Design and operation of a large dewatering system in Dubai

Etude et Réalisation d'un important système de rabattement de nappe à Dubai

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ABSTRACT

The new Terminal 3 works at Dubai International Airport required an excavation of plan size 420,000 m^2 by 25 m depth immediately adjacent to the existing terminal and control tower in water bearing granular soils. Dewatering was carried out using a perimeter ring of deepwells which was extended progressively to suit the available access and phasing of the works. The design of the dewatering system was undertaken using a 3-D time dependent finite element hydro-geological model which was calibrated using data from the progressive start-up of the dewatering system. Output from the model was used to ensure timely start-up of wells to meet the drawdown requirements for the phasing of the works.

RÉSUMÉ

La nouvelle aérogare numéro trois à l'Aéroport Internationale de Dubai nécessitait une excavation d'une surface en plan de 420,000 m^2 et 25 m de profondeur, à proximité de l'aérogare et la tour de contrôle existantes et ce dans des terrains aquifères granulaires. Le rabattement de nappe a été réalisé par l'utilisation de puits profonds en périphérie de l'ouvrage, dont l'implantation évoluait en fonction des nécessités du chantier (accès et aux différentes tranches des travaux). Le système de rabattement de nappe a été conçu en utilisant un modèle hydrogéologique tridimensionnel en fonction du temps par éléments finis; la calage du modèle a été réalisé avec les données recueillies au démarrage des travaux du système de rabattement. Les résultats issus du modèle ont permis d'adapter le réseau de rabattement de nappe en fonction des exigences du chantier et des différentes phases de travaux.

Keywords : Groundwater, dewatering, case history, permeability, settlement

1 INTRODUCTION

Dubai International Airport Terminal 3 and Concourse 2 opened in October 2008 following a 6 year construction program. This project is part of Dubai Civil Aviation's Phase 2 development of the airport which is designed to almost double capacity from 33 to 60 million passengers per year. Terminal 3 and the Concourse 2 basement comprise a multilevel underground structure. The passenger drop off and pick-up points, a future metro station, the departure and arrival halls are all located more than 10 m below the aircraft taxi way and apron. The construction works commenced with excavation over an area of 420,000 m^2 and the removal of 13M m^3 of spoil. Excavation side support was provided by 3.5 km of diaphragm wall with more than 2,500 ground anchors. Structural support, including uplift pressures, required the installation of more than 8,710 bored piles within the excavation. Original ground level was at +3 mDMD and excavation formation level was down to approximately -21.5 mDMD over much of the site.

Critical to the excavation and piling works was the requirement to lower the groundwater level over the whole structure foot print. In the permanent condition the diaphragm wall is supported by the base slab, internal floors and roof of the terminal. Temporary structural support for the diaphragm wall was provided by up to three rows of ground anchors. In addition a reduction in external groundwater level to -18 mDMD was required in the deeper areas of excavation to reduce the hydrostatic loads on the diaphragm wall. This paper describes the design basis, installation and performance of the deepwell dewatering system which was used to control groundwater levels during the 4 year period of the below ground construction program.

2 GROUND CONDITIONS

The new Terminal 3 site is located at the existing International Airport some 2 km north east of Dubai creek and 6 km from the seafront. The area is low lying and the shallow geology is dominated by aeolian dune sands of Holocene and Pleistocene age. These deposits comprise a shallow surface layer of loose silty sand underlain by dense and variably cemented carbonate sand and weak sandstone.

