دائرة الشؤون البلدية

بلدية مدينة أبوظبي

ABU DHABI CITY MUNICIPALITY

DEPARTMENT OF MUNICIPAL AFFAIRS

ABU DHABI CITY MUNICIPALITY
TOWN PLANNING SECTOR

GEOTECHNICAL, GEOPHYSICAL, AND HYDROGEOLOGICAL INVESTIGATION PROJECT (GGHIP)

DEWATERING GUIDELINES

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Dewatering Guidelines

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GEOTECHNICAL, GEOPHYSICAL, AND HYDROGEOLOGICAL INVESTIGATION PROJECT (GGHIP)

DEWATERING GUIDELINES

1.0 INTRODUCTION

1.1 OBJECTIVES OF THE GUIDANCE DOCUMENT

These Guidelines are prepared by the Town Planning Sector of Abu Dhabi City Municipality (ADM) and presents standard dewatering practices to support dewatering applications in the ADM. These Guidelines provide information for local dewatering engineers and contractors. A special emphasis is given to control and to eliminate the future subsidence/collapse problems related to dewatering projects in the ADM area.

In order to minimize future subsidence/collapse problems related to dewatering, construction companies and dewatering contractors need to be informed about the risks associated with dewatering and possible mitigative measures that can be employed to reduce risks. These Guidelines address issues and engineering approaches to lessen the potential risks associated with construction dewatering. This includes a review of engineering practices employed worldwide, guidance for conducting dewatering activities, and permit application packages, which will allow potential risks to be evaluated. These Guidelines also include examples of permit application packages for three types of dewatering projects; one involving shallow dewatering, one involving intermediate depth dewatering, and one involving deep dewatering. These Guidelines are a living document and will be updated as required, as additional information is collected and additional local experience is gained.

1.2 ADM REGULATORY RESPONSIBILITY AND ROLE

ADM is responsible for approval of permit applications for projects that involve dewatering. However, the contractor performing the work and the consultant overseeing the design and construction have the ultimate responsibility for designing and implementing effective dewatering measures. These Guidelines provide information and examples that may be used in support of the design and evaluation of projects; however, they are not a substitute for engineering judgment and experience. Geotechnical conditions vary between project sites and each project is unique, so the contractor and consultants are responsible for thoroughly evaluating each site and developing effective solutions.

ADM has been regulating the dewatering permits in the City Municipality area. An online permit application process has been in effect since 2012. These Guidelines are prepared to provide guidance during the dewatering permit application process.

For every dewatering application, ADM will require an application and supporting documents, and for important and critical dewatering projects, an independent review of the application documents will be required by a third-party consultant. The design and monitoring of the dewatering system shall be approved and reviewed by an entity licensed for similar activities by the Department of Economic Development before it is submitted to ADM. The designer of the dewatering system will be responsible for a thorough technical evaluation of the following information packages:

- Field investigation
- Dewatering design
- Monitoring program

These Guidelines are designed to support designers, contractors, consultants, and third-party reviewers regarding technical aspects of the dewatering project according to the three main phases listed above. The requirements addressed in these Guidelines are scalable to the magnitude of the dewatering project.

1.3 GUIDANCE BENEFITS FOR CONTRACTORS, CONSULTANTS, LANDOWNERS, AND OTHER STAKEHOLDERS

These Guidelines provide dewatering contractors, consultants, landowners, and the community at large with information related to the risks associated with dewatering and the mitigation measures available for addressing them. The Guidelines will be effective in helping to avoid dewatering-related subsidence and damage problems.

2.0 GROUNDWATER IN CONSTRUCTION

Groundwater is frequently encountered in construction projects and needs to be accounted for and dealt with during design and construction to complete the project successfully. This Section provides a brief overview of reasons for dewatering, technology that may be employed, and potential effects of dewatering.

2.1 REASONS FOR CONSTRUCTION DEWATERING

The most common reason for construction dewatering is to lower the water table at a site to a level that will allow for excavation to a foundation level and safe, dry construction of a structure. In each of these applications, construction dewatering must be carefully planned and executed to ensure that the dewatering system provides the desired outcome for the Project and to limit the risk of unwanted impacts to the environment, adjacent structures, and personal safety.

2.2 DEWATERING TECHNOLOGY

Typical modern dewatering systems used worldwide may include: drainage features such as swales, sumps, and surface pumping; small diameter wellpoints operating via suction from the surface (generally in relatively clean sands and shallow depths); ejector (also known as eductor) systems, which are adapted jet pumps useful in dewatering fine particles and effective at depths greater than suction devices; dewatering wells in a wide range of diameters paired with electric submersible pumps and capable of being installed at great depths in rock or soil conditions with properly designed screens and filters; and other more specialized techniques, such as cutoff walls, grouting, and horizontally installed trench drains. In Abu Dhabi City, dewatering contractors implement well points for dewatering in soil overburden, sumps for shallow dewatering, and deep wells for dewatering deep excavations in rock.

Care must be taken, however, during the dewatering system design process to select methods that will be effective for the expected subsurface conditions, and it is recommended to allow flexibility in the design to account for variations encountered during construction, whether they are natural or man-made.

Figures 2-1 through 2-5 provide examples of dewatering operations in Abu Dhabi City.

FIGURE 2-1
DEEP WELL INSTALLED IN THE DIAPHRAGM WALL OF A DEEP EXCAVATION IN ABU DHABI ISLAND, 2014

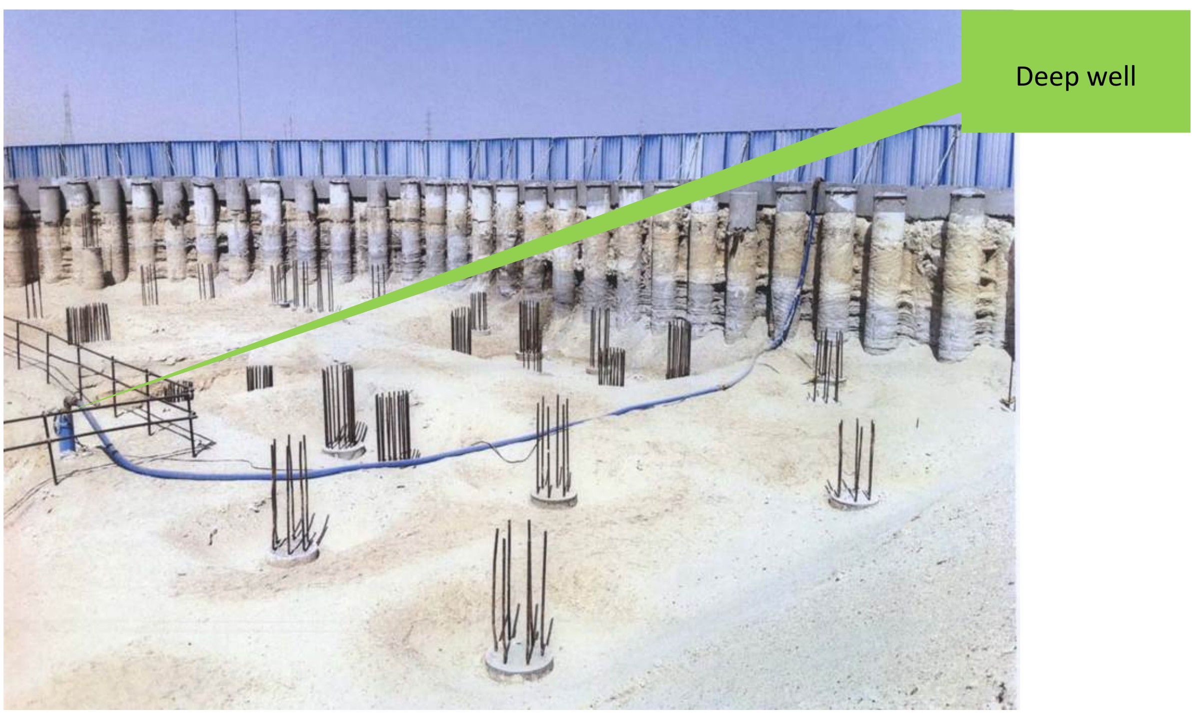

FIGURE 2-2
DEEP WELL INSIDE AN EXCAVATION IN SHAKHBOUT CITY, 2011

FIGURE 2-3
UNCONTROLLED DEWATERING FROM A SUMP WITHIN A SHORED PIT, SHAKBOUT CITY, 2013

FIGURE 2-4
WELL POINTS DEWATERING SYSTEM IN SHAKHBOUT CITY, 2011

FIGURE 2-5
WELL POINTS DEWATERING SYSTEM FOR AN INFRASTRUCTURE UTILITY LINE IN KHALIFA CITY, 2010

2.3 POTENTIAL EFFECTS OF DEWATERING

Dewatering can cause changes in porewater pressure, which changes the effective vertical stress in a soil mass. Dewatering can also cause changes in groundwater chemistry which in turn can cause changes in porewater pressure and also dissolution of soluble minerals in the soil and rock. Changes in porewater pressure and groundwater chemistry can have significant impacts on the soil and the rock. Potential effects of dewatering include:

- Change in effective stresses and shear strengths in soils
- Changes in seepage velocities and pressures
- Erosion or transport of soil particles and piping
- Settlement
- Collapse of subsurface cavities or voids
- Transport of groundwater containing contaminants
- Dissolution of soluble materials such as rock salt or gypsum

These effects can have a significant impact on the area being dewatered and on nearby structures located within the zone of dewatering influence. Proper dewatering design must take into consideration the potential negative impacts and implement mitigation measures to minimize or eliminate these impacts.

Several cases of ground failure and extensive settlements have been attributed to nearby uncontrolled dewatering activities. Ground failures and associated settlements exceeding 1.5 meters (m) occurred in Sector SW-12 of Khalifa City as shown on ***Figures 2-6 through 2-11***.

Other cases of ground failure and extensive settlements attributed to uncontrolled dewatering also occurred in Shakhbout City Road 5 (***Figures 2-12 and 2-13***), and in some areas of Mohammed Bin Zayed City.

Other examples of damage in Shakhbout City and Shamkha, Sector SH13, are given on ***Figures 2-14 through 2-17.***

FIGURE 2-6
FAILURE IN AN INTERNAL ROAD IN KHALIFA CITY SECTOR SW-12 ATTRIBUTED TO NEARBY DEWATERING

FIGURE 2-7
CLOSE UP OF FAILURE SHOWN ON FIGURE 2-6

FIGURE 2-8
GROUND FAILURE IN A RESIDENTIAL PLOT IN KHALIFA CITY SECTOR SW-12 ATTRIBUTED TO NEARBY UNCONTROLLED DEWATERING, 2010

FIGURE 2-9
DAMAGE TO SHALLOW SUPPORTED BOUNDARY WALL IN KHALIFA CITY SECTOR SW-12 DUE TO GROUND SETTLEMENT ATTRIBUTED TO A NEARBY UNCONTROLLED DEWATERING, 2010

FIGURE 2-10
GROUND FAILURE IN THE SIDEWALK OF A RESIDENTIAL PLOT IN KHALIFA CITY, SECTOR SW-12, ATTRIBUTED TO A NEARBY UNCONTROLLED DEWATERING, 2010

FIGURE 2-11
TILTING OF GRADE SUPPORTED GUARD ROOM IN KHALIFA CITY, SECTOR SW-12

FIGURE 2-12
COLLAPSE OF ROAD 5 AND ADJACENT PLOT IN SHAKHBOUT CITY CAUSED BY ADJACENT UNCONTROLLED DEWATERING ACTIVITIES, 2011

FIGURE 2-13
COLLAPSE OF ROAD 5 AND ADJACENT PLOT FROM UNCONTROLLED DEWATERING ACTIVITIES

Not all ground failures and settlement in Abu Dhabi City can be attributed to dewatering. Other causes and triggering factors for ground failure may include:

1. Construction activities, especially those associated with unsupported excavations or poorly designed shoring system.

2. Placement of uncontrolled or poorly compacted fill and its subsequent settlement under traffic, its own weight, or by hydro-compaction.

3. Leaks from underground utility pipes which can cause erosion in surrounding loose soils.

4. Placement of highly permeable layers in road layers, which can cause concentration of surface water flow and erosion of fine materials above and below.

5. Lack of storm water drainage system in inland areas of Abu Dhabi, which can cause flash floods and erosion of road and shoulder materials and collapse of pavements.

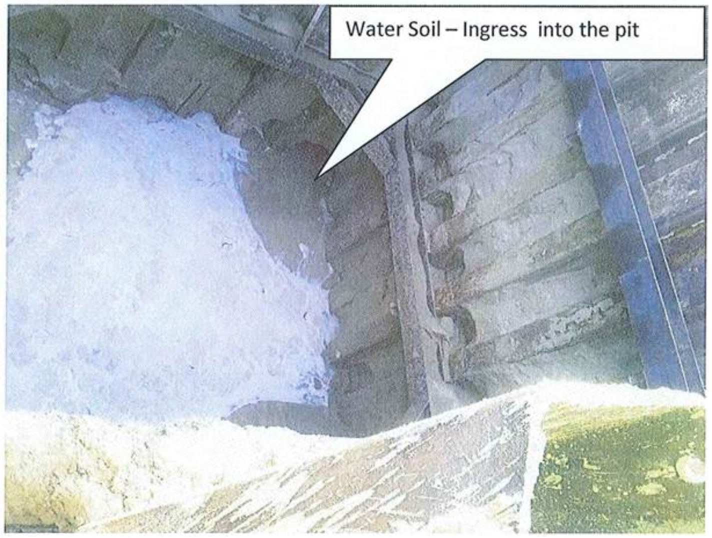

FIGURE 2-14
WATER-SOIL INGRESS INTO THE PIT OF A SHORED EXCAVATION DUE TO POOR INTERLOCKING OF SHEET PILES, SHAKHBOUT CITY, 2013

FIGURE 2-15
FORMATION OF A CAVITY OUTSIDE THE PIT DUE TO GROUND LOSS CAUSED BY POOR INTERLOCKING OF SHEET PILES, SHAKHBOUT CITY, 2013

FIGURE 2-16
COLLAPSE OF INTERNAL ROADS IN SHAMKHA, SECTOR SH-13, IN MARCH 2014 DUE TO EROSION OF ROAD SUBGRADE AND LOOSE UNDERLYING LAYERS FOLLOWING HEAVY RAINSTORMS

FIGURE 2-17
EXPOSED ROAD BASE, SUBBASE, AND UNDERLYING LOOSE LAYERS OF A COLLAPSED SECTION OF AN INTERNAL ROAD IN SHAMKHA, SECTOR SH-13, MARCH 2013

3.0 GENERAL GEOLOGICAL AND HYDROGEOLOGICAL CONDITIONS IN THE ADM REGION

This Section provides a summary of geological and hydrogeological conditions in the ADM. Additional information is available in the references included in ***Section 15.0***.

