Zur Wasserhaltung für NÖT-Vortriebe in den pleistozänen Dünensanden von Tel Aviv

Dewatering measures for NATM tunnels in Tel Aviv

Marcus Scholz[1], Levy Kroitoru[2]

[1] Dr. Marcus Scholz, Müller + Hereth Ingenieurbüro für Tunnel- und Felsbau GmbH, m.scholz@mueller-hereth.com
[2] Dr. Levy Kroitoru, Ecolog Engineering Ltd, kroitorul.@ecolog.co.il

Zusammenfassung

Im Zuge der Stadtentwicklung von Tel Aviv wird derzeit die Red Line als erste Stadtbahnlinie geplant. Etwa 12 km der 23 km langen Strecke werden unterirdisch verlaufen, wobei 8,5 km Tunnelstrecke mittels Erddruckschilden aufzufahren sind und 3,5 km als NÖT-Vortriebe erstellt werden. Die NÖT-Abschnitte umfassen eingleisige Streckentunnel und Weichenkavernen. Die Überlagerung über Firste variiert je nach Gradiente und Geometrie der Tunnel zwischen 4 m und 20 m. Die Tunnel liegen in den pleistozänen Dünensanden der Kurkar-Formation. Diese sind bereichsweise lose und bereichsweise. unterschiedlich stark verkittet. Der Grundwasserspiegel liegt etwa 10 m unter GOK, so dass die NÖT-Tunnel teilweise bzw. vollständig im Grundwasser liegen. Sand und Wasser sind für NÖT-Vortriebe eine große Herausforderung. Daher wurden verschiedene Methoden der Wasserhaltung (Druckluft, Vereisung, Injektion und Grundwasserabsenkung) erörtert. Letztlich wurde die Grundwasserabsenkung für die Genehmigung und für die Ausschreibung gewählt. Die Dimensionierung der Wasserhaltung erfolgte auf Basis empirischer Formeln. Da jedoch bekanntermaßen Unsicherheiten in diesen Formeln enthalten sind und bisher keine Erfahrungen im Baugrund von Tel Aviv vorliegen, wurde der Rechengang für einzelne Vergleichsabschnitte an 3D-Modellierungen mittels MODFLOW® kalibriert.

Schlüsselworte: NÖT, Kurkar-Formation, Sand, Wasserhaltung

Abstract

NTA (Metropolitan Mass Transit System Ltd) is in charge of the design and implementation of a Mass Transit System for the central Israel Dan Region. The first line to be executed is the "Red-Line", running from the cities of Bat Yam and Tel Aviv in the west to the city of Petah Tikva in the East. Some 12 km of the 23 km alignment will be tunnels. About 8,5 m of the tunnel alignemtn will be constructed by EPB-TBMs whereas ~ 3,5 km of the tunnels will be constructed by means of NATM. The NATM tunnels are located in Pleistocene sands and carbonate cemented sandstones of Kurkar-Formation partly under the groundwater table as well as in Holocene clay deposits of the Yarkon flood plain in. Extensive dewatering measures are required. The dewatering alternatives that were considered include: compressed air, sealing by means of injection (synthetic resin), sealing by means of freezing and dewatering by means of wells. The last option was selected in order to design the dewatering measure for the design build (DB) tender of the project. As only little hydological data was available, the assessement of the dewatering measures for the tender documents was based on well known simple empirical formulas. Nevertheless based on the client request a calibration of the empirical approach by means of 3D-modelling was performed. The 3D-modelling was performed with MODFLOW®. The results of modelling confirmed the results of the empirical aproach .

