International Conference on Case Histories in Geotechnical Engineering

(2013) - Seventh International Conference on Case Histories in Geotechnical Engineering

02 May 2013, 4:00 pm - 6:00 pm

# Case Studies of Dewatering and Foundation Design: Retail Warehouses in Taiwan

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Commemorate the Legacy of Ralph B. Peck
April 29 - May 4, 2013 CHICAGO
Seventh International Conference on
Case Histories in Geotechnical Engineering
and Symposium in Honor of Clyde Baker

# CASE STUDIES OF DEWATERING AND FOUNDATION DESIGN: RETAIL WAREHOUSES IN TAIWAN

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ABSTRACT

The case studies in this paper describe three retail warehouse sites in Taiwan that have high groundwater tables in common, but that have drastically different soil conditions. Two of the sites are in dense, permeable gravel and cobble and the third site is in interbedded alluvial sand and clay. At the first site, shallow footings and slab-on-grade floors were placed on top of a permanent passive drainage system that required accurate seepage volume estimates in the permeable gravel and cobble. At the second site, the hydraulic conductivity of the gravel and cobble is high and is sensitive to fluctuation of the regional groundwater table. A hybrid passive drainage and structural slab system minimizes pumping during the seasonal high groundwater table. At the third site, deep slurry walls constructed around the building cut off groundwater seepage, and permanent pumping wells within the building footprint lower the groundwater table. This system also eliminated the risk of soil liquefaction and allowed shallow footings and slab-on-grade floors to be used. This paper discusses the hydrogeological analysis of the three sites and the geotechnical design considerations for the dewatering and foundation systems, as well as soil liquefaction mitigation.

## INTRODUCTION

A US-based retail warehouse chain has built and operated ten retail warehouses to date in Taiwan as part of its ongoing expansion into the East Asia market. Many of these warehouses occupy an entire city block and require two to three levels of underground parking because of building space limitations in the densely populated urban environment. High groundwater tables are common for many of these building sites.

Local buildings with similar hydrogeological constraints typically require structural systems that include tie-down piles and watertight bathtub structures. This paper describes three of the warehouse building sites; each has unique dewatering and foundation solutions according to the soil and groundwater conditions at each site. These solutions include permanent passive under-slab drainage systems over gravel and cobbles, and a system of active pumping wells combined with deep slurry walls that cut off groundwater seepage. The three retail warehouses are believed to be the first to use these solutions in commercial building construction in Taiwan.

Geotechnical considerations of the shoring and foundation system for each site will be discussed conceptually. However, these case studies focus on the groundwater modeling and analysis that led to the substructure selection for each site. Each case study describes the chosen dewatering and drainage system and how that system influenced foundation selection and design.

## PROJECT INFORMATION

The three selected sites are in the cities of Hsinchu (Site A), Taichung (Site B), and Tainan (Site C), located from north to south along the west coastal plain of Taiwan. Figure 1 shows their locations on the island.

Taiwan is a seismically active region and has governing seismic design criteria similar to those used in the International Building Code (IBC). The peak ground accelerations for these project sites are about 0.38 g and 0.45 g for the Design Earthquake and Maximum Considered Earthquake (MCE), respectively. The Design Earthquake has a return period of 475 years, and the MCE has a return period of 2,475 years. Soil liquefaction is not a concern for Sites A and B because the buildings are founded on very dense gravel and cobble. However, at Site C, the medium dense silty sand


*Fig. 1. Location of Project Sites.*

below the groundwater table in Layer III is susceptible to soil liquefaction during a Design Earthquake. The dense sand in the deeper Layers V and VII are not. The silty clays in Layers II, IV, and VI are not susceptible to soil liquefaction due to its plasticity and cohesion.

Structural design data for these warehouses typically include a column load of about 2,000 to 3,000 kN (450 to 675 kips) and a floor load of about 14.5 to 19 kPa (300 to 400 lb/ft). Spread footings and slab-on-grade floors are understandably the preferred foundation system given the size of the building footprint, if high groundwater and soil liquefaction (at Site C) can be mitigated.

Subsurface explorations were completed at each site, followed by laboratory studies including grain size analysis. Hydrogeological studies consisted of installing wells, measuring groundwater levels, and aquifer testing. Slug and/or pumping tests were conducted at the sites to determine the hydraulic conductivity of the primary water-bearing zones.

A groundwater flow model was developed for each of the sites to determine the drainage required to maintain the water table below the excavation during construction and below the basement slab permanently. The model was constructed using the computer program MODFLOW (MacDonald and Harbaugh 1988). Groundwater Vistas (Environmental Simulations 2006) was used for pre- and post-processing the model.

The model grid was centered on the building footprint with one axis of the model grid aligned with the direction of groundwater flow. Model boundaries coincided with surface water bodies or extended a sufficient distance from the building footprint to minimize the effect of constant head boundaries on drawdown rates. Drains were used to simulate wells and building drainage systems. Grid spacing ranged from 5 to 10 meters. Modeling was conducted under steady-state conditions. The sensitivity of model output was evaluated by varying the hydraulic conductivity of the aquifer and constant head boundary elevations.

The depths referred to in the case studies assume the finish floor elevation is approximately the same as the ground surface elevation. No major cuts or fills lowered or raised the ground surface elevation outside the building footprint. Subgrade elevations in the case studies are measured from the ground surface, i.e., the ground surface elevation datum is 0.