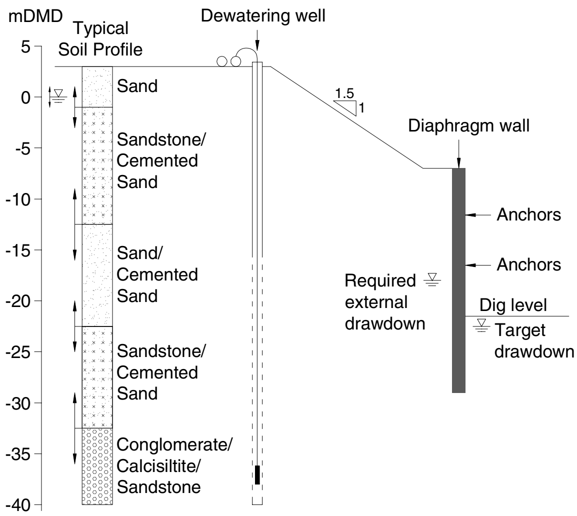

Figure 1. Typical section

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The degree of cementing generally increases with depth although borehole logs indicated evidence of uncemented silty sand zones down to below excavation formation level in some areas. Present from between -29 and -36 mDMD is a variable succession of rock comprising weak weathered conglomerate interbedded with calcisiltite and locally sandstone. The initial standing groundwater level was at between 1 and -1 mDMD. A typical section through the diaphragm wall showing the ground profile is shown in Figure 1.

The combination of a relatively high water table and the high levels of evaporation that occur in this arid region results in groundwater which is rich in chloride and sulphate with a salinity greater than seawater (Fookes et al 1985).

Two pumping tests were undertaken to provide a design basis for the dewatering works. The tests comprised 5 day constant discharge tests carried out on wells drilled to the top of the conglomerate at approximately -34 mDMD. Analysis of the test data gave very consistent values of 6.7 x 10^{-4} m^2/s transmissivity and a permeability (k) of 2 x 10^{-5} m/s assuming an aquifer depth of 34 m.

3 PROGRAMME PHASING AND ACCESS

The new terminal construction site formed part of a busy international airport where space was at a premium. For this reason access to the site could only be made available in stages as existing infrastructure was decommissioned and dismantled. The overall layout and access phasing is shown in Figure 2. Programme pressures meant that it was imperative to excavate to formation level for each phase immediately to allow the extensive piling works to commence. Between phases side support was by a 1:3 batter slope.

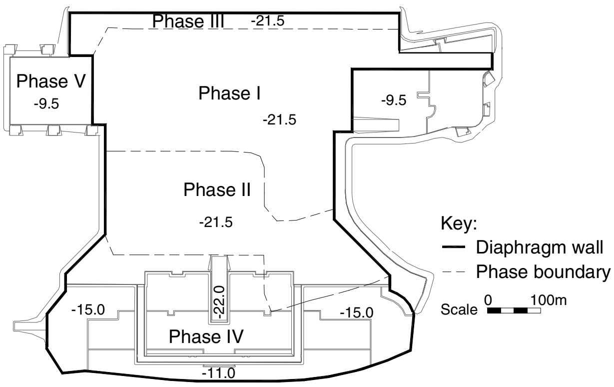

Figure 2. Phasing of the works and dig depth

In order to minimize disruption to the excavation and piling program the dewatering system was to be located around the perimeter of the excavation external to the diaphragm wall as far as possible. This was necessary in any case in order to meet the external drawdown requirements for the diaphragm wall and batter slopes.

4 DEWATERING DESIGN BASIS

The initial flow estimates for the dewatering system was based on simple 2 dimensional plane flow analysis for fully penetrating slots as described by Preene et al (2000) and is reproduced in Table 1 for the full excavation perimeter. A first estimate for individual yield for a 300 mm diameter well installed to fully penetrate the sand/sandstone aquifer can be estimated using Darcy's Law and a hydraulic gradient of 6 (Preene et al, 2000). This gives a well yield of 1.53 l/s for 13.5 m of wetted screen (the situation at full target drawdown). Combining this with the total flow estimate in Table 1 implies a need for at least 127 wells at a maximum spacing of 27.6 m. In view of the uncertainties in the calculations and the program pressures it was planned to install wells initially at 50 m spacing with the expectation that infill wells would be installed to reduce the spacing to 25 m. The actual well layout for the works is given in Figure 3 and it can be seen that the phasing resulted in a requirement for internal wells which were removed and decommissioned as access became available to subsequent phase areas.