3.1 GEOLOGICAL CONDITIONS

Most surfaces in the ADM are covered with a variable amount of made ground as much of the coastal strip has been reclaimed or developed. In the coastal strip, the made ground is often composed of carbonate sand dredged from neighboring lagoons. Further inland, many areas have been extensively landscaped, with large areas of dunes leveled flat or quarried for fill (Farrant et al., 2012a).

Underlying the made ground is typically Quaternary sediments, including a range of marine and supratidal facies such as coastal spits, bars and beach ridges, lagoonal muds, algal mats, and ooidal tidal deltas of the Abu Dhabi Formation deposited in the coastal areas (Farrant et al., 2012a, b). Landward of the coastal deposits is mostly Aeolian sand, of the Ghayathi Formation and Fluvial sand and gravel deposits of the Hili Formation, which represent Quaternary outwash from the Hajar Mountains to the east.

In much of the ADM area, there has been extensive carbonate-evaporitic sabkha development, particularly in the coastal region, but also in some interdune locations (***Figure 3-1***). The sabkha is characteristically found in low lying areas, which are prone to flooding at high spring tides. They consist mainly of loose silty fine carbonate sands, with cementation increasing with depth especially in coarser grained deposits.

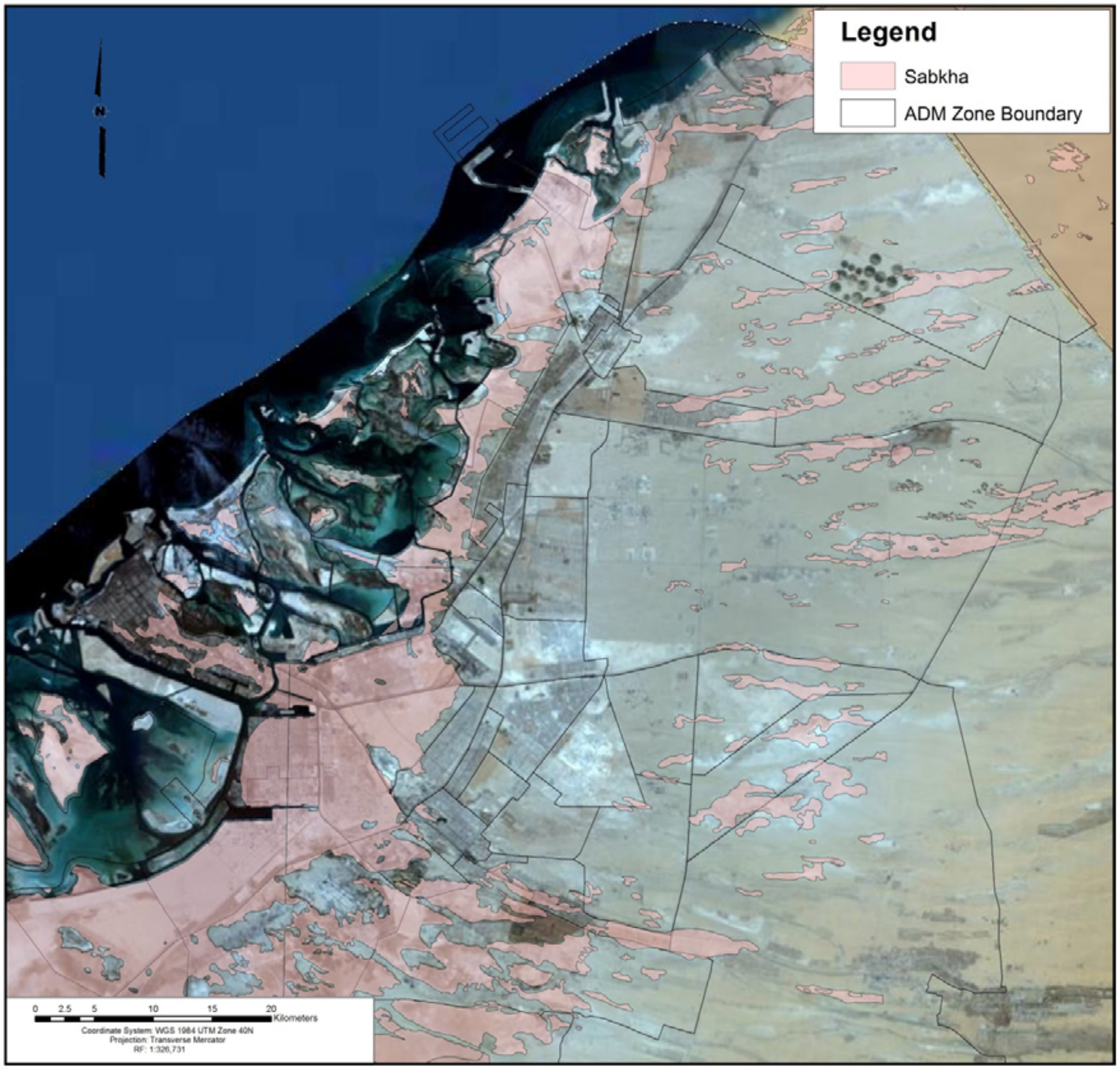

FIGURE 3-1
SABKHA DISTRIBUTION MAP
(Adapted from Farrant et al., 2012a, 2012b, 2012c, and Thomas et al., 2012b)

Figure 3-2 shows some excavated sections through typical sabkha sediments. These photos reveal the soft, loose, and wet original ground conditions that existed throughout much of the subsurface of coastal zone before extensive development. The photos also show the sandy, salty, and gypsiferous materials that make up the coastal and inland sabkha.

FIGURE 3-2
PHOTOGRAPHS OF SUBSURFACE SECTIONS THROUGH TYPICAL SABKHA SOILS IN ABU DHABI

Stratigraphically, below these sediments are rock layers of the middle to late Miocene age, referred to locally as part of the Fars Group (Farrant et al., 2012a). These units, which are composed of sandstones of the Shuwaihat and Baynunah Formations in the southern areas, dolomitic conglomerates, sandstones and siltstones of the Barzaman Formation in the northern areas, dolomites and limestones of the Dam Formation, and evaporitic mudstone and siltstone of the Gachsaran Formation (Farrant et al., 2012b). These units are mostly present in the eastern portions of the ADM in the higher elevation mesas, where they are overlain by Aeolian sand and made ground.

Descriptions of the sediments and rocks generally encountered at shallow depth in different regions across the ADM area are summarized below in in ***Tables 3-1 through 3-4***.

TABLE 3-1
GENERALIZED STRATIGRAPHIC COLUMN IN THE COASTAL ISLANDS
(e.g., Abu Dhabi Island, Saadiyat Island)

AGE	APPROXIMATE ELEVATION RANGE (m) (NADD)	AVERAGE THICKNESS RANGE (m)	STRATIGRAPHIC UNIT	COMMON LITHOLOGIES	AQUIFER / AQUITARD
Holocene	+10 to -5	0 to 10	Made Ground	Silty sand with gravel and bioclasts	Unconfined aquifer
	0 to -5	2 to 5	Rub al Khali Formation	Aeolian sand and silty sand	
	+5 to -10	3 to 10	Abu Dhabi formation	Calcareous silty sand with some gravel and bioclasts, clay and silt layers and lenses	
Pleistocene	-3 to -13	5 to 10	Saadiyat Formation	Marine limestone and calcarenite	Unconfined aquifer (occasionally confined aquifer)
	0 to -15	5 to 10 m	Ghayathi Formation	Calcareous silty sand or sandstone/calcarenite	
Miocene	-10 to > -100	>100	Gachsaran Formation	Interbedded mudstone, siltstone and gypsum	Aquitard

Note:

National Abu Dhabi Datum (NADD)

TABLE 3-2
GENERALIZED STRATIGRAPHIC COLUMN IN THE COASTAL SABKHA REGION
(e.g., Khalifa City, Mohammad Bin Zayed)

AGE	APPROXIMATE ELEVATION RANGE (m) (NADD)	AVERAGE THICKNESS RANGE (m)	STRATIGRAPHIC UNIT	COMMON LITHOLOGIES	AQUIFER / AQUITARD
Holocene	+5 to 0	< 1 to 4	Made Ground	Silty sand with gravel and bioclasts	Unconfined aquifer
	+3 to -8	3 to 10	Abu Dhabi formation	Calcareous silty sand with some gravel and bioclasts, clay and silt layers and lenses	
Pleistocene	-5 to -10	0 to 3 m	Ghayathi Formation	Calcareous silty sand or sandstone/calcarenite	Unconfined aquifer (occasionally confined aquifer)
Miocene	-10 to > -100	>100	Gachsaran Formation	Interbedded mudstone, siltstone and gypsum	Aquitard

Note:

National Abu Dhabi Datum (NADD)

TABLE 3-3
GENERALIZED STRATIGRAPHIC COLUMN ACROSS THE MIOCENE ESCARPMENT
(e.g., Shakhbout City)

AGE	APPROXIMATE ELEVATION RANGE (m) (NADD)	AVERAGE THICKNESS RANGE (m)	STRATIGRAPHIC UNIT	COMMON LITHOLOGIES	AQUIFER / AQUITARD
Holocene	+25 to 0	0 to 5	Made Ground	Silty sand with gravel and bioclasts	Unconfined aquifer
	+5 to -5	0 to 5	Abu Dhabi formation	Calcareous silty sand with some gravel and bioclasts, clay and silt layers and lenses	
Pleistocene	+25 to +10	0 to 5	Ghayathi Formation	calcareous silty sand or sandstone/calcarenite	Unconfined aquifer (occasionally confined aquifer)
	+25 to +10	0 to 8	Hili Formation	Silty sand, gravelly sand, silty sandy gravel or sandstone, gravelly sandstone and sandy conglomerate,	
Miocene	+20 to -5	0 to 10	Baynunah Formation	Sandstone and siltstone with mudstone	
	+5 to > -100	>100	Gachsaran Formation	Interbedded mudstone, siltstone and gypsum	Aquitard

Note:

National Abu Dhabi Datum (NADD)

TABLE 3-4
GENERALIZED STRATIGRAPHIC COLUMN IN THE EASTERN ADM
(e.g., Al Shamkha, Baniyas)

AGE	APPROXIMATE ELEVATION RANGE (m) (NADD)	AVERAGE THICKNESS RANGE (m)	STRATIGRAPHIC UNIT	COMMON LITHOLOGIES	AQUIFER / AQUITARD
Holocene	+45 to +25	0 to 10	Made Ground	Silty sand with gravel and bioclasts	Unconfined aquifer
	+40 to +20	0 to 8	Rub al Khali Formation	Aeolian sand and silty sand	
Pleistocene	+35 to +20	0 to 10	Ghayathi Formation	calcareous silty sand or sandstone/calcarenite	Unconfined aquifer (occasionally confined aquifer)
	+25 to +10	0 to 8	Hili Formation	Silty sand, gravelly sand, silty sandy gravel or sandstone, gravelly sandstone and sandy conglomerate,	
Miocene	+20 to 0	8 to 20	Baynunah Formation	Sandstone and siltstone with mudstone	
	+10 to > -100	>100	Gachsaran Formation	Interbedded mudstone, siltstone and gypsum	Aquitard

Note:

National Abu Dhabi Datum (NADD)

During various geotechnical investigations in the ADM area, voids have been identified by tool drops in fractured calcareous mudstones/siltstones with gypsum inclusions, calcarenite, and sands, between or above massive gypsum layers, and in what is referred to as the weathered/fractured top of rock. Intensive dewatering has also been interpreted as having the potential for either increasing the size of pre-existing voids in subsurface, or creating them by removal of fine particles that is commonly found in the voids (Farrant et al., 2012b). Irrigation of forestry, gardens, and farmland areas inland is exacerbating the situation by increasing local groundwater head, which, coupled with construction related dewatering within the urban area, are changing the hydraulic gradient creating one of the key triggers for sinkhole development.

Tool drops and loss of water circulation have also been described in the calcarenite layers encountered at depth in the ADM. These calcarenites are widespread, and occur at many different levels and with highly varied thicknesses (from 0.1 to 6 meters).

The occurrence of washout zones in salt layers below groundwater level has also been identified as probable cause for the presence of voids in the subsurface. Gaps in these salt layers are thought to form from both historical changes in groundwater levels and as a result of recent intensive dewatering in the surrounding areas. Core losses associated with these washout zones generally occur in calcareous mudstone, which is washed away during the drilling process.

Water loss is also commonly associated with highly permeable soils (silty sand with gravel/shells, and gravels). Settlement has been observed in certain areas after water level drops due to intense dewatering during construction activities, especially in those areas in which more permeable soils are described (gravel, sandy gravel, and sand bars). Dissolution of salt crystals in fill material is also a factor contributing to settlement in areas of the ADM.