## CASE STUDY A: HSINCHU

The Hsinchu site was vacant, and plans called for a building footprint of about 117 by 76 meters. The site grade varies about 2 meters across the site. The site is encompassed by city streets to the west and south, three-story buildings immediately to the north, and a vacant parcel to the east. The completed building is two levels above grade and has three levels of underground parking. The B3 floor is at –10.7 meters.

### Subsurface Conditions

Subsurface explorations included 12 test borings finished as groundwater observation wells to a maximum depth of 20 meters (Fig. 2).

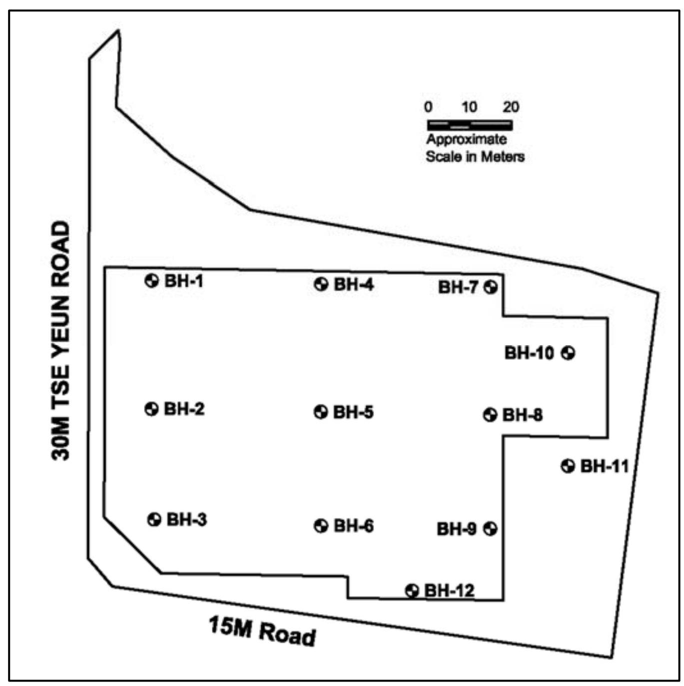


*Fig. 2. Exploration Map - Hsinchu*

Soils encountered from top to bottom in the explorations were:

Fill. The surface soils consisted mostly of soft, silty clay with variable amounts of sand, gravel, and organic material with an average thickness of about 1.0 meter (up to 2.4 meters at the southwest corner).

Clay. A layer of medium stiff to stiff, silty clay was encountered above the Gravel/Cobble unit to about –4.0 to –6.3 meters.

Gravel/Cobble. A very dense gravel and cobble deposit with sand and silt lay below the Clay unit and extended to about –10.6 to –14.2 meters.

Bedrock. Below the Gravel/Cobble unit, weakly cemented sandstone was encountered to the termination depths of the borings at –15 and –20 meters.

## Hydrogeological Characteristics

Groundwater was encountered from –2.0 to –4.5 meters. Given that the top of the Gravel/Cobble unit was at –4.0 to –6.3 meters, and the Clay unit did not appear to be saturated, this water-bearing Gravel/Cobble aquifer appeared to be confined between the low permeability Clay and Bedrock units.

Monitoring well slug tests were performed to estimate the hydraulic conductivity of the Gravel/Cobble unit. The hydraulic conductivity ranged from $1.7 \times 10^{-5}$ centimeters per second (cm/sec) to $1.1 \times 10^{-2}$ cm/sec, equal to 0.015 to 9.5 meters per day (m/day) with a geometric mean of $5.9 \times 10^{-4}$ cm/sec (0.5 m/day). Hydraulic conductivity values estimated from grain size analysis using the Hazen method were significantly lower, ranging from $1 \times 10^{-6}$ cm/sec ($9 \times 10^{-4}$ m/day) to $3.6 \times 10^{-5}$ cm/sec (0.03 m/day). The samples used for grain size analysis were biased for the silt matrix material surrounding the gravel and cobble-sized clasts, which likely underestimated the hydraulic conductivity of the water-bearing Gravel/Cobble unit. The hydraulic conductivity values based on the slug tests were considered to be better representative of soil conditions in the Gravel/Cobble unit.

## Groundwater Modeling

The goal of groundwater modeling at Site A was to estimate the extraction rates for temporary construction dewatering and permanent drainage systems. Under baseline conditions, the aquifer was assigned a hydraulic conductivity of 0.5 m/day ($6 \times 10^{-4}$ cm/sec) and the constant head boundaries were assigned a head of –2.85 meters. For purposes of sensitivity and uncertainty analysis, the water table was varied from –1.85 to –2.85 meters and the aquifer hydraulic conductivity from $1 \times 10^{-4}$ to $1 \times 10^{-2}$ cm/sec (0.09 to 9 m/day).

The lowest subgrade (B3) floor was designed at –10.7 meters. Given the likely thickness of the footing, the site needed to be excavated to at least –11.7 meters. To provide a dry work platform and avoid disturbing the foundation subgrade, the water table had to be lowered to at least –12.3 meters, which amounts to a drawdown of between 8.5 to 10 meters within the excavation footprint.

Based on the results of groundwater modeling, the amount of water that had to be extracted from the excavation to achieve dry conditions down to –12.3 meters was estimated to be 300 cubic meters per day ($m^3$/day) equal to 55 gallons per minute (gpm). Based on sensitivity analysis the likely range of extraction could be from 50 to 500 $m^3$/day (10 to 90 gpm). A design value of 400 $m^3$/day (70 gpm) was selected for construction dewatering.

