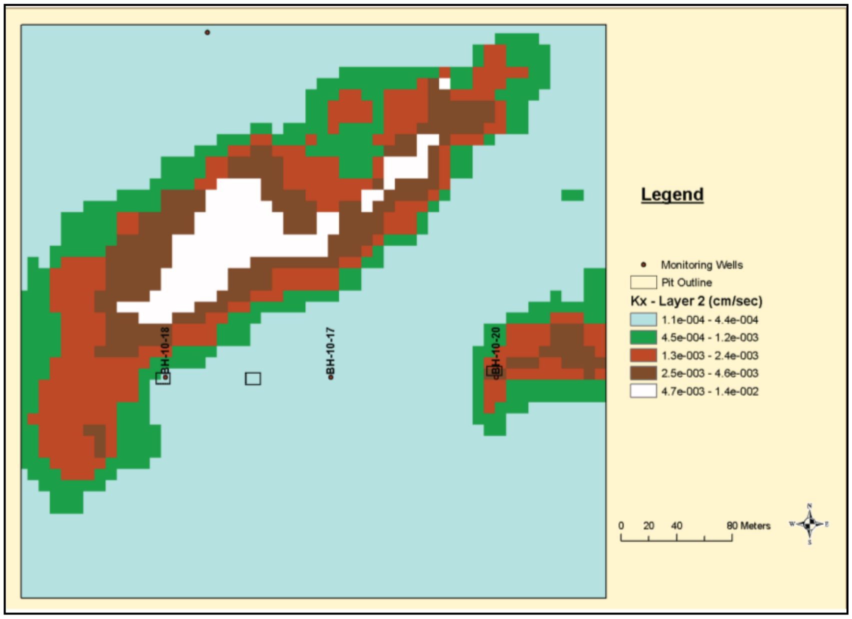

Figure 7. Distribution of Horizontal Hydraulic Conductivity Values in Layer 2 of the Groundwater Model at the pit areas.

columns in the model was 524 and 947, respectively. A steady-state simulation of the finer resolution model was run to compare the groundwater heads with those of the 10 by 10 m grid dimension model. The results of the heads compared satisfactorily within the model domain.

The three pits were dewatered in order, starting with pit 1, followed by pit 2 and then, ultimately, pit 3. It was assumed that once each pit was dewatered, concrete would be poured in each pit for construction (base and side walls). In the groundwater model, the pit area that had been dewatered was simulated as a no-flow zone for the next pit dewatering simulation. It was assumed that during the dewatering process, there was no rainfall at the site, and groundwater pumped from the pit was discharged offsite to avoid recharging the aquifer in the immediate area of the three pits.

Pit dewatering was simulated by pumping from wells along the perimeter of each pit. The wells were spaced at distances of approximately 1 m from each other. The number of wells, pumping rate, depth, and location of the wells are shown in Table 1. As dewatering is a transient process, aquifer storage terms were represented uniformly within each of the two layers. For layers 1 and 2, specific yield values of 0.21 and 0.18, respectively, were selected. These are average values for fine sand and silt as stated in Table 4.4 of Fetter (1994). The selection of the pumping rate of each well at 19.1 m^3/day was arbitrary and was based on experience with dewatering in such geologic settings. However, the grain size analysis data show that the sediments at the site are tight, and thus, higher pumping rates may not be feasible. Prior to beginning any field dewatering activities, an aquifer test including step-test analysis for well pumping should be undertaken to estimate the most feasible rate of groundwater pumping.

Pit 1 dewatering rates

A total of 34 pumping wells, each pumping at 19.1 m^3/day, were distributed along the perimeter of the pit. Pumping proceeded for a total of 30 days before the groundwater head decreased to a level of approximately -18.5 m elevation. Figure 8 shows the time versus head and discharge in the location of the center of pit 1. The total water discharged at the end of the 30 days of pumping was approximately 8,500 m^3.

Pit 2 dewatering rates

Prior to simulating dewatering at pit 2, the area of pit 1 was simulated as a no-flow zone (to mimic the construction of the pit). The configuration and rate of the

Table 1. Pumping Well Information Used in the Groundwater Model.