Table 1. Plane flow estimate of inflow to full excavation

For unconfined flow $Q = k \times (H^2 - h^2) / 2L$ where	
Q	Total flow to excavation
k	Permeability taken from the pumping tests as 2 x 10^{-5} m/s
x	Excavation perimeter, 3,500 m (length of diaphragm wall)
H	Thickness of the aquifer, 0 to -36 mDMD, 36 m
h	Residual depth of groundwater, -22.5 to -36 mDMD, 13.5 m
L	Distance of influence = $C (H - h) \sqrt{k}$
C	Empirical calibration factor, 2,000, giving L = 201 m
	Giving Q, total abstraction flow = 194 l/s

It was recognized that the computation in Table 1 is a steady state analysis with the likelihood that flows would be appreciably higher than estimated during the initial drawdown period. Also critical to the program for the excavation and piling was accurate estimates of the time required to achieve successive levels of drawdown during each phase of the works. This analysis was achieved by developing and calibrating a numerical model as described below.

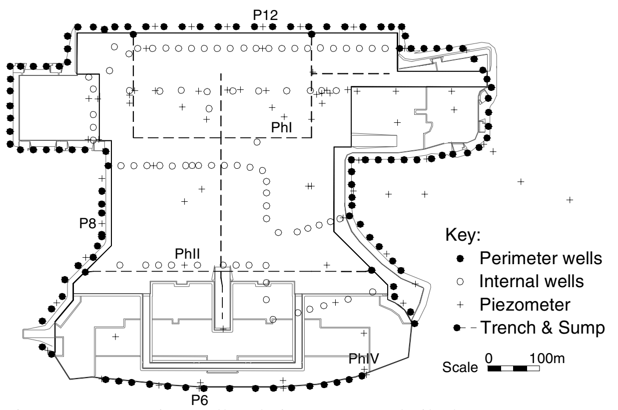

Figure 3. Dewatering well and piezometer 'as built' layout

5 MODELLING WORKS

The numerical model used was derived from the MODFLOW three dimensional finite difference generic model contained within the Groundwater Vistas Version 3.09 package, Rumbaugh & Rumbaugh (1999). The total plan area modelled was 3 km by 3 km with six layers which varied in depth to allow the ground conditions, well locations and diaphragm wall to be adequately represented. In plan the dimensions of the cells varied from 1 m by 1 m (to represent an individual well) up to 50 m by 50 m in the far field near the boundary. The initial head in each cell was set to 1 mDMD and a constant head boundary of 1 mDMD was applied at the 3.5 km square model perimeter. Initial calibration was carried out by modelling the two pumping tests and this was followed by a simulation of the Phase I dewatering works. Adjustments were made to the calibration for subsequent Phase models based on comparisons between the actual and modelled flow rates and groundwater levels. The final model had 18 time periods to represent the addition of perimeter wells, the removal of redundant internal wells and the construction of the diaphragm wall during the five phases of the works. The parameters used for the final model runs are given in Table 2.

The reduced vertical permeability was derived from the original pumping test calibration model runs. The key change from the pumping test calibration run is the division of the sand/sandstone layer into an upper higher permeability zone above -22 mDMD and a lower reduced permeability zone below this level. This change was introduced because the monitoring results had shown that the rate of drawdown slowed appreciable below about -19 mDMD and the use of internal trench drains feeding to sumps was required to control the residual groundwater ingress below this level.