The goal of the permanent drainage system was to maintain water levels below the base of the foundation slab at –10.7 meters. Based on groundwater modeling of the most likely groundwater conditions, the total volume of groundwater seepage was estimated to be 150 $m^3$/day (28 gpm) with a range of 40 to 400 $m^3$/day (7 to 70 gpm) based on sensitivity analysis. A permanent drainage rate of 300 $m^3$/day (55 gpm) was selected for building foundation design.

## Design Considerations and Performance

The planned excavation level allowed the building to be supported by shallow foundations on the native, very dense, silty to sandy gravel and cobbles. For a footings and slab-on-grade foundation system, the development plan required excavating to about –11.7 meters, which was about 7.8 to 9.3 meters below the existing groundwater table.

The available site space allowed mostly open cut slopes with a shoring system equivalent to soldier piles and tiebacks installed in limited areas for excavation support.

Temporary construction dewatering was accomplished by installing a series of wells around the perimeter of the building footprint. Because the groundwater drawdown occurred primarily in the dense gravel/cobble layer, ground subsidence caused by dewatering was negligible. Monitoring data from construction dewatering indicated that the maximum total pumping rate was about 150 $m^3$/day (28 gpm), compared to the 400 $m^3$/day (70 gpm) estimated in the design study. This is equivalent in the groundwater model to a hydraulic conductivity of 0.3 m/day ($3.5 \times 10^{-4}$ cm/sec)

For permanent foundation dewatering, the owner had a choice between a permanent drainage system and a watertight "bathtub" structure. Instead of installing and operating a permanent drainage scheme in perpetuity, the basement structure could be designed to resist up to 8 meters of hydrostatic pressure. This would not only require tie-down piles, it would also substantially increase the required thickness and reinforcing in the basement walls and slab. Comprehensive waterproofing below the groundwater table would also be required. The substantial up-front cost of these structural elements was much greater than the long-term operation and maintenance costs of a permanent drainage system.

A permanent drainage system that included a 30-cm-thick layer of drain rock and 15-cm-diameter perforated PVC pipes was installed under the slab-on-grade floor (Figs. 3 and 4). The drainage system included a redundant sump/pump system, complete with uninterruptible power supply.

The building has performed well since its completion in 2009.

CASE STUDY B: TAICHUNG

The Taichung site occupies a city block surrounded by city streets. The building design called for a footprint of about 127 by 91 meters. The site grade varied about 2.5 meters across the site. The new building is two levels above and two levels below grade. The B2 floor is at about –4.2 meters.

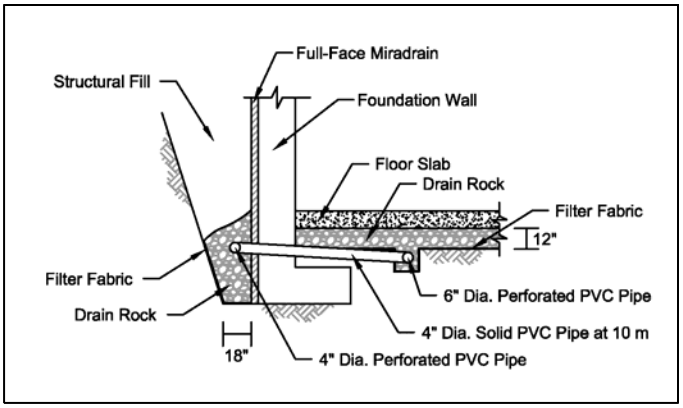


*Fig. 3. Drainage System Schematic - Hsinchu*

*Fig. 4. Drainage System Photo - Hsinchu*

Subsurface Conditions

Subsurface explorations included 13 test borings ranging from –15 to –30 meters (Fig. 5).

Soil units encountered from top to bottom were:

Fill. The surface soils consisted of mostly construction debris with an average thickness of about 1.5 meters (up to 2.8 meters).

Silty Clay. The Fill unit was underlain by a layer of soft to medium stiff, silty clay (old topsoil) from about –1.5 to –3.5 meters.

Gravel/Cobble. A very dense, sandy gravel and cobble deposit lay below the Silty Clay unit and extended to the termination depth of the borings at –15 to –30 meters.

Hydrogeological Characteristics

Monitoring wells were installed in each exploration and screened in the Gravel/Cobble unit. Groundwater at the site was encountered at approximately –5 to –6 meters within the unconfined Gravel/Cobble unit. The water-bearing Gravel/Cobble unit is part of a regional aquifer in the Taichung basin.

A constant rate aquifer-pumping test was conducted in a pumping well screened within the Gravel/Cobble unit at –5 to –15 meters. The pumping test network consisted of the pumping well and six observation wells arranged in an L-shaped configuration, with one set of wells perpendicular to the other set (Figs. 5 and 6).