3.2 HYDROGEOLOGICAL CONDITIONS

Very little published information is available with regard to the hydrogeology of the ADM's 89 municipal zones and the "core" ADM zones, wherein significant geotechnical issues have been encountered (e.g., Mohammed Bin Zayed City, Khalifa City, and Shakhbout City (previously Khalifa City B)). Hydrogeological data for the ADM is effectively limited to the static water levels reported for geotechnical boreholes completed in conjunction with major infrastructure and building projects, although some long-term groundwater level monitoring data, groundwater quality data, and minimal information related to aquifer or aquitard hydraulic properties are available.

Of the shallow rock units described in ***Section 3.1***, the lowermost (and thickest) strata underlying the ADM include alternating layers of gypsum and mudstone associated with the Gachsaran Formation (Miocene age). Hydrogeologically, this formation is considered a major aquitard that separates the shallow groundwater flow system from groundwater in deeper bedrock formations. However, the upper 5 to 20 m of the Gachsaran is weathered and solution cavities and karst features have been found in the upper gypsum layers contained within the Gachsaran Formation (Farrant et al., 2012a, 2012b).

Gachsaran Formation rocks (i.e., the Gachsaran aquitard) are overlain by approximately 10 to 20 m of Baynunah, Hili, Ghayathi, and Saadiyat bedrock deposits. In the ADM, the Baynunah, Hili, Ghayathi, and Saadiyat deposits constitute an uppermost bedrock aquifer. The upper 5 to 20 m of the Gachsaran Formation is also included in the shallow bedrock aquifer because of the presence of solution cavities. It should be noted that Barzaman sand and gravels, also part of the uppermost bedrock aquifer, are not observed in many areas within the ADM, but that these deposits are found in more northerly areas.

Above the bedrock lies approximately 5 to 25 m of clay, silt, sand, gravel, and fill deposits (unconsolidated overburden), which includes all beach sands and dune sands. Together, these unconsolidated overburden deposits constitute the Quaternary aquifer. In many areas within ADM, the unconsolidated deposits are very thin or absent. In these areas, bedrock units are exposed at the surface.

Interpreted potentiometric surfaces for the ADM indicates that groundwater elevations in the western and north-central ADM are relatively flat, ranging from approximately -5 up to about 10 meters mean seal level (m msl) (***Figure 3-3***). Groundwater elevations generally increase in an easterly direction, again mimicking ground surface elevations. In this eastern area, groundwater elevation data suggest that localized areas with steep hydraulic gradients are common.

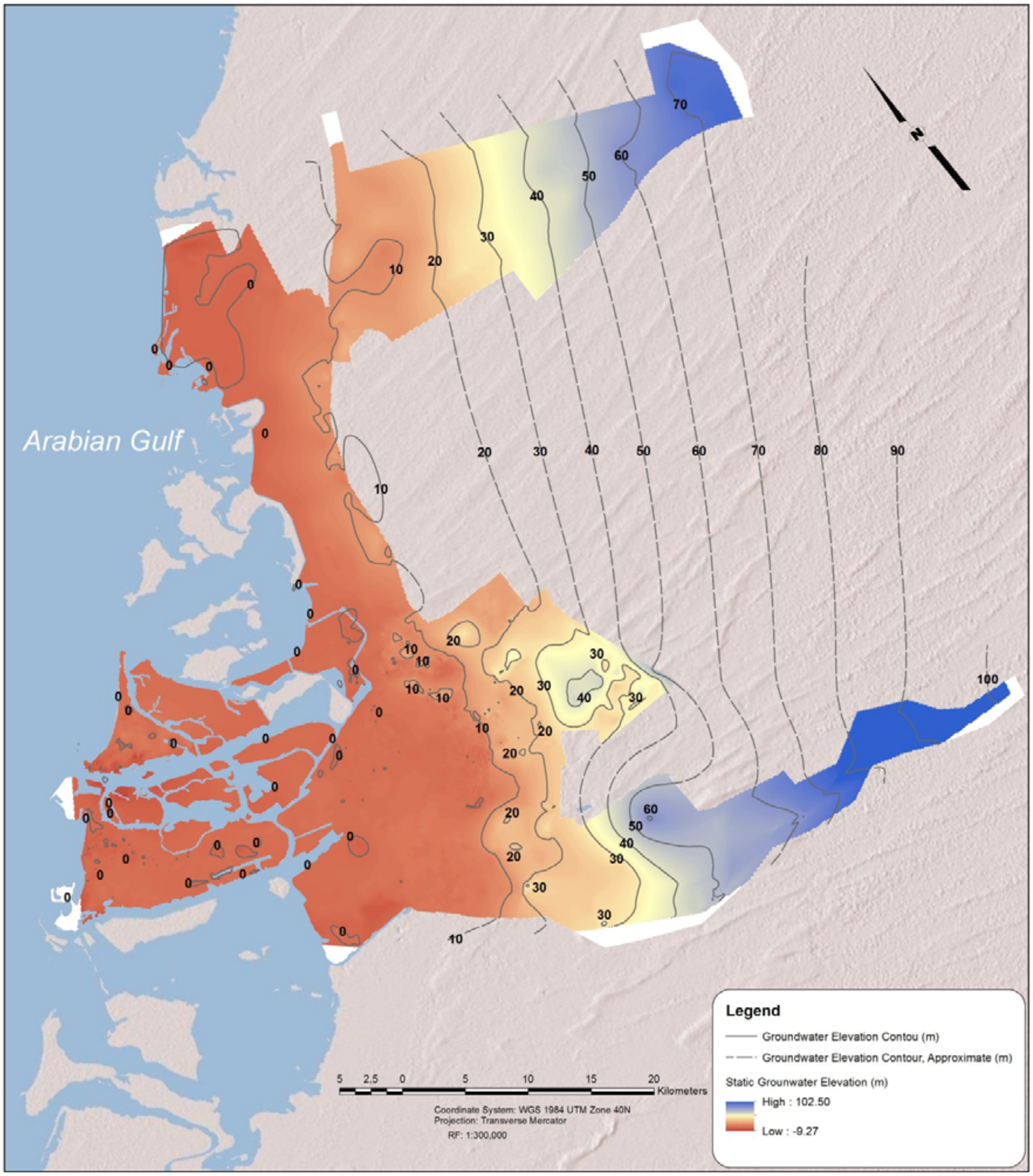

FIGURE 3-3
MEASURED GROUNDWATER ELEVATIONS IN ABU DHABI CITY MUNICIPALITY

It is important to note that top of bedrock elevations within the ADM typically range from about -15 to 100 m msl. Generally, the lowest top of bedrock surface elevations are located in western areas, but rise inland, effectively mimicking surface elevation changes. Total thickness of the unconsolidated sediments and fill material (made ground) within the ADM (i.e., Quaternary aquifer) ranges from 0 m to 18 m, although in most areas, the saturated thickness of unconsolidated sediments is typically 10 m or less (***Figure 3-4***). Greater saturated thicknesses generally occur in the south-central ADM, within the aforementioned core geotechnical hazard area. In the eastern ADM regions, the Quaternary aquifer (unconsolidated overburden) is commonly dry.

As shown on ***Figure 3-3***, the general direction of groundwater flow is from the east and southeast toward the west and northwest, toward the Arabian Gulf coastline. The shallow groundwater system is recharged primarily from the ground surface via precipitation, irrigation, and stormwater runoff detention ponds. Some groundwater flows laterally into the ADM region from upgradient areas (to the west and northwest) and vertically upward from deeper bedrock aquifers located beneath the Gachsaran Formation, which are under artesian pressure.

A portion of the groundwater flow is discharged laterally into the Arabian Gulf. However, a large portion of groundwater discharge can be attributed to evaporation to the atmosphere, especially in sabkhas and low elevation areas, where the water table surface is very close to the ground elevation (***Figure 3-5***). According to Sanford and Wood (2001), a water budget was estimated for United Arab Emirates (UAE) coastal sabkhas, based on data collected at two locations along the UAE coast. Their calculations showed annual precipitation at Abu Dhabi City is about 90 millimeters (mm)/year. Average annual recharge to the sabkha groundwater is about 45 mm/year (approximately 50 percent of precipitation. Annual pan evaporation is about 2,900 mm/year. Average evaporation from the sabkha surface is 88 mm/year (about 3 percent of pan evaporation). So, evaporation rates from sabkhas near Abu Dhabi are almost two times what the average groundwater recharge rates are. In addition, groundwater seepage flux upwards from deeper aquifers was calculated to be about 4 to 5 mm/year, and the seepage flux laterally into sabkha areas from upgradient areas is approximately 80 mm/year. Thus, the largest source of water entering sabkha groundwater is from precipitation recharge and the greatest loss of sabkha groundwater is due to evaporation. The calculations of water budget components for sabkha groundwater systems by Sanford and Wood (2001) are similar to the results found in a recent groundwater modeling study prepared for the Khalifa Port Industrial Zone, Area A (Mouchel, 2009).

According to Mouchel (2009), if fill materials are added to sabkha surfaces, the rate of evaporation from the ground surface is reduced considerably and the water table surface will start to rise over time. As much as 2 to 5 m of rise in water table is predicted by the Mouchel model if sabkha infilling is performed (Mouchel, 2009).

Approximately 1,000 different hydraulic conductivity field tests performed in the Abu Dhabi area have been collected from engineering reports and summarized in ***Table 3-4***. This Table shows that sands and gravels in the overburden, and sandstones, calcarenites, and conglomerates in the bedrock formations generally have the highest hydraulic conductivity values. Lower values were generally found in the finer-grained sediments, gypsum, claystone, and mudstones. However, there is considerable variation and overlap within and between lithologic types.

In the ADM, natural hydraulic gradients within the surficial aquifer are generally low. However, when large-scale construction dewatering programs are implemented, water levels can be drawn down considerably and thus create relatively steep hydraulic gradients, increased groundwater seepage velocities, changes in groundwater chemistry, and increased hydraulic uplift pressures. Changed hydraulic conditions at a construction site associated with groundwater dewatering can create forces that result in geotechnical problems as shown previously. These potential problems and mitigative measures are discussed in the remainder of this Document.

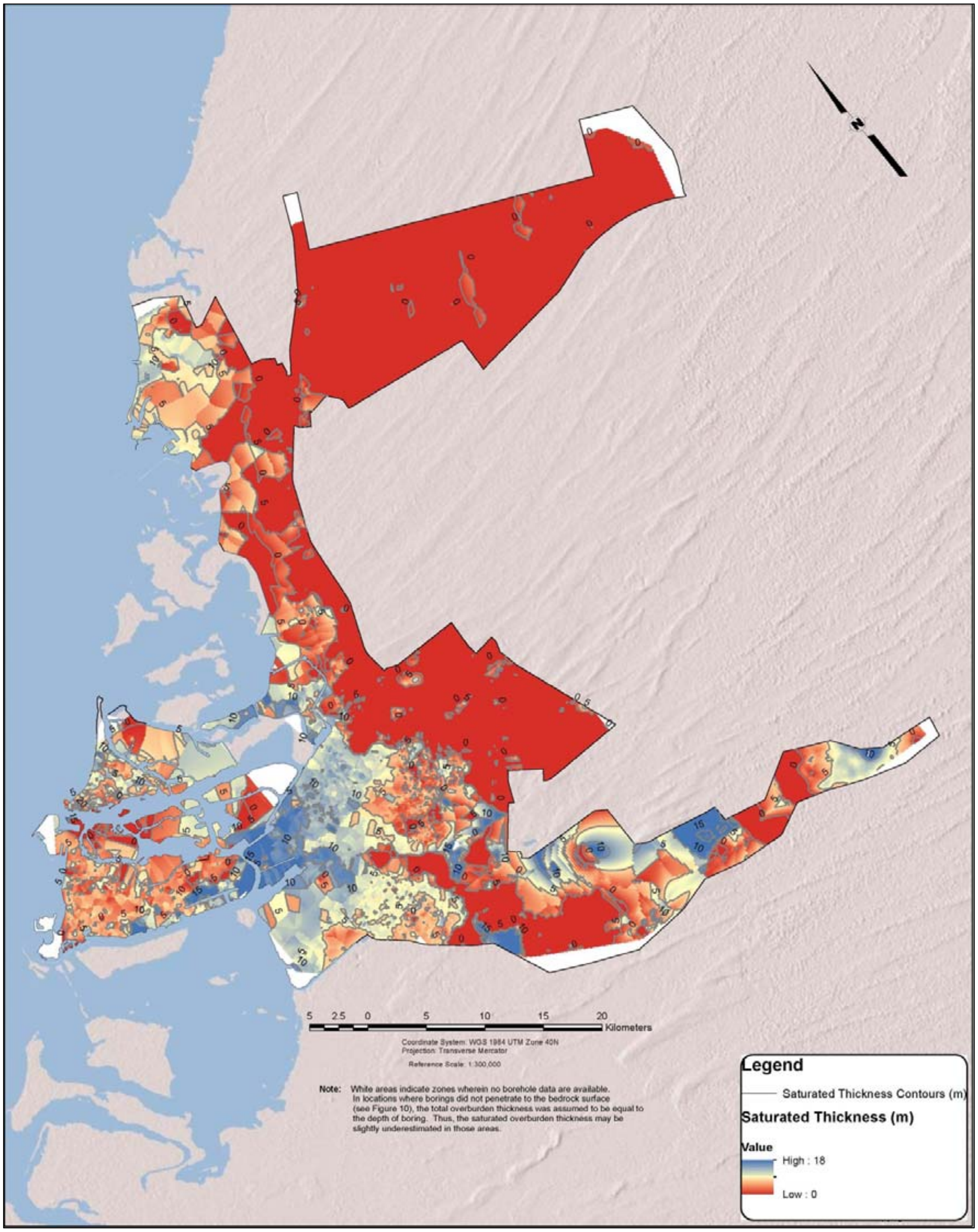

FIGURE 3-4
SATURATED THICKNESS OF UNCONSOLIDATED SEDIMENTS AND FILL

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TABLE 3-5
HYDRAULIC CONDUCTIVITY VALUES FOR VARIOUS LITHOLOGIES

LITHOLOGIC GROUP	LITHOLOGIC TYPE	HYDRAULIC CONDUCTIVITY (m/s)				
		N	MINIMUM[1]	MEDIAN	MEAN	MAXIMUM
Overburden	Sand with gravel	15	3.1E-06	1.2E-05	1.9E-05	4.0E-05
	Sand	37	6.8E-07	7.9E-06	4.3E-04	3.0E-03
	Silty Sand with gravel	52	1.7E-06	6.9E-06	1.7E-05	3.2E-04
	Silty Sand	196	0.0E+00	2.6E-06	3.3E-05	1.5E-03
	Silt and clayey silt	6	1.6E-07	8.6E-06	4.3E-05	2.2E-04
	Clay to sandy clay	17	1.4E-07	2.1E-06	5.2E-06	2.0E-05
	Clayey sand	4	2.8E-07	1.1E-06	2.3E-06	6.8E-06
	All Overburden	327	0.0E+00	3.9E-06	4.7E-05	3.0E-03
Rock	Sandstone	157	1.0E-09	2.0E-06	5.0E-06	7.0E-05
	Silty Sandstone	9	3.3E-07	3.9E-06	5.7E-06	1.7E-05
	Calcarenite	78	0.0E+00	7.1E-07	4.0E-06	2.8E-05
	Conglomerate	4	3.1E-07	1.5E-06	1.8E-06	3.8E-06
	Siltstone	133	0.0E+00	2.4E-07	4.0E-05	7.5E-04
	Gypsum	104	0.0E+00	2.0E-07	2.8E-06	5.9E-05
	Claystone	37	0.0E+00	2.3E-07	6.6E-07	1.5E-05
	Mudstone	149	0.0E+00	1.8E-07	7.9E-07	1.4E-05
	All Rock	671	0.0E+00	3.2E-07	1.0E-05	7.5E-04

Note:

[1] A minimum value of 0.00 indicates that the actual hydraulic conductivity value is less than the lowest measurable value realistically achievable by the measurement method.