Table 2. Phase 5 model parameters

Material	Top	Base	k_h m/s	k_v m/s	S /m	S_y
Sand high k	3	-22	$3.5e^{-5}$	$4.6e^{-6}$	$1e^{-6}$	0.1
Sand low k	-22	-36	$1.2e^{-5}$	$4.6e^{-6}$	$1e^{-6}$	0.1
Conglomerate	-36	-100	$1.0e^{-6}$	$1.0e^{-6}$	$1e^{-6}$	0.1

S is the specific storage per meter depth, Sy is the drainable porosity
k_h and k_v are the horizontal and vertical permeability respectively

The model was used to generate groundwater level contour plots for various scenarios and an example is shown in Figure 4. Figure 4 was prepared in May 2003 and is a simulation of the anticipated situation at the end of 2003 when the full perimeter system would have been in operation for about 6 months. In addition head transient plots (water level against time) were produced for individual piezometer locations and were compared against the field monitoring results for calibration. The future head transients were then used to study future groundwater levels at specific locations for various scenarios and well arrays. This information was essential to ensure that groundwater levels stayed below dig level as the excavation for each phase was opened up.

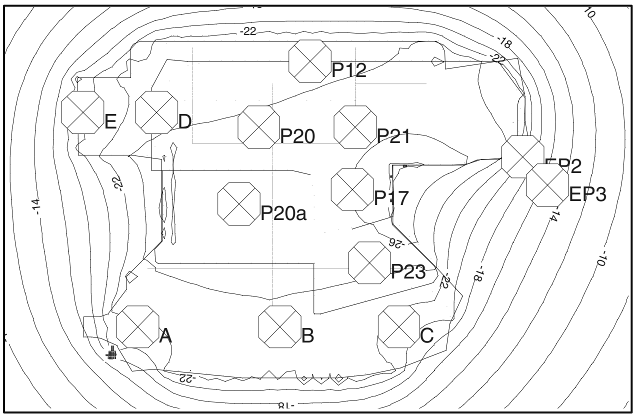

Figure 4. Model groundwater level contours 31 December 2003

6 SETTLEMENT RISK

It was apparent from the modelling works that the groundwater level would inevitably be lowered appreciably below the existing airport infrastructure which is in close proximity to the excavation. For example the drawdown below the control tower was initially estimated to be 5 to 7 m. Lowering of groundwater increases the load on the soil below the original groundwater level with a possible risk of settlement and adverse impact on neighbouring structures. In order to investigate the risk of adverse impact settlements were estimated using the procedures given in Preene et all (2000) as shown in Table 3.

The settlement estimate in Table 3 is critically dependent on the assumed value of the soil stiffness in one-dimensional compression which was taken as 200 MPa. The site investigation data gave SPT N values which increased from 25 at a shallow level to over 50 below -3 mDMD implying very dense soils and Preene et al (2000) suggest that the stiffness of an over consolidated sand might be approximately 200 MPa. In practice it was considered that these settlement estimates were conservative. Also the more significant airport infrastructure was founded on piles and would not be expected to be susceptible to surface settlements.

Settlement of granular soils would be expected to occur virtually instantaneously with drawdown. The computed settlements were not trivial and in order to investigate the situation, monitoring of surface monuments was carried out in close proximity to the wells where maximum drawdown would develop during the initial drawdown period. The accuracy of the monitoring was +/- 2 mm and for a drawdown of 10 m no discernable settlements was recorded. This implies that the stiffness in one dimensional compression for the substrata must be greater than about 1,000 MPa.

Table 3. Estimate of settlement

$\rho = D\,\Delta\sigma' / E'_o$ where	
ρ	Settlement of soil layer of thickness D
D	Thickness of soil layer, 0 to -36 mDMD, 36 m
$\Delta\sigma'$	Increase in vertical stress = δ_w s
δ_w	Specific weight of water, 9.81 kN/m^3
s	Drawdown, 0 to -7 mDMD, 7 m
E'_o	Stiffness in one-dimensional compression, taken as 200 MPa
	Settlement of soils above the lowered water table 1.2 mm
	Settlement of soils below the lowered water table 10 mm
	Total settlement estimate for 7 m drawdown 11.2 mm

7 DEWATERING MONITORING RESULTS

The dewatering was carried out using an array of wells installed around each phase of the excavation as shown in Figure 3 and Photo 1. Wells were generally installed from original ground level to toe into the conglomerate at approximately -36 mDMD. In order to maximize the rate of drawdown wells were initially installed at 50 m centers so that pumping could commence quickly around each phase perimeter. Infill wells were then installed to reduce the spacing to approximately 25 m.