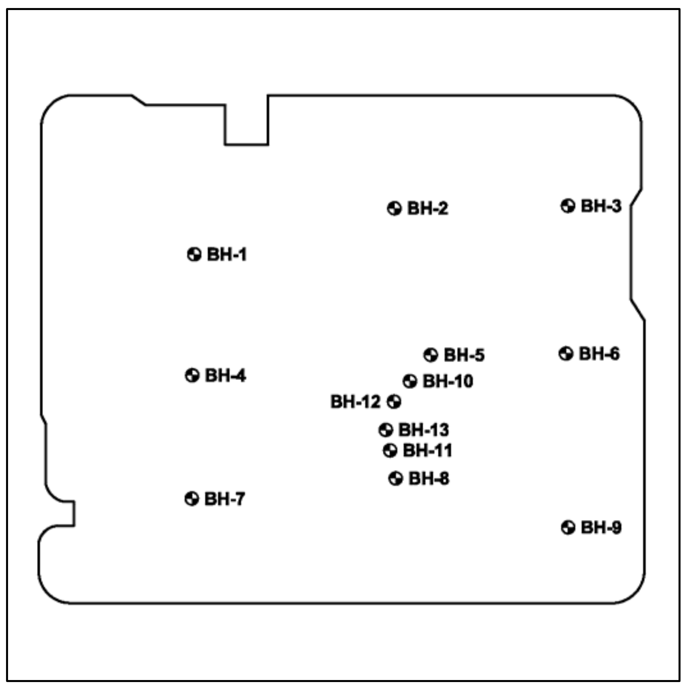


*Fig. 5. Exploration Map – Taichung*

*Fig. 6. Pumping Test Well Network - Taichung*

The pumping test was conducted at an average flow rate of 104 $m^3$/hour (458 gpm) for 72 hours. The maximum drawdown recorded in the pumping well was 7.31 meters. Figure 7 is a hydrograph of the drawdown and recovery data recorded during the test.

The constant rate discharge test data were analyzed using the Theis (1935) and Cooper-Jacob (1946) methods to estimate the transmissivity and storage coefficient of the aquifer. The

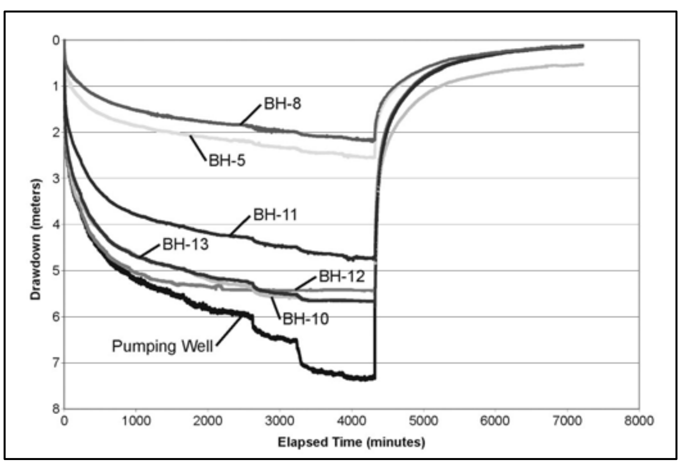


*Fig. 7. Pumping Test Hydrograph -Taichung*

average calculated transmissivity was 280 square meters per day. The Theis recovery method (1935) was applied to the recovery data of the six observation wells. The transmissivity from the recovery data was calculated to be 290 $m^2$/day. The value of 300 $m^2$/day was the designer's best estimate of the transmissivity of the aquifer.

The hydraulic conductivity was estimated by dividing the transmissivity by 10 m, which is the approximate length of the screen in the pumping well. The hydraulic conductivity was calculated to be 30 m/day. The storage coefficient was calculated to be from 0.02 to 0.06, which is consistent with storage values typical of unconfined aquifers.

## Groundwater Modeling

The goal of groundwater modeling at Site B was to estimate the extraction rates for temporary construction dewatering and permanent drainage systems, and compare the hydraulic performance of a permanent drainage system to a watertight structural system.

Under baseline conditions, the gravel/cobble aquifer was assigned a hydraulic conductivity of 30 m/d (0.35 cm/sec) and the constant head boundaries were assigned a head of –3.5 meters. For purposes of sensitivity and uncertainty analysis, the water table was varied from between –3.5 to –4.5 meters and the aquifer hydraulic conductivity from 30 to 60 m/day (0.35 to 0.7 cm/sec).

The structure includes below-grade parking levels, with the lowest finished floor (Level B2) at –4.3 meters. Given the likely thickness of the mat foundation (or structural slab and sub-slab drainage provisions), the site needed to be excavated to at least –4.75 to –5.1 meters during construction. To provide a dry work platform and avoid disturbance of the foundation subgrade, groundwater levels had to be lowered to at least –5.35 to –5.7 meters, which amounted to a drawdown of between 0.85 to 2.2 meters (3 to 7 feet) within the excavation footprint.

Lowering the water level by this amount could be achieved using dewatering wells installed around the perimeter of the excavation. Based on the results of groundwater modeling, the amount of water that had to be extracted from the excavation to achieve dry conditions down to –5.7 meters within the excavation footprint was estimated to be between 2,620 to 4,800 $m^3$/day (480 and 880 gpm).

The goal of the permanent drainage system was to maintain water levels below the base of the foundation slab at –4.75 meters. Based on modeling of the mostly likely groundwater conditions, the total groundwater seepage volume was estimated to be from 550 to 900 $m^3$/day (7 to 70 gpm). Because of the high permeability of the aquifer and the location of the site within the basin, groundwater levels can vary dramatically and a small rise in water levels can dramatically increase seepage volumes. Seepage rates are predicted to rise to over 16,350 $m^3$/day (3,000 gpm) in the event of unexpected recharge, and the groundwater level rises by 2 meters.

Design Considerations and Performance

The planned excavation level allowed the building to be supported by shallow foundations on the native, very dense sandy gravel and cobbles. Because of the type of foundation selected for this project, the development plan required excavation to about –4.75 to –5.1 meters, which was about 0.25 to 1.6 meters below the groundwater table.