Keywords: NATM, Kurkar-Formation, Sand, Dewatering

1 Project Outline

NTA (Metropolitan Mass Transit System) is in charge of the design and implementation of a Mass Transit System for the central Israel Dan Region. The first line to be executed is the "Red-Line", running from the cities of Bat Yam and Tel Aviv in the west to the city of Petah Tikva in the East. The underground section consists of two single track tunnels, ten Cut&Cover Stations and three Cut&Cover-structures acting as the “Depot” portal, and “Shenkar” portal in the north-east and as “Elifelet” portal in the south west of the Red Line alignment. Most of the running tunnels of the Red Line will be constructed by means of TBM, while in the north-eastern section of the alignment approx. 1895 m of single track tunnels and three bifurcation chambers (each of 90-115 m in length) will be constructed by means of NATM (New Austrian Tunneling Method). The NATM section is located within the municipalities of Bnei Brak and Petah Tikva.

Müller+Hereth GmbH was hired by NTA to prepare the design for the NATM section to be included in the Design Build (DB) bid package for this job. Ecolog Engineering Ltd, on behalf of Müller+Hereth GmbH, acts as the Israeli point of contact and provides the needed local support.

In the south section of the NATM project the two single track tunnels of Axis 1R and of Axis 2L are directed from the “Shenkar” portal to the west ending in two bifurcation chambers. These bifurcation chambers (chamber 1/5 and chamber 2/6) are merging the NATM tunnels of Axis 1R and of Axis 2L with the TBM tunnels of Axis 5 and Axis 6, which are directed to the Depot. Another single track tunnel (Axis 8) is connecting the tunnel (Axis 2L) from portal "Shenkar" through another chamber (chamber 2/8) toward the "Depot" portal in the northern portion of the Red Line alignment. The northern section of the NATM project consists of two single track tunnels of Axis 5 and Axis 6 between the "Depot" portal and the "Em Hamoshavot" station.

The overburden varies from 4-20 m above crown depending on the alignment and on the geometry of the tunnels.

The running tunnels have a circular cross section with an internal diameter of 6.5 m. The external diameter is 7.8 m. The primary lining consists of 25 cm thick reinforced shotcrete. The permanent lining consists of 40 cm thick reinforced water tight concrete.

The bifurcation chambers have a maximum span of 19 m. The primary lining of the bifurcation chambers consists of 35 cm thick reinforced shotcrete. The permanent lining consists of 70 cm thick reinforced water tight concrete.

All single track tunnels are connected in a distance of maximum 250 m by cross passages for safety reasons. These cross passages are to be constructed in a mined method.

All tunneling works include in principal:

- Dewatering, water treatment and discharge of water,
- Excavation, mucking and dumping,
- Primary Support,
- Inner lining by mean of water tight concrete,
- Preparatory works for systems installations,
- Additional works as jet grouting, compensation grouting, and deformation mitigation measures.

2 Geological Setting

2.1 General

The area under investigation is located in the middle of the Israeli coastal plain, approximately 6.5 to 7.0 km east from the coast line, at the north eastern edge of the city of Bnei Brak and north western edge of the city of Petah Tikva, some 3 km east of Tel Aviv.

The uppermost unit of the coastal plain of Israel is built mainly of carbonate cemented sandstones (Kurkar). The intensity of cementation varies. The Kurkar rocks are a lithification product of windblown sands that were piled up as dunes parallel to the shore during the Pleistocene-Holocene. The sharp drop in the global sea level and regression during the last glacial period exposed the continental shelf to sub-aerial processes and erosion, which lead to dune movement and the creation of several Kurkar ridges. The subsequent transgression of the sea flooded the Kurkar ridges and filled the depressions between them with delta derived sands (from the Nile) and clayey silt layers.

2.2 Hydrogeology

The Kurkar group forms the Coastal Aquifer of Israel. In the studied region, the aquifer is recharged by rain precipitation during the winter months and leakage from water lines, from sewer lines and from excess irrigation.