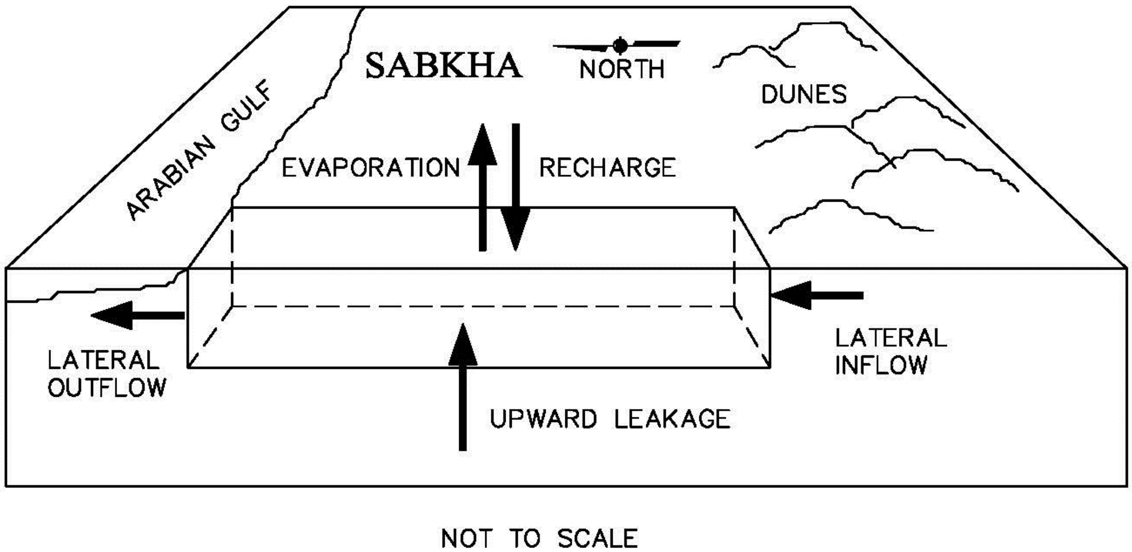

FIGURE 3-5
SCHEMATIC DIAGRAM OF GROUNDWATER BUDGET FOR SABKHA ENVIRONMENTS
(Sanford and Wood, 2001)

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4.0 METHODS OF CONSTRUCTION DEWATERING

There are four general methods of groundwater control for construction:

1 Open Pumping – Where surface drainage and gravity flow are employed as excavation progresses to feed water into a sump, from which it is removed by pumping.

2 Predrainage – Where various types of wells and subsurface drains are employed to lower the groundwater table prior to excavation.

3 Cutoffs – Where physical barriers to the flow of water, such as sheet pile walls and slurry trenches are used, often in conjunction with open pumping or predrainage.

4 Exclusion – Where compressed air, earth pressure shields, is used to exclude groundwater from the excavation. This method is primarily applicable for tunnel excavation.

The most common construction dewatering methods are open pumping and predrainage; however, it is not uncommon to see dewatering systems that combine two or more of these methods to effectively dewater a site. The following Sections describe the typical applications and suitability of these dewatering methods. Cutoffs and exclusion methods are discussed in ***Section 8.0***. ***Table 4-1*** below provides a summary of the typical suitable conditions for predrainage and general details of the various methods.

TABLE 4-1
CONDITIONS FOR SELECTION OF PREDRAINAGE METHODS
(Modified from Powers, et al., 2007)

CONDITION	WELLPOINT SYSTEMS	SUCTION WELLS	DEEP WELLS	EJECTOR SYSTEMS	HORIZONTAL DRAINS
Soil					
Silt and clayey sands	Good	Poor	Poor to Fair	Good	Good [1]
Clean Sands and Gravels	Good	Good	Good	Poor	Good
Stratified Soils	Good	Poor	Poor to Fair	Good [2]	Good
Clay or rock at subgrade	Fair to Good	Poor	Poor	Fair to Good	Good [3]

TABLE 4-1
CONDITIONS FOR SELECTION OF PREDRAINAGE METHODS
(CONTINUED)

CONDITION	WELLPOINT SYSTEMS	SUCTION WELLS	DEEP WELLS	EJECTOR SYSTEMS	HORIZONTAL DRAINS
Hydrology					
High hydraulic conductivity	Good	Good	Good	Poor	Good
Low hydraulic conductivity	Good	Poor	Poor to Fair	Good	Good
Proximate recharge	Good	Poor	Poor	Poor to Good	Good
Remote recharge	Good	Good	Good	Good	Good
Schedule					
Excavation					
Shallow (<6m below water table)	OK	OK	OK	OK	OK
Deep (>6m below water table)	Multiple stages required	Multiple stages required	OK	OK	Special equipment required
Cramped/limited access	Interference expected	Interference expected	OK	OK	May be OK
Typical System Characteristics					
Normal Spacing	1.5 – 3 m	6 – 12 m	>15 m	3 – 6 m	Continuous horizontal line
Range of Capacity – Per UNIT	0.4 – 95 lpm	190 – 2270 lpm	0.4 – 11360 lpm	0.4 – 150 lpm	Dependent on pipe size and pump capacity
Range of Capacity – Total SYSTEM	Low – 18930 lpm	7570 – 94635 lpm	Low – 227125 lpm	Low – 3785 lpm	Low – 7570 lpm
Efficiency of proper designed system	Good	Good	Fair	Poor	Good
Unit Cost Per Individual UNIT	Low to moderate	Moderate	High	Low to moderate	Low to moderate

Notes:

[1] If backfilled with sand or gravel

[2] If keyed into clay or rock

[3] Double pipe ejectors with wellscreen full length

4.1 SUMPS, DRAINS, AND OPEN PUMPING

In Abu Dhabi City, the use of open pumping is not recommended, especially in urban areas and near roads or infrastructure. While the use of these systems are discussed herein, it should be emphasized that their use must be on limited and as-needed basis in Abu Dhabi City and only in strict accordance with permit conditions as issued by ADM where no adverse effects will occur. Specifically, open pumping should be limited to dewatering operations involving cutoff structures (***Section 8.1***), where the entire area of the bottom of the excavation is in unweathered portions of sedimentary rock formations as given in ***Table 4-2***. In such cases, the role of open pumping shall be limited to extracting the relatively small amounts of water seeping through the rock bottom. In areas of weathered rock, open pumping shall not be performed as this would lead to erosion of unconsolidated materials through the mechanism of piping.

The use of drains, sumps, and open pumping is typically the least expensive dewatering option when site conditions make it feasible. In general, this method consists of establishing a series of French drain trenches graded to a sump established at the low point of the excavation, allowing seepage from the side slopes and floor of the excavation to collect in the sump and be pumped out. For staged or deep excavations, intermediate sumps may be established at various elevations as the excavation progresses. French drain trenches are typically arranged around the perimeter of the excavation to minimize the distance across the work area water must travel to be removed by the sump. Examples of dewatering through surface pumping are shown on ***Figures 4-1 through 4-4***.

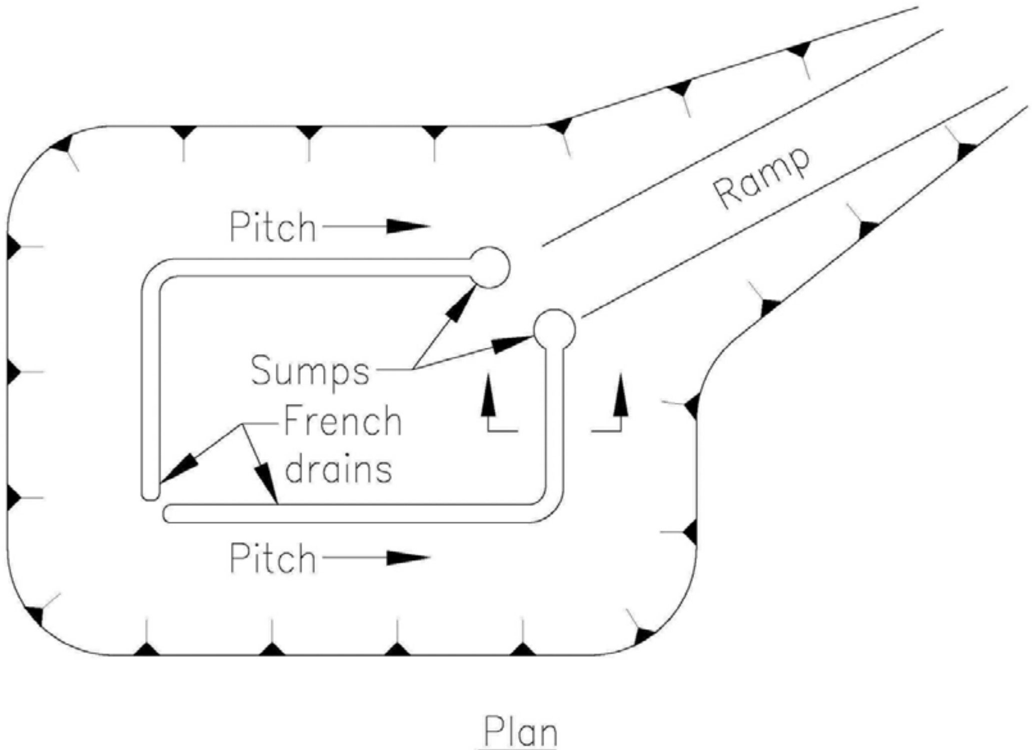


FIGURE 4-1
EXAMPLE OF PERIMETER FRENCH DRAINS
(Adapted From Powers et al., 2007)

FIGURE 4-2
DEWATERING BY SURFACE PUMPING

FIGURE 4-3
DEWATERING BY SURFACE PUMPING IN A DEEP SHORED EXCAVATION IN ABU DHABI CITY

When site conditions are not conducive to the use of open pumping and surface drainage, attempting to dewater the site with these methods is very difficult and possibly catastrophic. Surface pumping has caused several cases of damage in Abu Dhabi City, largely due to the removal of fine materials because of improper intake design and lack of filter around the intake.

FIGURE 4-4
UNCONTROLLED DEWATERING BY OPEN SUMP IN A DEEP EXCAVATION IN SHAKHBOUT CITY, 2011

At sites where open pumping appears to be feasible, it is advisable to maintain flexibility in the dewatering scheme to allow for additional dewatering measures to be enacted if changed or unexpected conditions are encountered during construction.

Table 4-2 lists site conditions that are favorable for use of open pumping dewatering systems.

TABLE 4-2
FAVORABLE CONDITIONS FOR OPEN PUMPING DEWATERING
(Modified from Powers et al., 2007)

CONDITION	DISCUSSION
Soil Characteristics:	
Sedimentary rocks	Hard strata are naturally resistant to piping erosion and base heave. As such, they are amenable to open pumping dewatering methods (e.g., unweathered Baynunah, Barzaman, and Gachsaran formations).
Hydrology Characteristics	
Low to moderate dewatering head	These characteristics all contribute to expected low groundwater seepage volumes, allowing effective use of sumps and drainage ditches (French drains).
Remote source of recharge (adjacent bodies of water, etc.)	
Low to moderate hydraulic conductivity	
Minor storage depletion	
Excavation Method	
Dragline, clamshell, and excavating equipment not operated from inside the excavation	These methods do not require traffic in the excavation, so the soft and wet material being excavated does not impede progress.
Excavation Slopes and Support	
Relatively flat slopes	Flat slopes selected appropriately for the soil type are inherently more stable than steeper slopes and more resistant to lateral seepage.
Excavation shoring structures – steel sheets, slurry walls, other cutoff structures	Excavation shoring helps to reduce or eliminate lateral seepage, increasing excavation stability and aiding open pumping systems.
Miscellaneous Other Items	
Open site with limited adjacent structures nearby	Limited slope slides may be acceptable if no adverse effects to adjacent structures will occur.
Large excavations that are slowly advanced	Very large excavations that take a long time to perform may allow drains and sumps to effectively drain the excavation.
Light foundation loads	If foundation loads on the soils at the base of the excavation are expected, disturbance of the surface by boils and seepage may be acceptable if the foundation design accounts for them.