The space available allowed mostly open cut slopes with a shoring system equivalent to large-diameter cantilevered soldier piles installed in limited areas for excavation support.

Site B required excavation to only about 0.25 to 1.6 meters below the existing groundwater table. Temporary construction dewatering was accomplished by installing a series of wells around the perimeter of the excavation. The dewatering volume was significantly lower than the estimated value because of lower groundwater levels at the time of site excavation. The low dewatering rate was likely due to a lower groundwater table resulting from seasonal fluctuation and construction dewatering activities from other nearby building sites. Because the groundwater drawdown occurred primarily in the dense gravel/cobble layer, ground subsidence because of dewatering was negligible.

For permanent foundation dewatering, a foundation system consisting of shallow footings and structural floors with a drainage system that included drain rock and a redundant sump/pump system was selected (Fig. 8).

Given that the flow volume in the highly permeable gravel/cobble layer was very sensitive to slight variations in the groundwater table, the owner elected not to entirely rely on the drainage system under the slab-on-grade floor to alleviate hydrostatic pressure. A reinforced structural slab was used to resist the hydrostatic pressure under the lowest basement floor. Given that the B2 floor slab is at or only slightly below the existing groundwater table, a 30-cm-thick layer of drain rock was placed above the structural slab to collect and drain the water that would seep through cracks in the slab. A much thinner slab-on-grade finish floor was installed over the drain rock. Waterproofing was not used on the floor slab. This hybrid system provided structural protection against hydrostatic uplift pressure and allowed water to seep through and be collected and drained before it reached the B2 floor.

The building was completed in 2007. The foundation and drainage system, in general, have performed well to date. A minor problem was observed in Level B2 where the finish floor slab appeared to have risen slightly at some of the corners at the construction joint between the floor slab and the support column (Fig. 9). This could have resulted from slight deformation of the lower structural slab due to seasonal groundwater fluctuation, and could have been mitigated by installing dowel bars across the construction joints between the finish floor and the column. No other major cracks were observed on the walls or finish floors. The vertical gaps were typically less than 1 to 2 cm, and the slab was ground down to level during maintenance work.

CASE STUDY C: TAINAN

The Tainan site was vacant, and plans called for a building footprint of about 120 by 84 meters. The site is encompassed by city streets to the east, west, and south, and a vacant parcel to the north. The site is generally flat. The warehouse was designed for two above-grade levels and three levels of underground parking, with the Level B3 floor at –10.5 meters.

Subsurface Conditions

Subsurface explorations included 14 test borings to a maximum depth of 30 meters below grade (Fig 10).

*Fig. 8. Footing Excavation – Taichung*


*Fig. 9. Repaired Joint - Taichung*

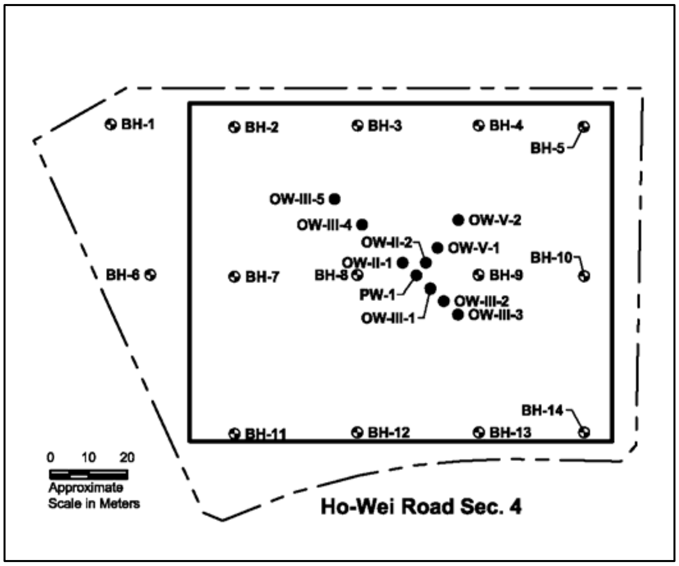


*Fig. 10. Exploration Map - Tainan*

Site soils encountered in the explorations consisted of interbedded silt/clay and sand units. Seven layers were identified in the borings as summarized below (Fig. 11).

Layer I - Fill. The surface soils consisted of mostly silty sand, sandy silt, and silty clay to about –3.2 to –4.8 meters. The average thickness of the fill unit is 3 meters.

Layer II - Soft Silty Clay. A layer of soft to medium stiff silty clay was encountered at about –6.3 to –11.0 meters. The average thickness of Layer II is 7 meters.

Layer III - Medium Dense Silty Sand. A medium dense to dense silty sand lay below the soft silty clay and extended to about –12.9 to –15.7 meters. The average thickness of Layer III is 4.5 meters.