The aquifer is composed of Pleistocene sand and calcite-cemented sandstone, interbedded with marine and continental silt and shale lenses reaching a total thickness of up to 200 m. The sand and the silt/shale lenses represent different depositional environments according to the sea water transgressions and regressions during the Pleistocene. Up to four sub-aquifers separated by the shale and silty shale lenses were formed in the sandstone layers. The lateral distribution of the shale and silty shale lenses of these sub-aquifers is commonly restricted to 5 km east of the Mediterranean shoreline (ISSAR, 1980; NATIV and WEISBORD, 1994).

The upper part (i.e. 150-200 m) of the sub-surface consists of Quaternary deposits bounded within the Kurkar Group. The Kurkar Group is shaped as a triangle, which thins from about 200 m near the shore to 0 m on the east at the foothills, and overlays clays of the Saqiye Group. The Kurkar Group includes mainly calcareous-sandstones, inter-layered with thin clay layers.

Under the investigated area the thickness of the Kurkar Group is 100-110 m and its base is at an elevation of approx. -70 m.a.s.l. According to the regional geological cross-sections the calcareous-sandstones unit is probably continuous, from the surface down to the top of the Saqiye Group. The uppermost part of the sub-surface is covered by relatively thin layers of loam and alluvium. Further to the north, the upper alluvium unit thickens, as it is approaching the flood-plain of the Yarkon River.

The general historical groundwater flow direction in the investigated area is westward, towards the sea, with groundwater levels decreasing from east to west. A gradient of about 1 m/km is typical to the western part of the aquifer, where the aquifer is thick and includes mainly porous and permeable calcareous sandstone.

During the last decades, a hydrological sink was developed about one kilometer to the S-W of the study area, under the city of Giva'taim and Tel Aviv with groundwater levels of up to 10 m below the sea-level. The current groundwater level in the project area is at +2.0 to +5.0 m.a.s.l.

Perched groundwater horizons can be encountered in the investigated area mainly within the clay and alluvium deposits of the Yarkon river flood plain. Perched groundwater horizons may occur while penetrating local lenses of sand within the overall clayey section.

The hydraulic properties of the coastal aquifer are not well known in detail. Based on local field investigations and studies including work done by Ecolog Engineering Ltd (ECOLOG) it is proposed to use a hydraulic conductivity value of 20 m/d ($2*10^{-4}$ m/s) for the Kurkar in horizontal direction. The hydraulic conductivity in vertical direction is ten times smaller. The hydraulic permeability of the clay and loam is ~1 m/d ($1.16*10^{-5}$ m/s).

2.3 Geotechnical Setting at Axes 1R, 2L and 8

The building ground for this section comprises mainly sandy soil types of the Kurkar formation (K1, K2 and locally K3). On top of the Kurkar formation there are sandy soil types (SP, SP-SM, SP-SC locally SC – soil types according to USC Unified Soil Classification). The subtypes of Kurkar Formation are characterized as follows:

Tab. 1: Kurzbeschreibung der verschiedenen Typen von Kurkar.
Tab. 1: Short description of types of Kurkar

Type	Description
K1	Fine calcareous sand with up to 10% of clay and 5-25% of Kurkar-gravel, generally uncemented.
K2	Alternation of loose sand and slightly to moderately cemented fragile Kurkar plates, cross bedded, with cemented veins (honeycomb like structure).
K3	Alternation of solid, hard, laminar Kurkar sheet (2-15 cm thickness) interbedded with loose uncemented sand.

Mixture of various amounts of K1, K2 and K3 are common. The varying grade of cementation of Kurkar results in mixed face conditions with soil and hard rock within the tunnel face. But cemented hard rock will be the minor part anyway.

Because of the porous cementation (slight to moderate grade) the Kurkar types K2/K3 have a metastable soil structure of extremely high compressibility. The cementation might break down at point load pressures without confining pressures.

The results of Standard-Penetration-Tests (SPT), which were performed at the tunnel elevation, display in general a medium to high grade of compaction of the Kurkar types.

The content of Quartz ranges between 25-60%. The content of calcareous shell fragments ranges between 20-40%.