Pit	Number of wells	Pumping rate of each well (gpm)	*Well screen elevation (m - msl)	Locations of wells
1	34	3.5	6.2 to -20	Along the perimeter of the pit
2	34	3.5	6.2 to -20	Along the perimeter of the pit
3	36	3.5	6.2 to -20	Along the perimeter of the pit

* The well screen elevation is based on aquifer depth identified in the groundwater model. For actual dewatering at the site, the well screen depth may vary depending on actual site hydrogeologic conditions encountered.

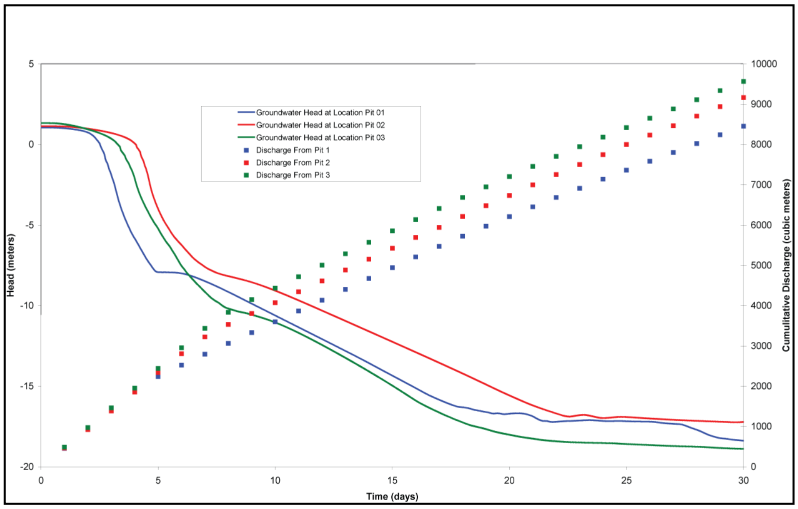

Figure 8. Groundwater Head and cumulative discharge with time for the three pits.

pumping wells were similar to those in the dewatering of Pit 1. Pit 2 was also dewatered for 30 days, with the groundwater head elevation decreasing to approximately -17.3 m at the target location in Pit 2 (Figure 8). The failure to reach the target elevation depth of -18.5 m after 30 days was attributed to a few dry wells and also to the lower hydraulic conductivities in the area of Pit 2. It is expected that sump pumps could be used in the bottom of the excavation to lower the groundwater head an additional meter to the target elevation. After 30 days of pumping, the total discharge from Pit 2 was approximately 9,200 m^3.

Pit 3 dewatering rates

Pit 3 was dewatered with a total of 36 pumping wells, each discharging at 19.1 m^3/day. The two additional wells in Pit 3 were the result of the slightly different pit outline and the fact that the initial groundwater table was higher than the level of the other two pits. Figure 8 shows the relationship between time and groundwater heads and discharge at Pit 3. The total groundwater discharge from Pit 3 at the end of 30 days of pumping was approximately 9,600 m^3. Pits 1 and 2 were simulated as no-flow boundary conditions to represent post-construction conditions.

Sensitivity analysis

To determine the impact that the hydraulic conductivity and storage values have on dewatering estimates for the three pits, a series of sensitivity analyses were conducted. Pumping rates were similar to the rates used in the dewatering of Pits 1, 2, and 3 as already discussed under “Pit dewatering estimate” above. The sensitivity analyses included:

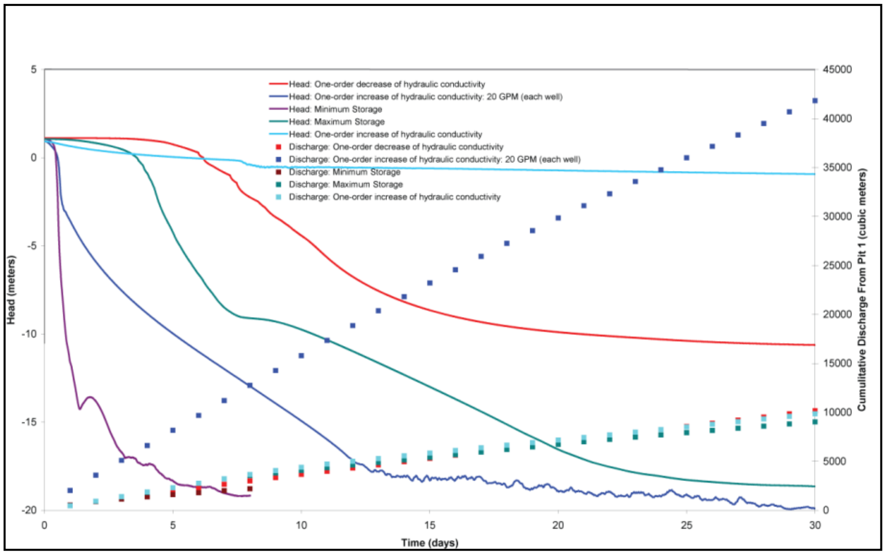

Figure 9. Sensitivity of Groundwater Head and cumulative discharge with time for Pit 1.

1. Increase of hydraulic conductivity (both horizontal and vertical) by one order of magnitude from the calibrated model values;
2. Decrease of hydraulic conductivity (both horizontal and vertical) by one order of magnitude from the calibrated model values;
3. Minimum storage (specific yield) values for fine sand (layer 1 in the model) and silt (layer 2 in the model) of 0.1 and 0.03, respectively; these values are shown in Table 4.4 of Fetter (1994) and
4. Maximum storage (specific yield) values for fine sand (layer 1 in the model) and silt (layer 2 in the model) of 0.28 and 0.19, respectively; these values are shown in Table 4.4 of Fetter (1994) Figures 9 to 11 show plots of time versus groundwater head and discharge for Pits 1, 2, and 3, respectively.

Pit 1 sensitivity analyses

Increasing the hydraulic conductivities by one order of magnitude resulted in very minimal groundwater dewatering (lowering of the groundwater elevation to -1.0 m) after 30 days of pumping. However, increasing the pumping rate of the individual wells from 19.1 to 109 m^3/day lowered the groundwater head to an elevation of -18.5 m within 15 days of pumping; however, pit dewatering reached a depth elevation of -20 m until 30 days of pumping. Although the required simulated depth of dewatering was achieved within 15 days of the start of pumping, it took another 15 days to achieve an extra 1.5 m (that is, to -20 m elevation) due to the groundwater fluctuations occurring very close to the fat clay below layer 2. The total groundwater discharge after 30 days with pumping rate of 109 m^3/day from each well was approximately 42,000 m^3 (Figure 9).

Decreasing the hydraulic conductivity by one order of magnitude resulted in a groundwater elevation at the pit after 30 days of pumping of about -10.5 m elevation. The total groundwater discharge after 30 days was about 10,000 m^3. Due to lower hydraulic conductivities (one order-of-magnitude decrease from calibrated values), the model showed that it will take more time (that is, more than 30 days pumping at a rate of 19.1 m^3/day) to lower groundwater heads at Pit 1 to a target elevation of -18.5 m.

Groundwater storage played a greater role in the estimate of groundwater heads with time during dewatering. Decreasing the groundwater storage (that is, specific yield) to 0.1 and 0.03 in layers 1 and 2, respectively, yielded faster pit dewatering. The model sensitivity run showed that Pit 1 achieved the target dewatering elevation of -18.5 m within 6 days of pumping. The total groundwater discharged after 6 days of dewatering was approximately 1,800 m^3. When specific yield values were increased to 0.28 and 0.19 for layers 1 and 2, groundwater heads reached the dewatering target of -18.5 m elevation within 28 days. The total groundwater discharged after 28 days was approximately 8,600 m^3.