4.1.1 Sump, Ditch, and Drain Construction

Sumps, ditches, and drains (e.g. French drains) must be designed for the expected dewatering volumes, soil conditions, and operating conditions for an individual site for open pumping to be effective. The following paragraphs outline design and construction best practices for sumps, ditches, and drains.

Ditches and drains should be sized and graded appropriately to allow for adequate flow volume to reach the sumps for removal. These calculations are performed using open channel flow calculations for open lined or unlined structures or by using Darcy's law and assumed hydraulic conductivities for ditches and drains completely filled with gravel. In addition to the dewatering water volume, the system should be designed to carry a volume of surface runoff from rainfall that is suitable for the Project. Surface runoff can often greatly exceed normal dewatering volumes, and as such may overwhelm the system

At a minimum, ditches and drains that will be in regular or permanent use for dewatering should be lined with gravel (***Figures 4-5 and 4-6)***. The gravel will limit surface erosion caused by flow through the trench and provide some filtration for water entering the trench through the sides and bottom of the trench. If additional filtration is needed, a non-woven geotextile should be placed between the soil and gravel layers. Soft or loose soils and soils that are subject to high erosion or sloughing of the sides of the trench may require the ditch or drain to be filled with gravel to maintain stability. If required, filling the structure with gravel will greatly reduce the flow capacity of the drain, so additional measures, such as increasing the size of the drain or addition of a perforated drain pipe along the base of the drain should be considered. Because sedimentation of the sump can be a major maintenance item during construction, the design of ditches and drains should be optimized for the required dewatering volume and to ensure that the water entering the sump has the least sediment load possible.

Providing adequate pumping capacity from a sump is typically the easiest part of sump design. Additional or larger submersible pumps or additional intake hoses for large diameter trash pumps are often readily available and easy to mobilize and set up. The critical aspects of sump design are then the elevation of the sump and the features affecting cleaning and maintenance of the sump.

The sump must be deep enough so that it will adequately drain the excavation to a suitable working level. This may mean that the sump bottom must be established several meters below the lowest point of the excavation for large sites. Intermediate sumps at progressively deeper elevations may be required as the excavation progresses, depending on the depth of excavation and soil conditions.

The sump must be designed to limit the amount of damaging fine particles traveling through the pump and discharge lines and should be configured so that it can be easily accessed for mucking out and pump replacement as necessary. The gravel lined ditches and drains will reduce the sediment load into the sump; however, it is necessary to provide an additional zone of gravel around the pump well that will act as a filter. Over the course of the Project, this gravel filter may need to be excavated and replaced several times as it becomes clogged with trapped sediment. In the gravel filter, a pump well should be established that is significantly larger than the pump or intake that will be used to facilitate cleaning out of the pump well and maintaining or replacing the pumps. This pump well is often constructed of a large diameter perforated steel or plastic pipe section or steel bar screens welded to form a box. The pump or intake should not rest on the bottom of the pump well, but should be hung high enough that it will not be affected by the accumulation of fine particles in the pump well. Typical dewatering sumps are shown on ***Figures 4-7 through 4-9.***

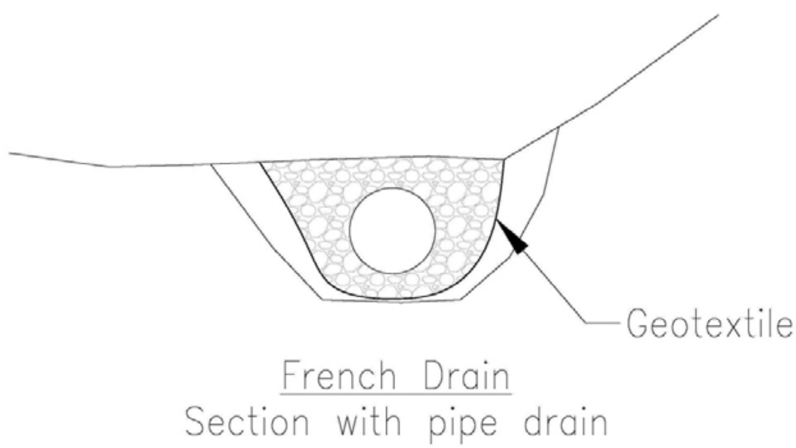


FIGURE 4-5
SECTION OF FRENCH DRAIN WITH PIPE DRAIN AND GEOTEXTILE
(Adapted From Powers et al., 2007)

FIGURE 4-6
TOP OF A FRENCH DRAIN FOR DEWATERING OF A DEEP EXCAVATION IN SHAKHBOUT CITY, 2011

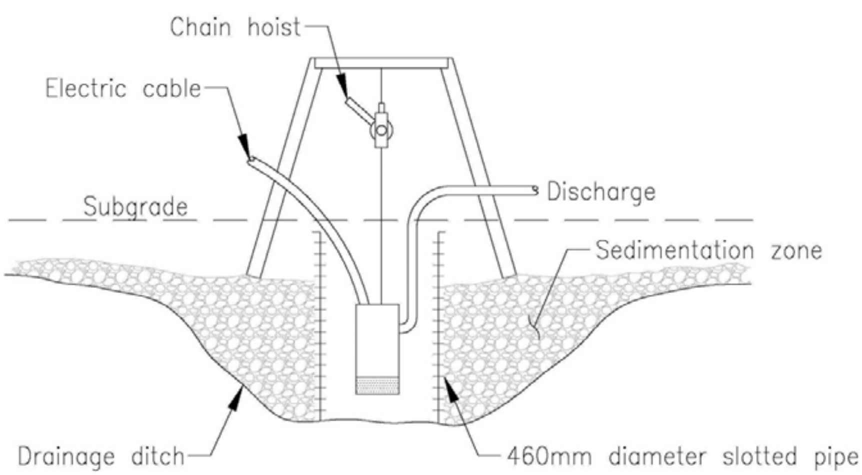


FIGURE 4-7
SUMP FOR SMALL EXCAVATIONS
(Adapted from Powers et al., 2007)

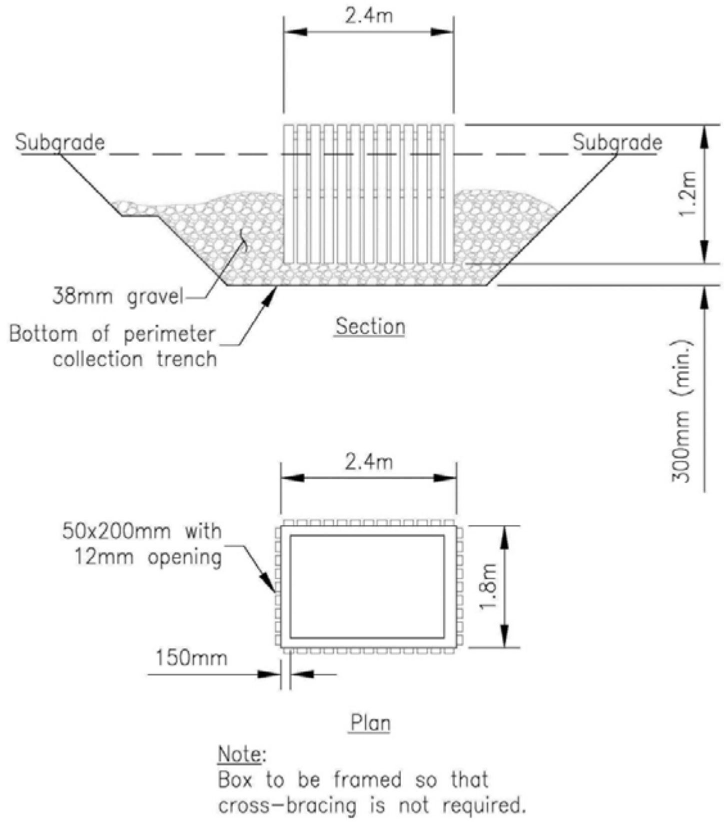


FIGURE 4-8
SUMP FOR LARGE EXCAVATIONS

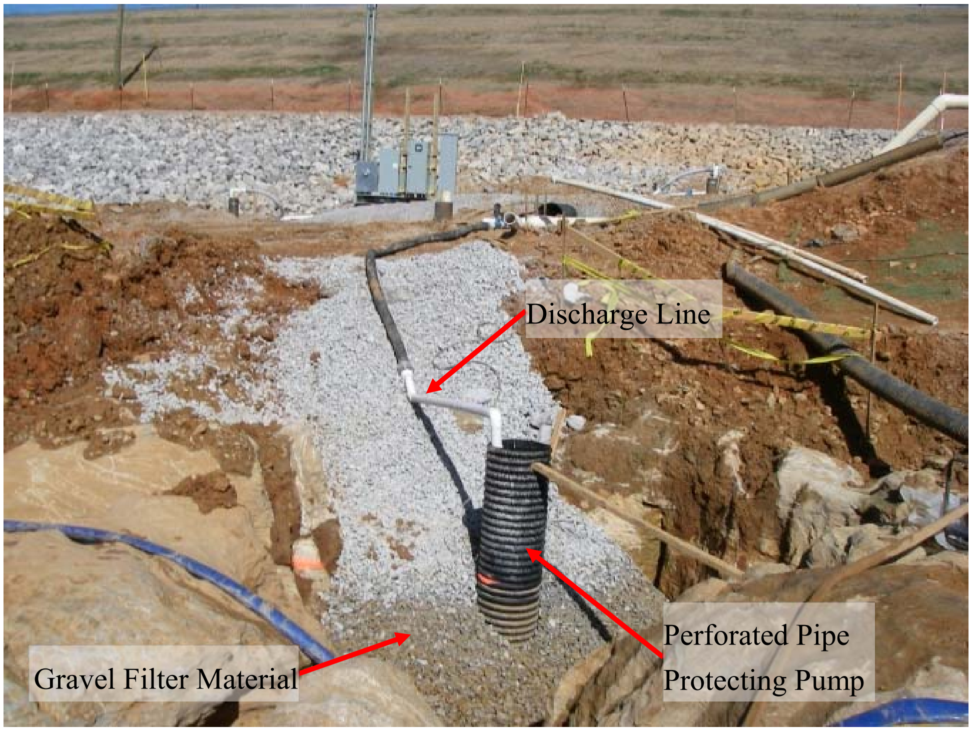


FIGURE 4-9
TYPICAL SUMP ARRANGEMENT

4.1.2 Other Construction Considerations

When water is coming through the material at the base of an excavation, a gravel bedding layer is often employed to provide drainage to the sump and allow for foundation work to be performed in the dry. This bedding layer may be underlain with a non-woven geotextile to increase filtration, if necessary. The use of a bedding layer may not be effective if the base material is cohesionless, particularly in the case of fine sand. Piping of fine particles from the subgrade materials must be prevented in order to preserve the support of the foundation materials. If gravel bedding is found to be ineffective at managing the seepage volume through the floor of the excavation, additional measures, such as wellpoints or deeper drains, and sumps are warranted.

Maintaining stable slopes when using open pumping to dewater an excavation can be a challenge. As excavation proceeds, careful observation of the slopes and drainage features must be performed to ensure that slope instability, excessive erosion, piping, boils, or heave are not occurring. In the event that these conditions cannot be mitigated, the excavation must be allowed to flood and additional dewatering measures installed. These potential hazards are discussed in additional detail in other sections of these Guidelines.

If concerns about slope stability are raised prior to excavation beginning, a possible solution is to perform an initial excavation with much flatter slopes. After this excavation is completed and the open pumping system has had time to operate and effectively lower water levels around the perimeter of the excavation, the slopes can then be recut at the final, steeper grade. In the case where slope instability due to surface or internal erosion/piping is noted after excavation, it may be possible to stabilize the slope using a sandbag berm or by placement of a gravel berm underlain by a non-woven geotextile.

4.2 PREDRAINAGE VIA WELLPOINT SYSTEMS

Wellpoint systems are the most common and often the least expensive predrainage option for construction dewatering. They are generally well suited to situations where the water table needs to be lowered to 4.5 to 6 m or less, or when the excavation is suited to the use of multiple stages of wellpoints every 4.5 to 6 m of water table elevation. The effective depth of wellpoint systems is limited due to the limitations of vacuum lift. It is important to note that a wellpoint system is only efficient to the depth of the shallowest installed wellpoint. The implication of this observation is that the depth reached by individual wellpoints should be uniform across a wellpoint system when installed in a uniform geologic layering. The unit cost for individual wellpoints is typically low and multiple wellpoints can be operated with a single pump and header system. Because of the low unit cost of wellpoints, they are well suited for conditions requiring very close spacing, such as stratified conditions and excavations, where an impermeable layer is near to the base. A wellpoint dewatering system consists of three basic items, namely the wellpoint pump, the header pipe and connections, and the individual wellpoints.

The wellpoint pump performs three tasks: it pumps air, which provides suction to the system, pumps water out of the system, and separates air and water before discharging the water. The pump must be sized so that it will be able to provide the required vacuum while handling adequate air and water volumes and must be able to develop the necessary dynamic head to push the water to the discharge point. Various pumping configurations can be used, including a single pump at one end of the header, pumps at both ends of a header, or multiple pumps spaced periodically along the length of the header. The use of a single pumping point is the most convenient; however, it requires a larger header line to limit friction to suitable levels in the system. Having multiple pumps connected to the same length of header main should be avoided as doing so would lower the system efficiency. It is standard that a well point system is 100 m in length with a single pump installed. A 100 m long system would typically have between 70 and 100 wellpoints.

Valves should be provided at necessary points along the header alignment to allow for sections of the system to be isolated during construction, in case of modification of the system or failure of part of the header due to vehicle collision or other means. Valves in the header will also allow for balancing of the system when the header is being pumped from both ends or intermediate locations. At a minimum, the header should be valved every 130 m and at critical connections, such as bends, pump connections, etc. The header and discharge lines should be braced or strapped appropriate to the operating pressures of the system and to protect from the damaging effects of water hammer. Thrust blocks may be appropriate at changes of direction.