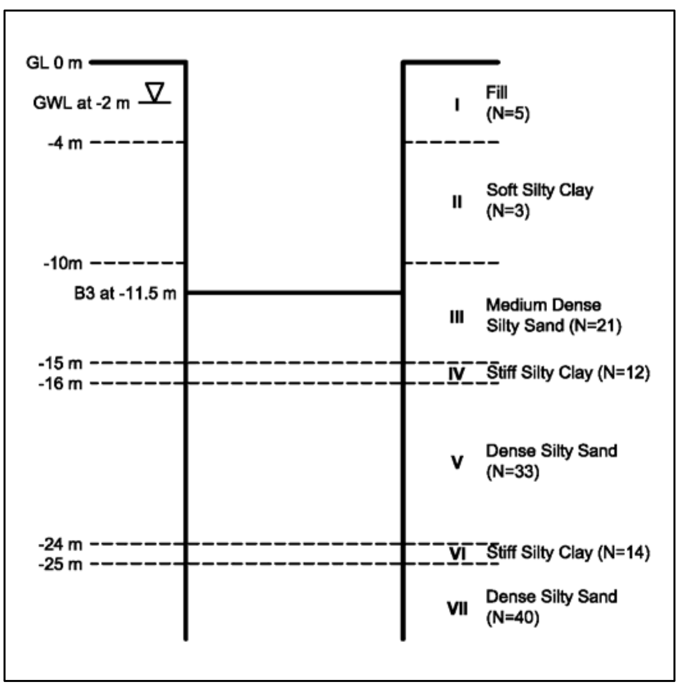


*Fig. 11. Site Stratigraphy - Tainan*

Layer IV - Stiff Silty Clay. A medium stiff to very stiff silty clay and clay silt was encountered at about –14.8 to –18.5 meters. Layer IV is about 1 meter thick and is either very thin or absent in two borings (BH-4, BH-10).

Layer V - Dense Silty Sand. A dense to very dense silty sand lay below the stiff silty clay and extended to about –21.5 to –25.7 meters. The average thickness of Layer V is 8 meters.

Layer VI - Stiff Silty Clay. A medium stiff to hard silty clay was encountered at about –23.9 to –26.8 meters. The average thickness of Layer VI is 2 meters.

Layer VII - Dense Silty Sand. A dense to very dense silty sand was encountered to the termination depth of the borings.

Hydrogeological Characteristics

Monitoring wells were installed in each exploration (Table 1). Groundwater at the site was generally at about –1.3 to –3.1 meters with an average of –2.4 meters. The site is located near the Taiwan Strait and the wells are screened below sea level. On-site water quality testing indicated that the groundwater had a conductivity of 50 ms/cm, which corresponded to a salt content of about 3.2 percent. Water temperature was about 30 degrees Celsius.

TABLE 1. WELL CONSTRUCTION SUMMARY - TAINAN

| Name | Depth to Screen Interval (m) | Layer Installed | Distance From Pump Well (m) |
|---|---|---|---|
| PW | 11~14 | III | - |
| OW-2-1 | 5.5~7.5 | II | 7.4 |
| OW-2-2 | 5.9~7.9 | II | 7.9 |
| OW-3-1 | 12.3~15.3 | III | 15.3 |
| OW-3-2 | 11~14 | III | 14 |
| OW-3-3 | 12.4~15.4 | III | 15.4 |
| OW-3-4 | 10.6~13.6 | III | 13.6 |
| OW-3-5 | 11.6~14.6 | III | 14.6 |
| OW-5-1 | 20.3~23.3 | V | 23.3 |
| OW-5-2 | 20.3~23.3 | V | 23.2 |
| BH-1 | 8~12 | III | 88.6 |
| BH-2 | 9.7~13.5 | III | 56.1 |
| BH-3 | 16.5~20.5 | V | - |
| BH-4 | 10~11.5 | III | 40.1 |
| BH-5 | 14~18 | V | 68.4 |
| BH-6 | 16~20 | V | 61.2 |
| BH-7 | 16~20 | V | 35.4 |
| BH-8 | 16~20 | V | 10.3 |
| BH-9 | 17~21 | V | 19.9 |
| BH-10 | 14~18 | III | 67.9 |
| BH-11 | 13.5~17.5 | III | 62.1 |
| BH-12 | 16.3~20.3 | V | - |
| BH-13 | 8~12 | III | 55.8 |
| BH-14 | 11.5~14.5 | V | - |

A pumping test was performed to determine the hydraulic properties of Layer III, to evaluate groundwater drawdown, and to assess the nature of the hydraulic connection between Layers III and V.

A constant-rate aquifer-pumping test was conducted in a pumping well screened in Layer III at –11 to –14 meters. Water levels were monitored in the pumping well and nine observation wells. Two observations wells were screened in Layer II; five observation wells were screened in Layer III; and two observations wells were screened in Layer V (Table 1). A layout of the wells is shown on Fig. 10.

The pumping test was conducted at an average pumping rate of 3.24 $m^3$/hour (14.3 gpm) for 72 hours. The maximum drawdown recorded in the pumping well was 9.7 meters. The effects of a magnitude 6.3 earthquake were recorded during the pumping test. Figure 12 shows a hydrograph of the drawdown and recovery data for wells completed in Layer III. The water level response for wells in Layer II and V during the pumping test is shown in Figure 13. Little response was noted in wells completed in Layers II and V suggesting that the hydraulic connection between Layer III and V is limited. The changes in water levels recorded in Layer II and V during the pumping test are likely due to a combination of pumping test effects and tidal influences.

The constant rate discharge test data were analyzed using the Theis (1935) and Cooper-Jacob (1946) methods. The hydraulic conductivity of the Layer III sand was about 3.6 x $10^{-3}$ cm/sec (3.1 m/day).

Groundwater Modeling

The goal of groundwater modeling at Site C was to estimate the extraction rates for temporary construction dewatering and permanent drainage systems, and compare the hydraulic performance of various configurations of a cutoff wall.