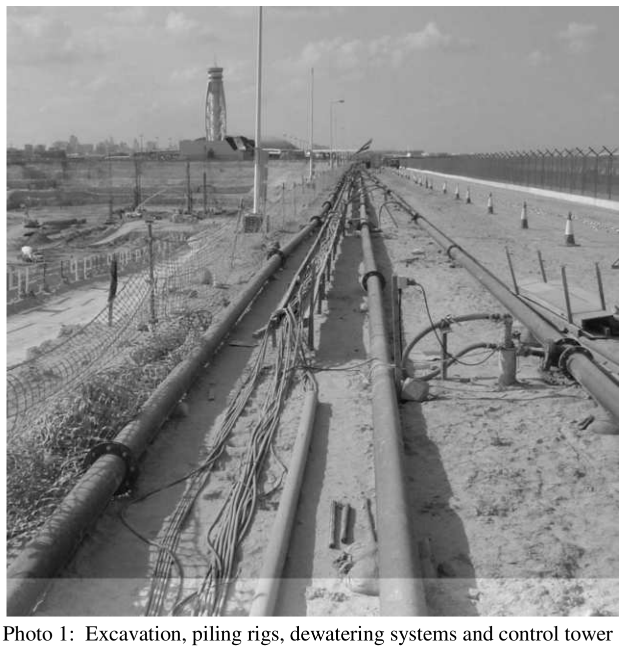

Photo 1: Excavation, piling rigs, dewatering systems and control tower

Discharge was to three 200 mm diameter ring mains which fed to discharge points at opposite sides of the excavation. Initial flows from the wells were 6 to 7 l/s but this reduced significantly as the drawdown was established. The pumps were powered by duty and standby generators from 4 control cabins. The control cabins housed the automatic switch gear which changed over to standby power if there was an interruption of the duty supply. The control system also included an alarm which warned operation and maintenance staff by GSM text message of a power failure or pump stoppage.

Monitoring of groundwater levels was carried out in an array of standpipe piezometers (Figure 3) using a combination of data logging with vibrating wire transducers (accessible piezometers) and manual daily dips. The flow rate was monitored using v-notch weir tanks located at the discharge points. Figure 5 gives groundwater level results for sample piezometer and the total flow records for the works. Note the phased start-up of the works in 2002/03. The slow recovery at P2 in 2007 was due to the dewatering required for the adjacent Concourse 3 construction which commenced in February 2005. This is also the cause of the further reduction in total flow below 100 l/s in the same year.

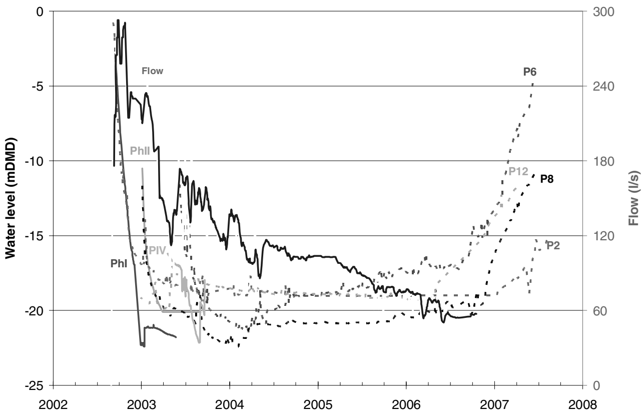

Figure 5. Flow & piezometer records (locations shown on Figure 3)

Samples of groundwater were taken from the discharge tanks and sent for chemical analysis and the results are summarized in Table 4. It can be seen that the groundwater was highly saline with a sodium chloride concentration four times greater than typical seawater. In addition the temperature of the groundwater was approximately 30^{o}C. Although stainless steel pumps (grade 904L) were used, cathodic protection measures proved to be essential to achieve a reasonable pump life. The density of water was sufficiently high that it needed to be taken into account for pump size selection and also the groundwater level monitoring transducers, which record pressure, needed to be specially calibrated.