The alignment of the header and discharge line must be carefully considered prior to starting construction. The system should be placed such that access is convenient during installation and during all stages of construction requiring dewatering. Interference with later construction activities can bring the Project to a halt while the system is reconfigured, if adequate planning is not performed early in the Project. If elements of the wellpoint system are to be left under permanent structures or slopes, they must be properly abandoned by filling them with grout to prevent future issues.

Individual wellpoints are typically small diameter (4 to 5 centimeters [cm]) pipes provided with perforated screens in the zone targeted for dewatering. Wellpoints may be installed by jetting, driving, or with the use of various drilling tools. In the ADM, pre-drilling is the standard method, and jetting may not be adequate for the local ground conditions. Wellpoints are typically provided with a sand filter zone around the screen to prevent the migration of fine particles into the system during dewatering. The filter zone also serves to increase the effective diameter of the wellpoint. Suction wells are larger diameter (up to 20 cm) wellpoints that are available for use in high yield conditions. The wellpoints are connected to the header by the swing connection, which consists of a flexible hose, tuning valve, and connection hardware, elbows, and nipples that are compatible with the header and wellpoint. The swing connection is used to deliver suction and transfer water to the header, and the tuning valve is used to adjust the flow of air and water through the wellpoint. Tuning of individual wellpoints allows the operator to prevent any one wellpoint from pulling in too much air or fine particles and to control the overall efficiency of the wellpoint system.

Selection of the location of the screened interval and filter zones of the wellpoint will significantly influence the effectiveness of the wellpoint. Ideally, the screen will be placed well below the desired elevation of the water table. When the screen is partially above the water table, the system begins to draw air from the surrounding soil as well, and can greatly reduce the effectiveness of the well or even cause an airlock in the system. In the case where dewatering is being performed to reduce the water table to the elevation of an impermeable layer, such as rock or clay, the use of shorter screens or socketing the screens into the impermeable layer may be appropriate. In many cases, extension of the filter zone along most of the length of the borehole aids in the effectiveness of the wellpoint. To mitigate the drawing of air into the wellpoint, it is a good practice to create a seal made of appropriate materials (e.g. cohesive soil arisings, bentonite, or grout) after the wellpoint and aggregates have been installed. Typically, a thickness of 50 centimeters for the seal is used. ***Figure 4-10*** shows recommended wellpoint tip depths relative to several commonly encountered subsurface conditions.

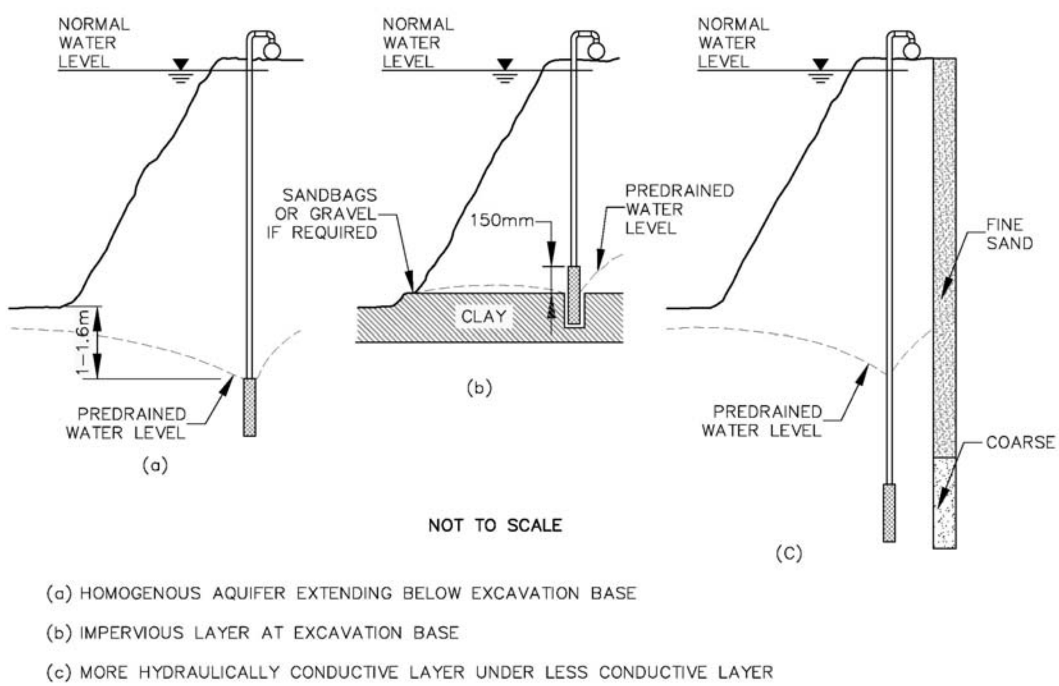


FIGURE 4-10
RECOMMENDED WELLPOINT CONFIGURATIONS FOR VARIOUS SUBSURFACE CONDITIONS
(Powers, et al., 2007)

Spacing of wellpoints can have a significant effect on the performance of the system, and must be selected based on the site soil conditions and expected flows. Typical spacings for small diameter wellpoints range from 1 to 4 m. For spacings larger than this range, suction wells or dewatering wells may be more appropriate. For relatively homogenous aquifers that extend relatively far below the base of the excavation, wellpoint spacing may be selected on the basis of the volume of water extracted. The total pumped volume is split between the numbers of wellpoints to ensure that the friction losses, caused by flow through each of the wellpoints, are maintained at acceptable levels. In some cases, this calculation may indicate that larger diameter wellpoints or wells are appropriate. Spacing in stratified or highly variable soils may need to be closer than typical in order to ensure that the various contacts are adequately intercepted. The filter zones, in this case, also provide significant additional vertical drainage capacity. When wellpoints are used to dewater the contact of a relatively impermeable layer, such as rock or clay, very close spacing may be required to collect the flow along the contact.

4.3 PREDRAINAGE VIA DEWATERING WELLS

Dewatering wells generally have larger diameter than wellpoints, typically 15 cm up to greater than a meter, and consist of a solid casing provided with a screened interval in the target zone. Dewatering wells in smaller diameters are possible with the use of recently developed very small pumping equipment. Similar to wellpoint systems, a filter zone is constructed around the screened interval to prevent the pumping of fine particles from the surrounding soil. A well pump is placed in each well casing near the screened interval, allowing for very high production from an individual well without the limitations of suction pressure. Dewatering wells are typically spaced widely and installed at a depth well below the excavation bottom. They are most effective in situations where a relatively homogenous and transmissive aquifer exists, allowing for maximized production from each well. The unit cost for dewatering wells is high, due to the depth and size of the holes required for installation and the cost of the individual pump installed in each casing. Power costs for high production well systems can also be significant. ***Figures 4-11 and 4-12*** show a typical dewatering well configuration.

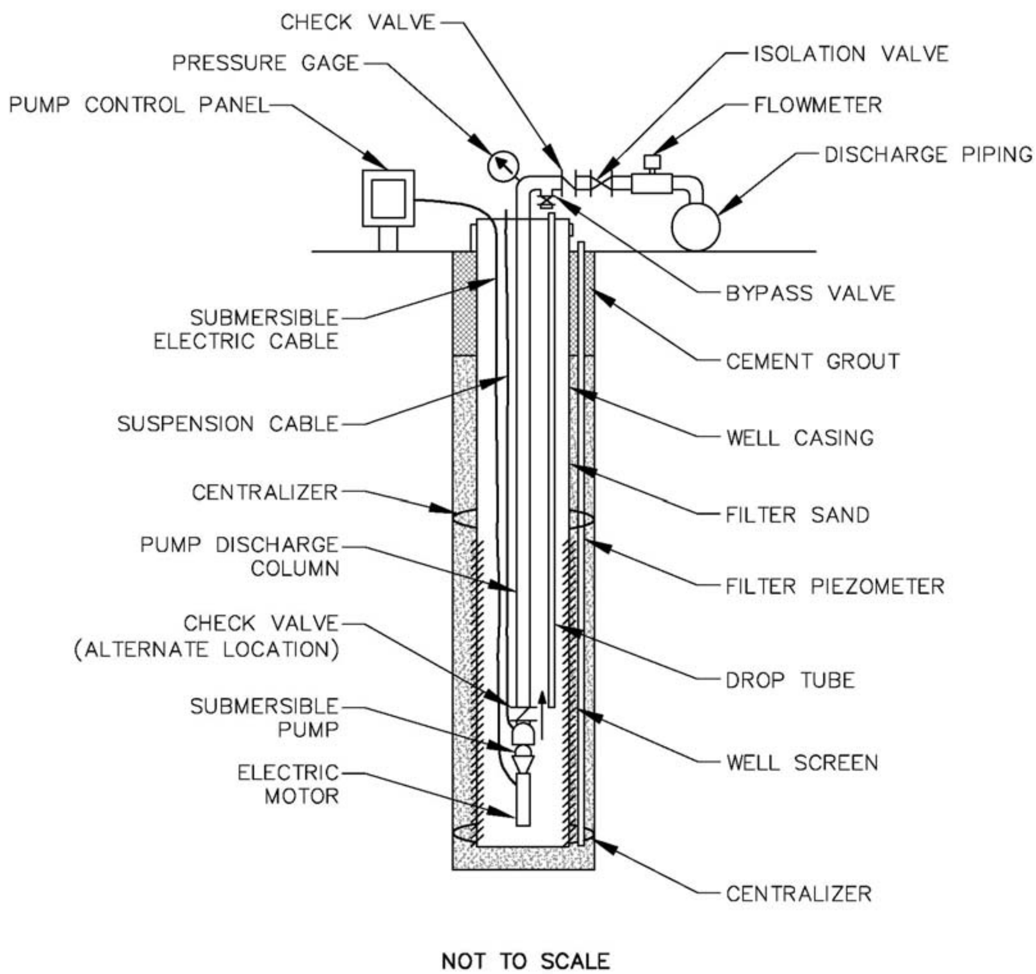


FIGURE 4-11
TYPICAL DEWATERING WELL CONFIGURATION
(Powers, et al., 2007)

FIGURE 4-12
TYPICAL DEWATERING WELL

Dewatering wells are installed using various equipment types dependent on regional preferences, geologic conditions, and safety or environmental concerns. These methods can be divided into four main groups, such as jetting, bucket Auger Drilling, rotary Drilling, and cased Drilling methods.

Table 4-3 summarizes some details of these various methods. When selecting the method for installation of dewatering wells, the ease of installation in the site's geologic setting, impacts on adjacent structures, and the effort required to effectively develop the wells should all be considered. Some methods may not be appropriate to urban settings due to ambient noise and dust control regulations, while those involving injection of high pressure fluids may adversely impact adjacent utilities, foundations, or slopes. If a method requiring the use of drilling mud to hold the hole open is used, the product selected should be one of the commercially available polymer compounds that can easily be broken down with chemical injections or that will naturally break down with time. The use of clay based muds should be avoided due to the tendency of a mud cake to build up on the borehole walls and seal off the surrounding aquifer from the well. Clay-based mud cakes are very hard to remove with normal surging/hole cleaning techniques and are likely to greatly reduce well efficiency. For the same reason, careful installation techniques and properly selected drilling methods must be used when drilling through fine grained lenses in the subsurface to prevent smearing and clogging of the borehole walls.

TABLE 4-3
GENERAL DETAILS FOR WELL INSTALLATION METHODS
(Powers, et al., 2007)

WELL INSTALLATION METHOD	DESCRIPTION OF METHOD	TYPICAL WELL DIAMETER AND MAXIMUM DEPTH
Jetting	Holepuncher on swinging or fixed leads is jetted into ground using high pressure water.	Diameters up 600 millimeters (mm), Depths up to 35 m
Bucket Auger Drilling	Bucket auger driven by Kelly bar, typically mounted on excavator or crane is used for excavation. Similar to drilled shaft excavation.	Typical diameters: 40 cm to 1 m, Depths up to 27 m
Rotary Drilling	Drill rig circulates fluid (drill mud or water) to move cuttings to surface. Typically drilled with roller or drag bits.	Diameters up to 450 mm typical, Depth dependent on capacity of drill rig, very deep depths possible.
Cased Drilling	Drilling method where casing is advanced with the drill head to maintain borehole stability. Bit is typically advanced by means of rotary percussion. Numerous applications and proprietary brand names.	Dual Rotary ("Barber"): diameters from 300 to 600 mm Duplex Percussive ("Odex", "Tubex," "Summetrix"): diameters up to 915 mm Down the hole percussive (best used in rock with casing above rock line): diameters up to 200 mm Sonic Drilling: diameters up to 305 mm Depth dependent on capabilities of drill rig, very deep depths possible.

Wellscreens and casings are available in various materials and sizes, and with a wide variety of perforations and total open areas. The minimum size of the screen and casing will be dictated by the size of the selected well pump. Screens and casings are available in polyvinyl chloride (PVC), high-density polyethylene (HDPE), and various types of metal including stainless and galvanized steel. The selection of the casing and screen material should be made in consideration of the expected loads on the pipe and depth of the well, as well as environmental considerations, such as corrosivity, prevalence of iron bacteria, and the presence of other contaminants. The apertures in wellscreens may be closely spaced parallel slots, continuous slots created by wrapping a wire around the screen cylinder, louvered slots punched into a sheet of metal before it is rolled into a cylinder, wire mesh fastened over a pipe perforated with round holes, or other options. Prepacked wellscreens are available, which include a filter installed in the screen for dewatering wells, where placement of a conventional filter zone is not feasible. The open area of commercially available wellscreens ranges from as low as 3 percent of the surface area up to 45 percent of the surface area depending on the type and material. The slot size of a wellscreen should be chosen to pass 10 percent of the fine material portion of the selected filter material and none of the coarse portion of the filter material.