According to the variation Quartz content and cementation the Cechar Abrasiveness Index (CAI) ranges between 0.64 and 2.86. This characterizes the Kurkar rock material as slightly to very abrasive with a mean range of medium abrasive to abrasive.

The UCS (uniaxial compressive strength) of slightly to moderately cemented strata of Kurkar varies between 0.5 and 15 MPa.

The sandy soil and the Kurkar comprise the Coastal Aquifer. They are overlain by soil of type SC with varying thickness and with low permeability. At this location the aquifer is under phreatic conditions. The current groundwater levels increase from approx. +2.5 m.a.s.l. at Chamber 1/5 to approx. +5.0 m.a.s.l. at the eastern edge of the area.

The maximum required drawdown for Axis 1R is 6.39 m. The minimum required drawdown for Axis 1R is 2.39 m.

The maximum required drawdown for Axis 2L is 10.51 m. The minimum required drawdown for Axis 2L is 4.58 m.

The maximum required drawdown for Axis 8 is 7.69 m. The minimum required drawdown for Axis 8 is 4.96 m.

2.4 Geotechnical Setting at Axes 5 and 6

This section of Axis 5 and Axis 6 is located in the Yarkon River flood plain north of Em Hamoshavot station. The building ground for Axes 5/6 comprises mainly clay with high to low plasticity (CL and CH according to USC). Some lenses of clayey sand and non-cohesive soil (SC, OH, SW, SW-SM, SP-SM, SM – soil types according to USC Unified Soil Classification) with varying thickness are interbedded locally. The consistency of the fine grained soils varies from liquid to firm whereby most of the samples have stiff consistency.

The base of the clay deposits is located at elevations varying between -5 m.a.s.l. and -10 m.a.s.l. in the western half part of the section whereas it is located at a steady elevation of about -10 m in the eastern half part of the section.

The clay deposits act as an aquitard and are fully saturated below the groundwater table. Underneath the clay deposits, sandy soil types are classified as SP, SP-SC (according to USC) and form the aquifer. Accordingly the aquifer is confined. The invert of the running tunnels will be located in close distance to the boundary between clay and the underlying sandy soil.

The current groundwater levels increase from approx. +3.0 m.a.s.l. at Em-Hamoshavot Station to approx. +5.0 m.a.s.l. at the Depot Portal. Based on the stated groundwater levels the aquifer can be considered to be under confined conditions. However, dewatering of ~0.5 m below the tunnel invert will lead to drawdown and the development of local phreatic conditions with projected groundwater levels down to at least approx. –10.0 m.a.s.l.

The maximum required drawdown for Axes 5/6 is 15.26 m. The minimum required drawdown for Axes 5/6 is 9.76 m.

3 Assessment of soil behavior for NATM

The main features of tunneling in Kurkar-Formation are as follows:

- Slightly wet sand is commonly of high grade of compaction and therefore manageable by NATM.
- Because of the labile stability of the sandy soil types of Kurkar, generally a short length of advance is required.
- Pre-support is required in the loose soil types.
- In the slightly cemented Kurkar-types (K2, K3) it will be possible to handle the NATM drives without pre-support but with short length of advance.
- Mixed-Face-Condition with loose sand and sandstone with varying distribution of hard rock and soil in the tunnel face will be encountered all along the alignment.
- Face support in the tunnel face are necessary in terms of reducing the range of deformation (they hinder relaxing of the face).
- Water management from tunnel and tunnel face (filter drains, vacuum drains, drainage mats).

- Strata of wet, saturated sand may appear in short sections within the invert of the tunnels. Even if they appear occasionally and with thin thickness, they can induce deformation. Experienced handling in tunneling is required.