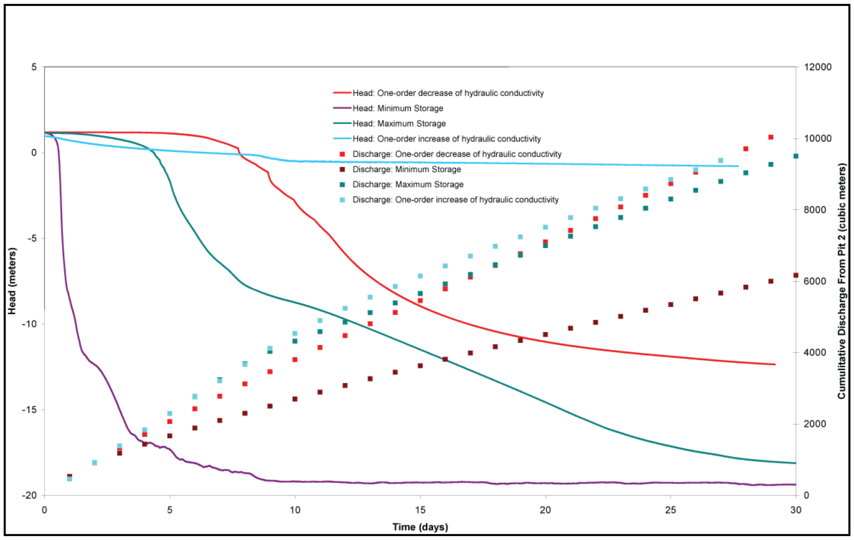

Figure 10. Sensitivity of Groundwater Head and Cumulative Discharge with Time for Pit 2.

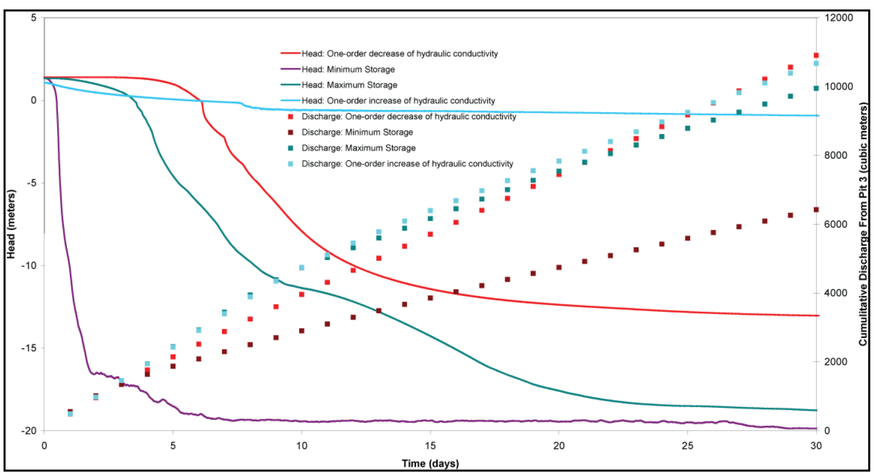

Figure 11. Sensitivity of Groundwater Head and Cumulative Discharge with Time for Pit 3.

Pit 2 sensitivity analyses

Pit 2 sensitivity analyses were conducted under conditions similar to the Pit 1 sensitivity analyses. The results of increasing and decreasing the hydraulic conductivity and storage values were similar to those in the Pit 1 sensitivity analyses (Figure 10).

Pit 3 sensitivity analyses

Pit 3 sensitivity analyses were conducted under conditions similar to the Pits 1 and 2 analyses. The results of increasing and decreasing the hydraulic conductivity and storage values were similar to the results from the Pits 1 and 2 sensitivity analyses (Figure 11). The sensitivity analyses show that a one order-of-magnitude increase of hydraulic conductivities above the calibrated hydraulic conductivity values will require a longer time to achieve the target dewatering elevation of -18.5 m, with approximately 34 to 36 wells pumping at a rate of 19.1 m^3/day. However, if the pumping rates for the individual wells are increased to 109 m^3/day, then it is possible to achieve the target dewatered elevation depth for the pits, as shown in the sensitivity analysis of pit 1. Storage plays a large role in dewatering estimation; a slight increase or decrease of storage impacts the time it takes to reach the groundwater level dewatering target. The lower the storage values for the sediments, the faster the groundwater dewatering target elevation is attained.

Assumptions

The following assumptions were used to evaluate the groundwater dewatering heads in the model:

1. The groundwater at the site represents uniform density;
2. There is no tidal influx affecting the dewatering pits;
3. The hydraulic conductivity and storage used in the model represent the values at the site;
4. There is no recharge during the dewatering processes;
5. Lithologic and groundwater information in the model at distances away from the pit areas is not ascertained from the available geotechnical field investigations and
6. Pumping rates used in the calculation during dewatering are constant. Initial water levels prior to dewatering of the pits are similar to those shown in Figure 1.