The area of openings in the wellscreen should be selected to avoid excessive entrance velocity. The theoretical screen entrance velocity, V_s, has been widely used. V_s is defined as the total flow Q per unit length of screen divided by the area of openings, A_0, per unit length of the screen. In metric units, the following equation is used:

$$V_s = \frac{10Q}{A_0} \qquad \text{(Equation 4-1)}$$

where,

V_s = Theoretical screen entrance velocity in meters per minute

Q = Total flow in liters per minute per lineal meter

A_0 = Area of openings in square centimeters per lineal meter

Selection of the value of V_s should be based on the hydraulic conductivity of the surrounding filter materials in contact with the screen. ***Table 4-4*** gives values of screen entrance velocities for different values of hydraulic conductivities for filter materials.

TABLE 4-4
RECOMMENDED ENTRANCE VELOCITIES IN VARIOUS SOILS
(Powers, et al., 2007)

COEFFICIENT OF HYDRAULIC CONDUCTIVITY OF FILTER MATERIALS (m/s)	RECOMMENDED SCREEN ENTRANCE VELOCITIES (m/s)
> 67.9 X 10^{-3}	0.061
67.9 X 10^{-3}	0.056
56.6 X 10^{-3}	0.051
45.3 X 10^{-3}	0.046
34.0 X 10^{-3}	0.041
28.3 X 10^{-3}	0.036
22.6 X 10^{-3}	0.030
17.0 X 10^{-3}	0.025
11.3 X 10^{-3}	0.020
5.7 X 10^{-3}	0.015
< 5.7 X 10^{-3}	0.010

The hydraulic conductivity of the filter materials will be discussed in the filter packs discussion below. Once the hydraulic conductivity of these materials is determined, a recommended screen entrance velocity can be established from ***Table 4-4***. With the screen entrance velocity and flow rate for the well, the area of openings can be determined, and therefore the appropriate selection of the wellscreen, can be made. There are various commercially available wellscreens. ***Table 4-5*** gives examples of wellscreens available commercially in the United States.

TABLE 4-5
WELLSCREENS COMMERCIALLY AVAILABLE
(Powers, et al., 2007)

SCREEN TYPE	COMMERCIALLY AVAILABLE DIAMETER (mm)	OPENINGS (mm)	SLOT CHARACTERISTICS	ADVANTAGES	DISADVANTAGES	VARIANTS	DESIGN NOTES
Slotted PVC screen	75 - 450	0.25 - 2.5	Deep slots	Easy installation; corrosion resistant	Deep slots lead to sand clogging	Smaller sizes available for piezometers and observation wells; Schedule 80 available for higher loading (deeper wells) (typically Schedule 40)	Use somewhat lower values of V_s to obtain slightly larger open area
Continuous slot wellscreen	30 - 900	0.08 - 6	Shaped slots	Screens can be reused; several variants commercially available	Prone to corrosion depending on material; PVC screens may be subjected to excessive stresses if surge block is used in development	For wells 300 mm in diameter or larger, wellscreens are constructed of stainless steel, galvanized steel, or low-carbon steel, with other alloys available; smaller diameters available in PVC	Makes well development more effective
Bridge slot and louvered wellscreens	203 - 1220	0.75 - 4.7	Raised sections with a slot on each side	High strength for reuse	Slot dimension is not held as precisely as with continuous wire or slotted PVC screens	Available in galvanized and stainless steel for use in corrosive waters	Best suited for gravel-packed wells
Wire mesh wellscreen	Various	< 0.5	Woven wire mesh mounted on perforated pipe body	---	Not recommended for drilled wells requiring development	---	Most suitable for jetted wells, particularly for fine soils
Prepacked wellscreens	Various	Various	Available in slotted PVC and continuous wire steel or PVC	Integral filter pack held in place between two concentric screens	---	Available in slotted PVC and continuous wire steel or PVC	Used where placement of conventional filter pack is difficult, e.g., angled or horizontal borehole

The filter zone of the well must perform the following tasks:

1. It must fill the annulus between the well casing and screen to prevent borehole collapse.

2. It must retain enough of the surrounding material to prevent continuous pumping of sand or fine particles through the well. It must be permeable enough to allow mud cake and a small amount of fine particles to pass through the filter during well development.

3. It must be coarse enough to allow sufficient flow into the well during pumping.

The gradation of the filter material must be selected with consideration of the gradation of the surrounding soil and the expected yield of the well. The filter must balance the need to prevent the removal of fine particles with the need to allow efficient flow through the well, and should have a uniform gradation so that it can be placed without segregation and so that it will have a high hydraulic conductivity. The filter material should have the following characteristics:

1. They should consist of rounded silica sand. Silica sand is hard and insoluble and rounded particles promote hydraulic conductivity.

2. They should be uniform, with a uniformity coefficient $C_u < 3.0$. However, C_u of the filter should not be higher than the C_u of the surrounding material.

3. The D_{50} of the filter should be 4 to 8 times greater than the D_{50} of the surrounding material.

For uniform soils ($C_u < 3$), D_{50} of the filter pack should be in the low range (4 to 5 times D_{50}). For uniform, but more well-graded soils (C_u from 4 to 6), D_{50} of the filter pack can be between 5 to 6 times the D_{50} of the soil. For very well graded soils ($C_u > 7$), it is desirable to develop some fine particles from the soils to increase the well yield. For this purpose, D_{50} of the filter pack can be safe up to 8 times the D_{50} of the soil. Some dewatering consultants familiar with ADM dewatering practice report good results with little migration of fine particles into wells by using a 2 mm slotted PVC screen with 5 mm graded aggregate. These materials are readily available locally.

The nominal thickness of the filter pack should vary between 50 and 150 mm. In general, 75 mm is the optimum filter thickness.

A more precise method of filter selection is the Prugh Method. This method has been used effectively in situations where the pore velocities are expected to be high in fine-grained uniform soils. The D_{50} size of the filter pack should be between 4 and 5.5 times the D_{50} of the soil. The D_{15} size of the filter should be between 5 times the D_{85} of the soil and 4 times the D_{15} of the soil. The maximum value ensures continuous movement of fine particles, while the minimum value ensures free movement of water, so that the capacity of the well is maintained.

4.3.1 Well Development

After installation of the well casing, screens, and filter zone, the remainder of the annulus between the well casing and borehole is filled with additional filter material, a bentonite plug zone, and grout. The completed well must be developed before production pumping begins. Well development consists of pushing water back and forth through the screen to increase the efficiency of the well under production pumping. When properly performed, the development process reduces the amount of filter cake in the borehole wall, removes a limited amount of fine grained material from the vicinity of the wellscreen, and reorients the particles in contact with the screen in the filter zone so that they are favorable to flow through the filter into the well. The level of effort required to develop a well varies widely and is primarily dictated by the material surrounding the well. Freely draining sands may be able to be developed by cycling the well pump only, while very dense or high fine particles may require the use of an air lift or surge block to repeatedly force water through the filter zone in both directions.

An alternative well development method that has the potential for good results in the ADM area is airlifting. ***Figure 4-13*** shows an air lift system. A conductor pipe is lowered to close to the bottom of the well. An air hose is connected by a U-shaped fitting at the bottom of the conductor pipe to an interior nipple perforated with holes about 3 mm in diameter (Powers et. al., 2007). ***Table 4-6*** provides pressure and volume of air necessary as a function of lift height and submergence ratio, B/C. Submergence ratio is defined as submergence, B, over total height to discharge point, C, as defined on ***Figure 4-13*** . Below a submergence ratio of about 0.4, the system ceases to function. ***Table 4-7*** gives recommended pipe sizes for airlift pumping. A simple airlift can be constructed by installing only the air hose in the well, and using the well casing as the conductor pipe. This simplified method is recommended only for cleaning small diameter piezometers.

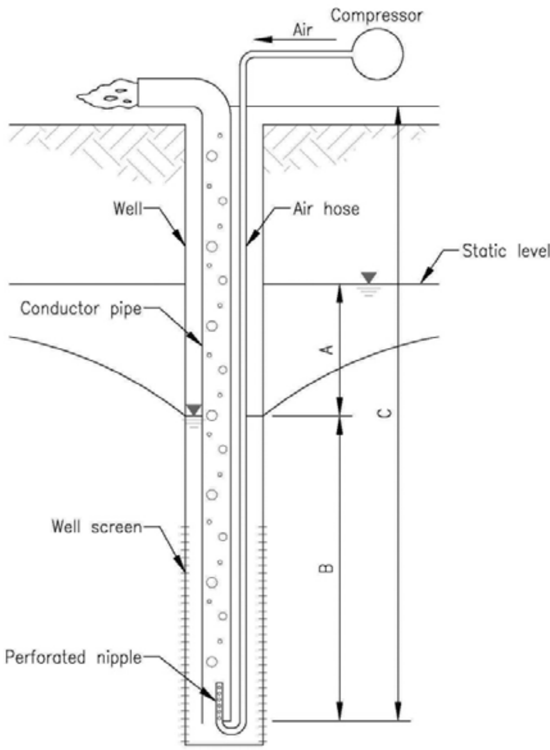


FIGURE 4-13
AIRLIFT ASSEMBLY

TABLE 4-6
PERFORMANCE OF AIRLIFT PUMPS
(Powers, et al., 2007)

LIFT C-B (m)	TOTAL DEPTH C (m)	SUBMERGENCE B (m)	SUBMERGENCE RATIO B/C	m^3 OF AIR PER LITER OF WATER	STARTING PRESSURE (kPa)
7.62	16	9	0.54	1.65E-03	89.6
7.62	24	16	0.68	9.00E-04	158.6
7.62	32	24	0.76	5.20E-04	234.4
15.2	31	16	0.51	2.99E-03	158.6
15.2	44	28	0.65	1.72E-03	275.8
15.2	55	39	0.72	1.12E-03	386.1
30.5	58	27	0.47	5.24E-03	262.0
30.5	76	46	0.60	2.77E-03	448.2
30.5	92	62	0.67	2.02E-03	606.7
45.7	80	34	0.43	7.11E-03	337.8
45.7	101	56	0.55	3.67E-03	544.7
45.7	120	75	0.62	2.77E-03	730.8

TABLE 4-7
RECOMMENDED PIPE SIZES FOR AIRLIFTS
(Powers, et al., 2007)

PUMPING RATE (l/min)	MINIMUM WELL DIAMETER (mm)	SIZE OF PUMPING PIPE (mm)	SIZE OF AIR LINE (mm)
113 – 227	101.6	50.8	12.7
227 – 303	127	76.2	25.4
303 – 378	152.4	88.9	25.4
378 – 568	152.4	101.6	31.75
568 – 946	203.2	127	38.1
946 – 1514	203.2	152.4	50.8
1514 – 2649	254	203.2	63.5

4.4 PREDRAINAGE VIA EJECTOR SYSTEMS

Ejector dewatering systems, also called eductor systems, use a jet of water through a venturi nozzle to create lift. The system is such that with each injection of water through the nozzle, a certain amount of water in addition to the injected volume goes up the riser to be discharged. While more complicated in operation, more inefficient, and higher in cost than a simple wellpoint system, ejectors are not subject to suction limitations and are highly effective in soils with low hydraulic conductivity, where the application of suction to the soil matrix will have a beneficial effect on stability of the excavation. A typical eductor installation is shown on ***Figure 4-14***.

FIGURE 4-14
TYPICAL EJECTOR INSTALLATION

Ejectors are typically supplied in two pipe or single pipe configurations as shown on ***Figure 4-15***. In a two pipe ejector, the injection water is supplied through one pipe, with the discharge through the second. In single pipe systems, the injection is accomplished through the outer area of the pipe, with discharge flowing up the center of the pipe. Two pipe ejectors are typically simpler to operate, but require a minimum hole size around two times the diameter of a single pipe ejector. The remainder of an ejector system consists of a pump, an open topped tank, and associated discharge and injection piping. The tank maintains adequate water for the injection phase of the cycle and excess water is allowed to overflow into an appropriate discharge system. Because ejectors act as self-priming pumps and will pump air, tanks are typically left open to allow the air to dissipate easily, protecting the pump from cavitation. The ability of eductors to pump air can be utilized to apply suction to fine grained materials, which can considerably increase excavation stability even if overall dewatering volumes are low.