Based on laboratory tests it can be stated that most of the tested samples of fine grained soil have stiff consistency. Occasionally some strata of very stiff consistency is present. However some of the strata is of soft consistency and can influence the soil behavior in a negative way. One stratum of liquid plasticity was recorded. This displays the influence of groundwater at boundaries between sand and clay. Strata of liquid consistency are unfavorable and not easy to handle in general. The main features of tunneling in fine grained soil are as follows:

- The stiff material is proper for tunneling in soil, when treated in the right way with short length of advance and immediate ring closure.
- Pre-support is necessary in terms of reducing the range of deformation.
- Anchors in the tunnel face are necessary in terms of reducing the range of deformation (they hinder relaxing of the face).
- The strata of soft, very soft and liquid consistency is unfavorable. Even if it appears occasionally and with thin thickness, it can induce high deformation. Experienced handling in tunneling is needed to deal with this danger.
- Water management from tunnel and tunnel face (filter drains, vacuum drains, drainage mats).

4 Dewatering

4.1 General

Due to the hydrogeological situation dewatering measures are required for all NATM sections. For Axis 1R, Axis 2L, Axis 8, Chamber 1/5, Chamber 2/6 and Chamber 2/8 dewatering is required in order to lower the water table within the sandy soil underneath the invert of the tunnel. For Axis 5 and Axis 6 dewatering will be needed in order to reduce the water pressure within the sandy soil underneath the clay deposits in order to avoid an uplift of the invert of the tunnel during excavation.

Dewatering measures can be done by:

1. Compressed Air – not feasible because of the high permeability of the sand and because of the varying elevation of the gradient of the tunnels and accordingly the tunnel crown is not everywhere covered by groundwater.
2. Sealing by means of injection (synthetic resin) – technically and financially not feasible because of the small pore size of the sand.
3. Sealing by means of freezing – technically and financially not feasible because of the temperature of the groundwater (approx. 16.5°C).
4. Dewatering by means of wells – technically feasible.

Dewatering shall be performed prior to tunnel excavation and shall be maintained at a level of 0.5 m under invert level until the excavation face has left the radius of influence of a single well. The shotcrete lining will be designed strong enough to bear the loads resulting from earth pressure and maximum ground water level. Therefore it will be possible to shut down the dewatering wells after the tunnel face has reached a certain distance from the single well. Dewatering shall be performed by installation of vertical filter wells from the surface. If required, additional dewatering measures shall be installed in the tunnels from the excavation face to drain remaining water not captured by the filter wells from the surface, such as water pockets within self-contained sand lenses.

For the particular ground conditions of the NATM tunnels, in general the following dewatering measures will be applied along tunnel alignment in accordance with the encountered ground conditions in order that:

- Excavation of the tunnel is possible in general (stabilize excavation face),
- No erosion effects can occur at the excavation face,
- The ground arch around the tunnel can withstand seepage forces.

In general, dewatering can be achieved by:

- Filter-wells from the surface,
- Horizontal drainage pipes including vacuum drains installed from existing excavations,
- Drainage pipe umbrella including vacuum drains installed from the tunnel excavation face.

Groundwater pumped during the dewatering activities will be treated according to its quality. Following are the treatment options:

- Discharge to the local sewer system: if the water is polluted and does not exceed the threshold values for discharge to the local sewer system.
- On/Off site treatment: if the water is polluted and exceeds the threshold values for discharge to the local sewer system.
- Recharge to the aquifer: if the water is not polluted and meets the regulator discharge standards.
- Discharge to local streams and drainage channels: if the water is not polluted and meets the regulator discharge standards.
- Irrigation: if the water is not polluted and meets the regulator discharge standards.

At this time we are not in a position to propose detailed treatment options for the groundwater associated with the dewatering activities. The reason for this is that at this time there is no sufficient data as to the water quality along the proposed Red Line NATM alignment.

4.2 Empirical Assessment of Dewatering Measures by Means of Wells

The empirical calculations assume a steady state and are based on the multi-well formulas developed by FORCHHEIMER (refer to HERETH and ARNDTS, 1994). The calculations for Axis 1R, Axis 2L and Axis 8 are performed for phreatic water table conditions. The calculations for Axis 5/6 to Depot are performed for confined water table conditions.