Conclusions

A groundwater model was developed for the purpose of estimating both the dewatering rates and the volume of groundwater that required removal from the three pits to achieve a construction dewatering groundwater elevation of -18.5 m. The model was calibrated against estimated maximum observed groundwater heads at the area of the pits. The estimated range of hydraulic conductivities was based on the lithologies from the geotechnical borehole logs. Dewatering of the three pits was evaluated by performing model simulations, starting with Pit 1, then Pit 2, and lastly, Pit 3. For Pit 1 dewatering, a total of 34 pumping wells, each pumping at 19.1 m^3/day for 30 days, were simulated until the target elevation was reached. The pumping wells were distributed along the periphery of the pit. The same well configuration was used to simulate dewatering at Pit 2. For Pit 3, 36 pumping wells were used because the pit outline (perimeter) is slightly different than those of Pits 1 and 2. In addition, the preconstruction dewatering groundwater heads at Pit 3 are slightly higher than the heads at Pits 1 and 2. The approximate total amount of water diverted to lower the dewatered groundwater elevation for each pit to -18.5 m was about 9,500 m^3. This estimation was based on model simulation values.

A series of sensitivity analyses were conducted to determine the effects of increasing and decreasing aquifer hydraulic conductivities and storage values on the dewatering estimates. The sensitivity analyses show that a one order-of-magnitude increase of hydraulic conductivity above the calibrated values would require more than 30 days of pumping to achieve the target dewatering depth. However, if the pumping rate was increased from 19.1 to 109 m^3/day for each of the 34 wells, the target depth of -18.5 m would be achieved within 15 days. Decreasing the hydraulic conductivity by one order of magnitude shows that the groundwater target depth is not achieved within the 30 days. Complete dewatering will require more time if the pumping rate from each well remains at 19.1 m^3/day.

Storage (or specific yield) values have a significant impact on the dewatered groundwater elevation depths. Decreasing the specific yield to 0.1 and 0.03 for layers 1 and 2 caused the groundwater elevation at the Pits to reach the target dewatered elevation within 6 days. Increasing the specific yield to 0.28 and 0.19 for layers 1 and 2 showed that the time to achieve the target elevation was approximately 28 days. The sensitivity analyses show that the range of total water dewatered from each pit depends on the selected hydraulic conductivity and storage values and varies from approximately 9,000 to 27,000 m^3 (for a one order-of-magnitude increase of hydraulic conductivity with a 109 m^3/day pumpage from each well) to achieve the target dewatered elevation of -18.5 m.

The model results show that it is possible to dewater the three pits using groundwater wells; however, specific information of dewatering pumping rates needs to be deciphered from site-specific pumping tests (step and constant rate discharge tests) to determine the hydrogeologic properties. The actual amount of water discharged will also depend on groundwater level conditions at the site. The current groundwater model is a planning-level tool to estimate dewatering potential at the three pits; however, for project design, site-specific information such as pumping tests should be performed to better estimate the site's dewatering capability. The number of wells used for dewatering simulations is not expected to be the exact number that is required for the actual dewatering process. For a required inflow into an excavation, there is more than one potential solution; a greater number of smaller wells, for example, achieve the same results as a smaller number of larger, higher-volume wells.

It is therefore recommended that a pumping test (step and constant rate discharge tests) be performed at any one of the pit areas. The tests would further determine how much water can be diverted from each well and would fine-tune the well pumping rate. The rate of flow into a pumped well or well point depends on the area and permeability of the ground immediately outside the well and on the hydraulic gradient causing the flow. Evaluation of the pumping test would provide a refined estimate of average hydraulic conductivity and storage of the pumping domain. The groundwater model would then be refined to better estimate the dewatering rates.

If the site step tests and pumping tests show that each of the individual wells can be pumped at rates greater than 19.1 m^3/day, then the groundwater model would be run with the new pumping rate. The model would help determine the required number of wells needed and the placement of the wells before groundwater dewatering is initiated. Additional dewatering methodologies should also be considered, including a series of well points, sumps, and/or cutoff walls and depends on the hydraulic conductivity and storage values determined from site-specific pumping tests.

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