Table 4. Chemical analysis of groundwater samples

Parameter	Units	Min	Average	Max
pH		6.2	6.9	7.3
Electrical conductivity	ms/cm	187	277	384
Total dissolved solids	mg/l	98,580	169,090	207,220
Sodium	mg/l	39,220	55,751	69,640
Chloride	mg/l	53,175	94,380	120,530
Suspended solids	mg/l	1.0	5.0	16.0
Density	g/ml	1.07	1.11	1.13

The development of the drawdown away from the excavation is shown in Figure 6. The more remote data for 134 weeks was obtained from a piezometer array installed for another project near by. After this the external groundwater levels were affected by dewatering on an adjacent site.

8 EXCAVATION AND TRENCHING

Large scale conventional excavation plant was used. Excavation was carried out in dry conditions down to approximately -19 mDMD where the rate of drawdown was found to have slowed appreciably. The ground conditions at this level comprised weak sandstone which proved to be stable even under the influence of the residual groundwater ingress present. Trenching was carried out to control groundwater below this level. In order to achieve a groundwater level to 1 m below excavation formation level a trench system was installed in the base of the excavation as shown in Figure 3. This comprised a 0.5 m wide by 1.5 m deep trench lined with a geotextile and backfilled with 20 mm stone. A 150 mm land drain was laid in the base of the trench which fed to four sump wells that were installed in the diaphragm wall in box outs.

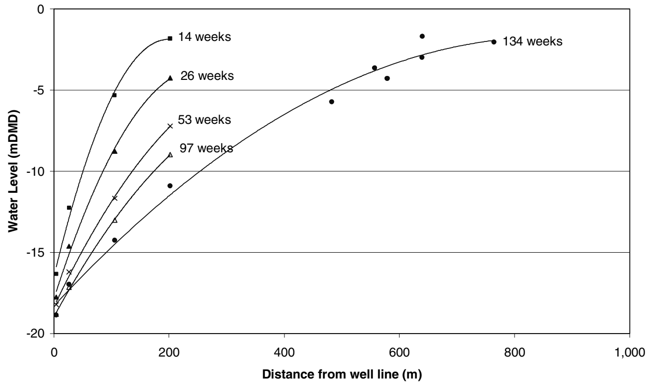

Figure 6. Development of external drawdown

9 CONCLUSION

Dubai has been undergoing intensive development and expansion over many years. Many of the developments and associated infrastructure require deep excavations with a need for extensive dewatering over prolonged periods. This paper is a case study of one such large scale long term temporary works dewatering scheme. The scale of the works and its proximity to existing airport infrastructure has meant that the dewatering operations have been unusually well monitored and documented. Access for the excavation became available progressively providing both a constraint on the works but also an opportunity to use data and monitoring from the early phases to refine the programme and dewatering scheme arrangements for later phases. This has been carried out using a 3-D finite element time dependent model of the works. This case study explains the design development process and the benefits achieved. There was some concern about the possible adverse impact of drawdown below existing adjacent infrastructure. The model provided a design basis for estimating drawdowns and the assessment concluded that risks to infrastructure were acceptably low. This was confirmed by subsequent monitoring.

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REFERENCES

Fookes, P.G., French, W.J., & Rice, S.M.M. 1985. *The influence of ground and groundwater geochemistry on construction in the Middle East*. Quarterly Journal of Eng. Geol., London, Vol.18 pp.101-128.

Preene, M., Roberts, T.O.L., Powrie, W., and Dyer, M.R. *Groundwater control – design and practice*. 2000. CIRIA C515, London.

Rumbaugh, J., and Rumbaugh, D. 1999. *Guide to using groundwater vistas*. Environmental Simulations Inc., Reinhold, USA.