The calculations are base also on the following assumptions:

- The ground water table before pumping is slightly inclined.
- The aquifer is heterogeneous and anisotropic.
- There are no aquifer boundaries within the radius of the reach of the dewatering measures.

For the design of the dewatering measures the tunnel alignment is subdivided into sections of 40 m of length and a width of 8 to 18 m in general. A well pattern of about 8 wells per 40 m section was selected, since this pattern results in relatively low feed rates and relative short well lengths compared to a wider pattern of 4 or 6 wells per 40 m section. Besides this, the relative dense pattern of 8 wells per 40 m section helps to cover recently unknown but not obviated heterogeneities in the aquifer. This has to be taken into consideration as the permeability of the building ground as well as the groundwater levels are still not defined.

The wells are generally located along the tunnel alignments. The pattern and spacing of wells had to be adapted to the infrastructure situation (traffic lanes and utilities). Therefore the well pattern cannot be optimized in terms of hydrology. The general concept of the dewatering measure for the NATM-drives intends to minimize dewatering according to the needs of the NATM-drives. Dewatering will only be active in these sections of the alignment, where an active NATM-drive will be performed. This means that for a minimum, one 40 m section per tunnel axis will be treated with dewatering, whereby for a maximum two 40 m sections per tunnel axis will be treated with dewatering at the same time.

The drawdown is calculated at critical points that are located in critical positions (e.g. largest distance from any well) whereby the drawdown must exceed the required drawdown to a level of 2.2 m below the design elevation.

The calculations assume a well diameter of 24 inch.

4.3 Steps of the empirical Assessment

(1) Input and definition of empirical parameter eta η, which represents the theoretical radius of a single "equivalent" well.

(2) Calculate the radius of influence of pumping using SICHART and KYRIELEIS empirical equation.

(3) A total penetration of the aquifer is assumed for calculating the required feed rate Q in [m³/s] for the steady state for a single section using the standard equations according to FORCHHEIMER.

For phreatic conditions:

$$Q = \frac{\pi * k * (H^2 - h^2)}{2{,}3 * (\lg R_0 - \lg A_{RE})}$$

For confined conditions:

$$Q = \frac{\pi * k * (H^2 - h^2)}{2{,}3 * (\lg R_0 - \lg A_{RE})}$$

where:

s: required drawdown in [m],
k: hydraulic conductivity in [m/s],
R_0: radius of influence in [m],
A_{RE}: radius of equivalent well in [m],
B: thickness of the aquifer from top of the aquifer to the bottom, which is considered to be at the depth of the bottom of the pumping wells (in accordance to the with the Dupuit assumptions) [m].

(4) Adding a plus of 10% for safety reasons and handling time.

(5) Adding a plus of 25% in order to take into consideration the partial penetration which leads to an additional vertical groundwater flow from the bottom of the well that is not considered in the Dupuit assumption.

(6) Calculation of the capacity of a single well (q) in [m³/s] with diameter of 24" assuming a local drawdown in the single well and definition of the number of wells needed to reach the total feed rate Q in [m³/s].

(7) Arrangement of well pattern and calculation of equivalent distance between all wells and a critical point of the section $\frac{1}{n} * \sum \lg x$, whereby x is the distance between a well and the critical point).

(8) Assessment of amount of water for required drawdown of (s) at the critical point B (greatest distance from wells) using superposition:

For phreatic conditions:

$$Q = \frac{\pi * k * (H^2 - h^2)}{2{,}3 * (\lg R_0 - \frac{1}{n} \sum \lg x)}$$

For confined conditions:

$$Q = \frac{2\pi * k * B * s}{2{,}3 * (\lg R_0 - \frac{1}{n} \sum \lg x)}$$

where:

n: number of wells in well pattern
x: distance between wells and critical point in well pattern.