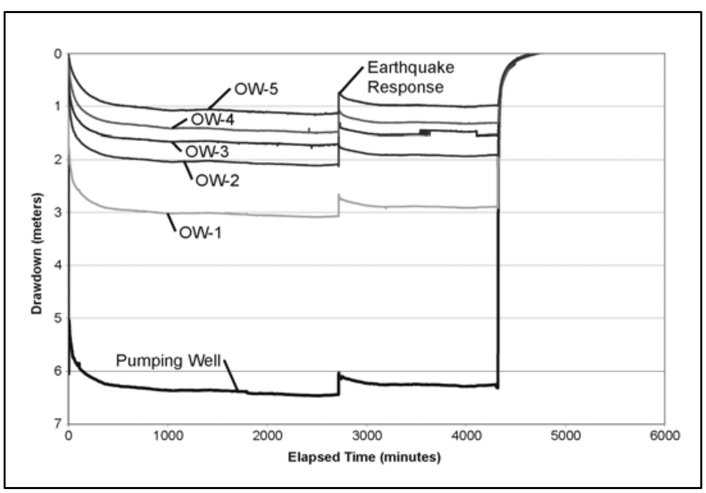


*Fig. 12. Pumping Test Hydrograph Layer III Wells – Tainan*

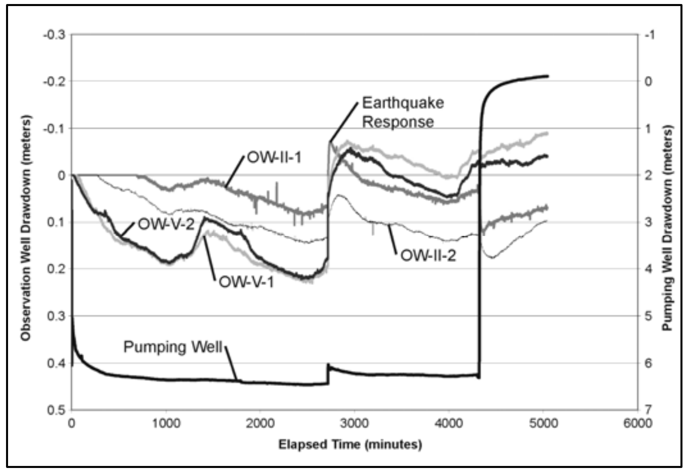


*Fig. 13. Pumping Test Hydrograph Layers II & V - Tainan*

The following assumptions were used in developing and evaluating the groundwater model for Site C. Design groundwater for the site was 0 meters. The lowest finished floor (Level B3) is at –10.5 meters. Given the likely thickness of the footing, the site needed to be excavated to at least –11.5 meters during construction. To provide dry working conditions at the bottom of the excavation and avoid heaving or disturbance of the foundation subgrade, it was determined that groundwater needed to be drawn down to at least –14 meters corresponding to the bottom of Layer III.

The groundwater model was constructed using seven layers corresponding to the seven soil layers identified at the site. The thickness of the model layers were assigned a value equal to the average thickness of the corresponding soil layer. The sand layers (III, V, and VII) were assigned a hydraulic conductivity of 3.1 m/day (4 x $10^{-3}$ cm/sec). Layers II, IV, and VII consisting of silt/clay were assigned a hydraulic conductivity of 0.009 m/day (1 x $10^{-5}$ cm/sec). Each model layer in the baseline model was assigned a constant head of 0 meters. For purposes of sensitivity and uncertainty analysis, the water table was varied from between 0 to –2 meters and the aquifer horizontal hydraulic conductivity by factors of 0.75 to 1.5. The vertical hydraulic conductivity values were set at 10 percent of the horizontal hydraulic conductivity values.

A 0.9-meter-thick cutoff wall surrounding the building footprint was assigned a hydraulic conductivity of 0.0009 m/day (1 x $10^{-6}$ cm/sec). Various depth configurations (Layers III through VI) of a cutoff wall were tested.

Groundwater modeling results showed that a cutoff wall would be required to minimize groundwater drawdown beyond the site boundaries. Without a cutoff wall, groundwater drawdown at the site boundary would be as much as 8 to 10 meters when dewatering Layer III to –11.5 meters (Fig. 14). Layer II would be completely dewatered at a distance of 50 meters from the site boundaries. Testing various cutoff depths showed that the deeper the cutoff wall, the less drawdown in Layers III and V would occur.

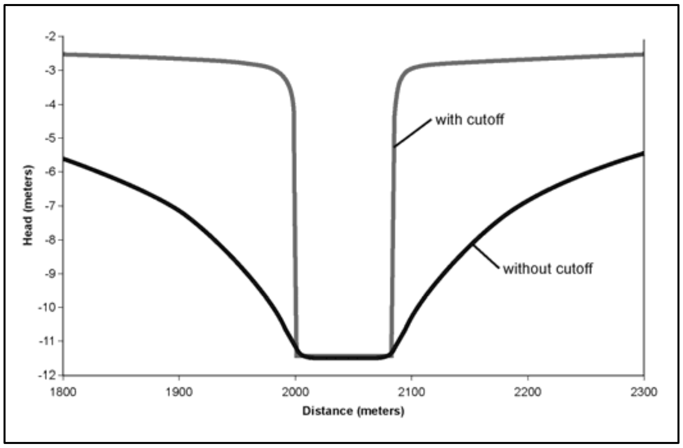


*Fig. 14. Simulation Dewatering Response with and without Cutoff Wall - Tainan*

Groundwater modeling indicated that the water-bearing sand in Layer V would be depressurized to a water head of about 8 meters for a cutoff wall installed to –28 meters. The recommended dewatering program would provide a minimum factor of safety of 1.2 against the risk of heave or piping due to excessive hydraulic gradients at the bottom of the excavation.