(9) Assessment of required capacity (q) of a single well by division of Q by the number of wells.

(10) Assessment of local drawdown in a single well S_{EB} with average distance b between the single wells:

For phreatic conditions:

$$S_{EB} = h - \sqrt{h^2 - \frac{1{,}5q * 2{,}3 * (\lg b - \lg r)}{k * \pi}}$$

For confined conditions:

$$S_{EB} = \frac{1{,}5q * 2{,}3 * (\lg b - \lg r)}{k * \pi * 2B}$$

where:

S_{EB}: local drawdown in a single well in [m],

q: feed rate capacity of a single well [m³/s],

r: radius of well in [m],

b: half of average distance between the wells in [m],

(11) Assessment of local drawdown in a single well S_{EB} with average distance b between the single wells.

(12) Assessment of real capacity of a single well:

$$q = 2 * \pi * r * h' * \frac{\sqrt{k}}{15}$$

where: h': wet length of filter of well in [m],

(13) For calculation under confined conditions: verification of drawdown:

$$s = \frac{Q}{2 * \pi * k * B} * (lnR - \frac{1}{n}\sum \ln x)$$

(14) Adapting length of well in order to reach the required drawdown and/or the required quantity of feed rate. In some cases for Axis 5 and Axis 6 also the number of wells was varied in order to minimize the total quantity of water.

(15) Adding a plus of 25% in order to take into consideration the partial penetration of the wells.

4.4 Calibration to the Results of 3D-modelling

3D-modelling with MODFLOW ® was performed for four 40 m-sections in phreatic condition taking also into account the difference of 1/10 between horizontal and vertical conductivity, which cannot be done by the empirical approach. For the confined condition two 40 m-sections were chosen for calibration.

The results of 3D-modelling differed from the results of the empirical approach and required the addition of 0.2-7.0% as far as the length of wells for the phreatic conditions and 11-24% for the confined condition so the results of the empirical approach will match the 3D modeling results.

Accordingly the following modifications were done:

- For phreatic conditions: adding 10% to the length of wells.
- For confined conditions: adding 25% to the length of wells.

4.5 Results

Dewatering of Axis 1R, Axis 2L, and Axis 8 including all three chambers will require the pumping of about 19,000,000.00 m³ within the construction period of 3.5 years. Dewatering of Axis 5 and Axis 6 will require pumping of about 14,000,000.00 m³ within the construction period of 2.0-2.5 years.

The deformation resulting from dewatering is estimated to be 0.5 to 2.0 cm.

5 Final Remark

The empirical approach based according to FORCHHEIMER was used in order to obtain the number and length of wells, which are needed for dewatering of the NATM sections. One of the goals of the proposed approach was to reduce the lengths of the wells to a minimum. Therefore, initially the well lengths were set to a minimum. Thus, it was possible to enlarge the length of the wells in order to calibrate the analytical calculation to the results of the 3D-modeling.

The differences in the results of the empirical approach and the 3D-modelling are not insignificant. Accordingly it is recommended to calibrate any empirical approach either by field tests or by 3D-modelling.

Literature

HERTH, W. & ARNDTS, E. (1994): Theorie und Praxis der Grundwasserabsenkung (Dewatering in theory and practical experience). 3. Auflage, Ernst & Sohn, Berlin.

ISSAR, A. 1980 Stratigraphy and Paleoclimates of the Pleistocene in Central and Northern Israel. Palaeogeogr. Palaeoclim. Palaeoecol. V. 29, pp 261-280.

NATIV R. AND WEISBROD N., 1994: Hydraulic connections among subaquifers of the coastal plain aquifer, Israel. Ground Water, Volume 32, Issue 6, pages 997–1007.

SNEH, A., BARTOV, Y., AND ROSENSAFT, M. 1998. Geological Map of Israel, 1:200,000, sheet 2. Ministry of National Infrastructures, Geological Survey of Israel, Jerusalem, 1998