Groundwater modeling and settlement analysis indicated that a water head drawdown in the Layer V sand to 8 meters outside the cutoff wall would likely fully drain the Layer II soft clay and result in excessive settlement of adjacent ground. By providing a cutoff wall to –28 meters, dewatering was limited to inside the hydraulic cutoff wall, which resulted in a lower pumping rate and negligible settlement outside the property line.

Based on the groundwater modeling results, the amount of water that had to be extracted during construction dewatering from the excavation footprint to maintain water levels at –14 meters was estimated to be between 300 to 550 $m^3$/day (55 to 100 gpm). A design value of 550 $m^3$/ day (100 gpm) was selected for construction dewatering.

Inflows into the permanent drainage system were predicted to be on the order of 300 to 500 $m^3$/day (55 to 100 gpm) based on a long-term groundwater table at ground surface outside the cutoff wall and a target drawdown to –11.5 meters inside the cutoff wall.

Design Considerations and Performance

Local engineers initially considered supporting the building using a structural mat foundation and piles to resist the uplift pressure from buoyancy. Temporary construction dewatering to drain and depressurize Layers III and V sands would be necessary to construct the building below the groundwater table. A deep slurry wall was considered to avoid dewatering outside the cutoff wall, which could draw down regional groundwater and cause excessive ground subsidence. Based on the laboratory consolidation test data, it was estimated that a groundwater drawdown of 8 meters would likely fully drain the Layer II soft clay, resulting in about 10 to 12 cm of ground settlement at the property line and about 5 cm at 200 meters from the property line.

Local construction practices typically require the use of more rigid shoring systems in deep alluvial soils such as Site C. A displacement-based analysis estimated that lateral displacement using steel sheet piles and internal steel bracing would exceed 10 cm (4 inches). This ground displacement was considered unacceptable because of the effects on adjacent structures and underground utilities. A more rigid concrete diaphragm shoring wall constructed by slurry method was selected for the shoring system (Fig. 15).

Given that a deep slurry cutoff wall was required for shoring and seepage control, it was decided to install a permanent dewatering system to allow the building to be supported by shallow footings and slab-on-grade floors. The slurry cutoff wall was extended below the Layer VI clay to –27 meters to allow dewatering in the Layer III sand to avoid heave and piping at the bottom of the excavation. Six permanent dewatering wells were installed inside the building footprint to drain the water in the Layer III sand, which would prevent soil liquefaction during the design earthquake. This system also helped depressurize the Layer V sand to control uplift, and allowed the use of spread footings and slab-on-grade floors to be supported on the Layer III sand.

Six dewatering wells were installed to a minimum depth of –18 meters within the building footprint inside the slurry cutoff wall. The minimum inside diameter of the wells was 30 cm. Well construction and sump pump selection considered the saline water, because the wells and pumps were incorporated into the permanent dewatering/drainage system after completion of the below-grade levels. Water conveyance from the wells to the sump pits were kept separate from the floor slab drainage. As part of the dewatering system, observation wells were installed to monitor the head in the Layers III and V sands inside the cutoff wall, as well as the groundwater drawdown in Layers II and III soils outside the property line.

*Fig. 15. Installation of Slurry Wall - Tainan*

An underslab drainage layer was constructed under the slab-on-grade floor, which included 30 cm of drain rock hydraulically connected to multiple sump pumps (Fig. 16). Given that water would be collected primarily by the dewatering wells, a system of cross drains (15-cm-diameter perforated PVC pipes bedded in the drain rock) was not necessary. The drain layer was constructed of well-graded, free-draining coarse sand and gravel (less than 3 percent fines based on the minus 3/4-inch fraction). To prevent clogging, a layer of geotextile filter fabric was placed between the drainage material and native soil. The drainage system included a redundant sump/pump system complete with uninterruptible power supply.

The building has performed well since its completion in 2011.

CONCLUSIONS

As described in these three case studies, extensive geotechnical and hydrogeological explorations and testing were performed at each of the sites to characterize the hydraulic conductivity and model groundwater responses to dewatering. The actual reported seepage flow volumes are, in general, on the same order of magnitude or lower than the values estimated from the modeling.

In Sites A and B where foundations were placed on very dense gravel and cobbles, the use of permanent drainage systems under the floor slab to draw down the groundwater table allowed the buildings to be supported on the more cost-effective shallow footings and slab-on-grade floors. An additional structural slab was used at Site B to resist the hydrostatic pressure.

In Site C, where the interbedded sand and clay were the predominant soil type, a more robust slurry (reinforced concrete diaphragm) cutoff wall was required to limit lateral movement during excavation and to control groundwater seepage for both construction and permanent conditions. The use of the slurry cutoff wall and permanent dewatering wells also mitigated soil liquefaction in the Layer III sand and allowed the building to be supported on the more cost-efficient shallow footings and slab-on-grade floors.


*Fig. 16. Drainage Layer Installation – Tainan*

ACKNOWLEDGEMENTS

The authors would like to thank Eric Lindquist for drafting support, Sue Enzi for word processing and formatting, and Mary Gutierrez for editorial review of the manuscript.

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