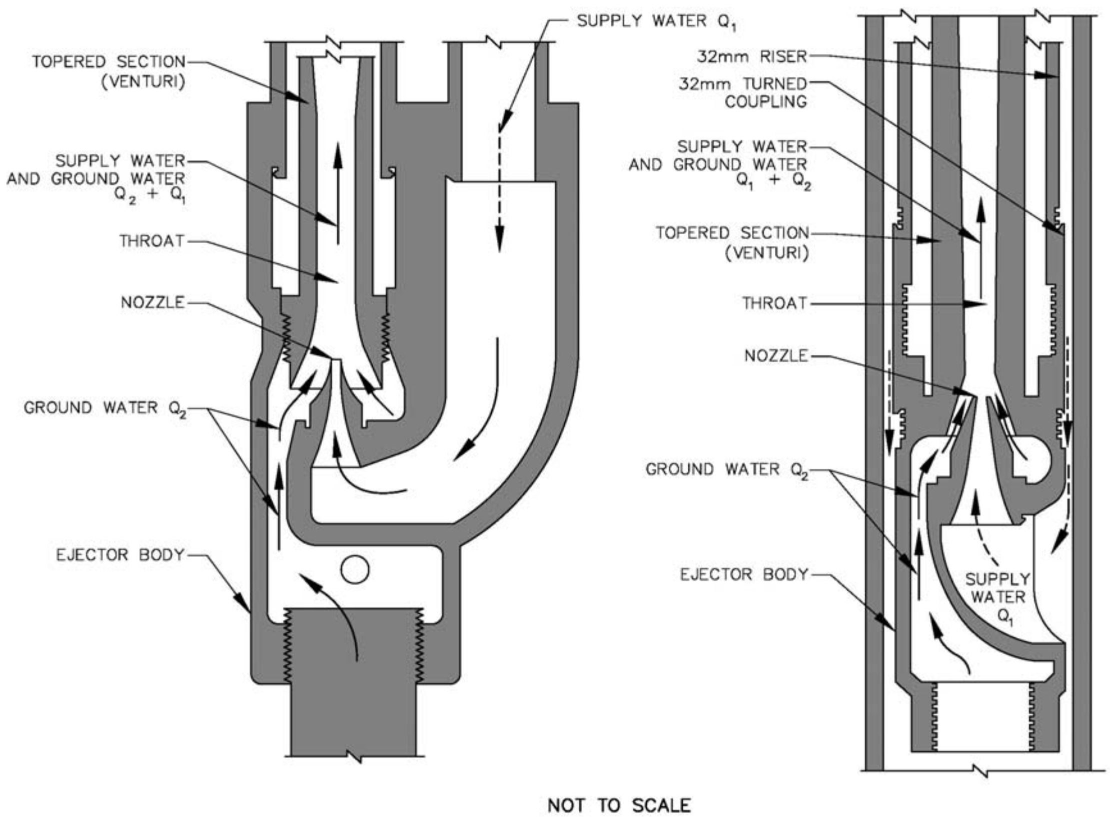


FIGURE 4-15
TWO PIPE AND SINGLE PIPE EJECTORS
(Powers, et al., 2007)

Ejector system installation is very similar to installation of wellpoint systems. Ejectors are typically installed in jetted or mud rotary drilled holes with similar requirements for filter zones, screens, and casings. The use of sonic drilling techniques is very common around sensitive structures, such as embankments or buried utilities. Ejector systems are particularly susceptible to damage caused by pumping sand, so care must be taken to provide clean installations. The system efficiency is also highly impacted by friction losses in the headers, risers, and swing connections and often high pressures are required in the supply side of the system to overcome the losses.

Ejectors, wellpoints, and dewatering wells are often used in combination to take advantage of the respective strengths of each predrainage method. ***Figure 4-16*** depicts a common dewatering arrangement where eductors, deep wells, and wellpoints are used together to achieve the required drawdown for an excavation. In this case, a deep excavation was required for the toe of an existing dam made up of a mixture of slowly draining fill materials and relatively faster draining alluvial materials underlain by rock.

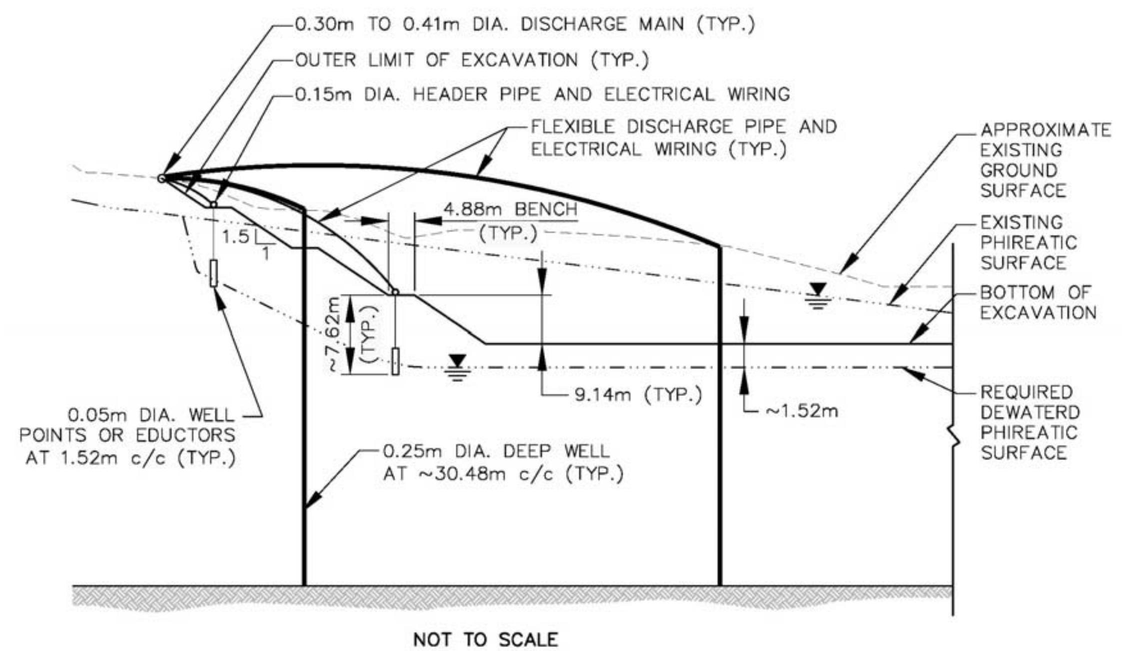


FIGURE 4-16
EXCAVATION DEWATERING USING EJECTORS, WELLPOINTS, AND DEEP WELLS

5.0 POTENTIAL DEWATERING IMPACTS

This Section describes potential impacts of dewatering, including the potential impacts on soil and rock and on nearby structures.

5.1 CHANGES IN EFFECTIVE STRESS

The effective stress is defined as the portion of the total stress in a soil mass carried by the soil solids at their points of contact (Das, 2002). Reductions in groundwater level will reduce pore pressures; the portion of the total stress carried by water in the soil void spaces, and increases the effective stress in the soil. The effective stress is an important parameter for a soil mass because changes in effective stress can affect the shear strength of granular soils and can cause consolidation, especially in compressible or weak soils.

Change in effective stress is particularly problematic when dewatering a highly permeable layer under a compressible layer. The highly permeable layer dewaters rapidly, allowing free drainage of the overlying compressible layer. When the compressible layer loses water, and therefore porewater pressure, it loses the support of the water-filled voids and easily compresses under its own weight. Situations susceptible to this failure include sand overlying a gravel lens or soft cohesive layers overlying a uniform sand layer.

In Abu Dhabi City, rapid drawdown of the water table in areas where Sabkha or silty sands overlie fractured bedrock poses the risk of creating a downward gradient that could drive the migration of fine materials from the overlying soils into the fractures in the bedrock. A strong indicator of the potential of this mechanism is the reported circulation water loss in the rock during drilling activities.

Changes in effective stress can also occur due to changes in groundwater chemistry. In some parts of ADM, especially Mohammed Bin Zayed and Shakhbout City, it is not unusual for total dissolved solids (TDS) in natural pre-development groundwater to exceed 20 percent (over 200,000 ppm), in which case, the density of water is well above 1.0 gram per cubic centimeter (g/cm^3). Changes in the chemistry due to urbanization and dewatering may occur with potential drop in TDS, and consequently lower water density and subsequent increase in effective stress. ***Table 5-1*** shows the relationship between salinity and density of water as a function of temperature.

TABLE 5-1
DENSITIES OF VAPOR-SATURATED NACL SOLUTIONS, G/CM^3

Temperature (°C)	Weight Percent												
	1	3	5	7	9	11	13	15	17	19	21	23	25
0	1.00755	1.02283	1.03814	1.05354	1.06908	1.08476	1.10060	1.11660	1.13276	1.14906	1.16551	1.18210	1.19880
25	1.00411	1.01823	1.03247	1.04688	1.06146	1.07624	1.09122	1.10639	1.12176	1.13732	1.15307	1.16900	1.18509
50	0.9948	1.0085	1.0222	1.0362	1.0503	1.0647	1.0793	1.0942	1.1093	1.1247	1.1403	1.1561	1.1722

Notes:

The uncertainties in the densities are: 5-place figures ±10^{-5}, 50°C data ±10^{-4}

Adapted from Potter and Brown (1977)

5.2 CHANGES IN SEEPAGE VELOCITIES AND PRESSURES

Dewatering through surface pumping or underground systems can cause changes in seepage velocities and porewater pressures. This can lead to transport of soil particles, or piping, as described in ***Section 5.3***, heave or boiling, which is defined below, or other negative impacts.

In cases where water seeps upward through soil, the upward seepage forces a reduction of the effective vertical stress in the soil mass. The upward flow of water produces a frictional drag force or seepage pressure that tends to lift the soil grains. When the vertical gradient approaches the critical gradient in granular soils, the effective stress in the soil is reduced to zero and the soil loses its strength. This condition can be referred to as boiling or a “quick condition.” When the upward seepage forces exceed the resisting force of the soil and the ground becomes unstable, it can also be referred to as base heave. As an example, heave could occur on the inside of an excavation with bracing when water is being pumped out of the excavation as shown on ***Figure 5-1*** below.

The risk of base heave is of special concern in layering configurations in which a confining layer is in between aquifers. The lower, confined aquifer can exert high pressures on the bottom of the thinned layer after excavation. For such conditions, it is recommended to depressurize the lower, confined aquifer prior to and during excavation works.

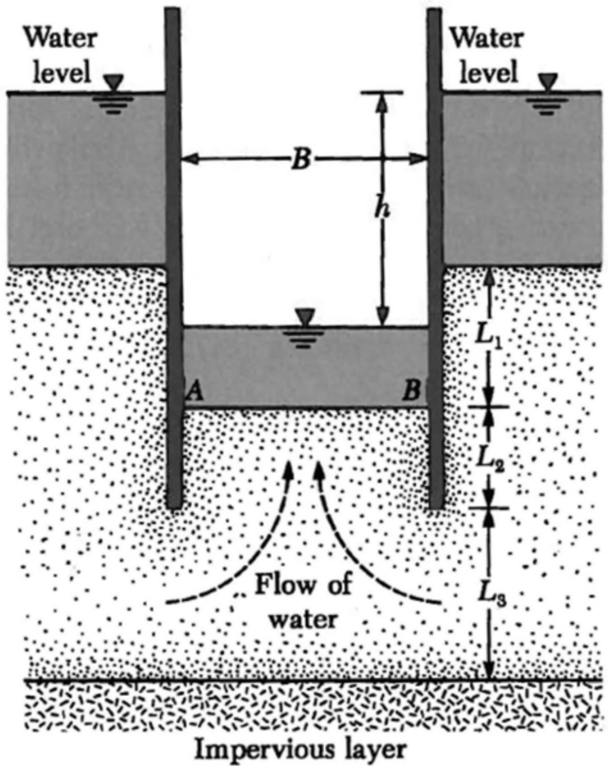


FIGURE 5-1
POSSIBLE CONDITION FOR HEAVE
(Das, 1995)

5.3 EROSION OF FINE PARTICLES AND PIPING

The hydraulic gradient is defined as the head loss over a distance when groundwater is flowing through a soil. When the hydraulic gradient exceeds the critical gradient, transport of materials can occur. This is generally referred to as piping. Piping starts with transport of the finer particles in the soil, and can progress gradually as fine particles are removed from the soil until complete failure occurs.

The erosion of the fine fraction of granular soils due to seepage is a possible cause of the settlements of shallow foundations. Estimations of the quantity of eroded material in the vicinity of pumping wells can be made with the use of numerical models, and settlements in nearby buildings can be inferred (Cividini et al, 2009).

Fine particles can also be transported out of a soil mass when components of dewatering systems, such as screens or filters, are improperly designed.

In ADM, the current practice to avoid removal of fines is to monitor the rate of water table drawdown, rather than pumping rates.

Skilled dewatering contractors in ADM suggest fine-tuning the drawdown rate to allow sands to compact naturally. An excessively high rate of drawdown has the potential to cause rapid washout of fine materials such as silts, leading to a sudden collapse of the sands. An excessively slow rate of drawdown would require a more time consuming operation that would give compressible layers more time to deform. An optimal rate of drawdown of 0.5 m/day has been found by closely monitoring the outflow waters for turbidity. Turbidity is not allowed to exceed 50 Nephelometric Turbidity Units (NTU). Reference standards for 5, 55, and 515 NTU are shown on ***Figure 5-2***. Monitoring of the rate of drawdown during dewatering should be made in piezometers installed halfway between dewatering wells, as specified in ***Table 12-2***. This point corresponds to the crest of the cone of depression between wells.

Monitoring of turbidity and water levels may be performed by using real time, remote data access systems, which are available in the monitoring device industry. Data can be retrieved and analyzed at any moment, which facilitates monitoring of drawdown rates and turbidity levels. These systems also offer the possibility of automatically increasing data logging during a specific event, offering flexibility for data acquisition depending on site-specific conditions during dewatering operations. It is highly recommended that automatic systems be employed in challenging dewatering projects.

In addition to water levels and turbidity, other parameters that can be monitored are the pH, temperature, Total Suspended Solids (TSS), TDS, and electrical conductivity. Data loggers are connected to the telemetry system, which is connected to a GPS network that sends instant results. Data loggers are installed inside the pumping well or observation well.

In addition to telemetry systems, there are also handheld devices. The main disadvantages with handheld devices are: the data needs to be registered manually, and if the sample collected is not treated in a hygienic way, it will give inaccurate figures.

The recommended spacing of monitoring wells for Abu Dhabi City is as given in ***Table 12-2***.

FIGURE 5-2
TURBIDITY IN NEPHELOMETRIC TURBIDITY UNITS (NTU)
(USGS, 2014)

5.4 COLLAPSE OF EXISTING CAVITIES

In soil and rock masses with existing cavities, which are common in the ADM, an increase in effective stresses through dewatering can cause increases in stress at and around existing cavities in the soil or rock, which can lead to the collapse of the cavities. Changes in groundwater levels may also cause soil raveling into underlying cavities, leading to development of surface sinkholes. This is discussed in detail in ***Section 9.3***. ***Figure 5-3*** depicts cases where a negative impact may result from the interaction between existing cavities and dewatering operations adjacent to sensitive structures.

5.5 DIFFERENTIAL SETTLEMENT

Dewatering for construction purposes has occasionally resulted in settlement of the surrounding area, sometimes with damage to existing structures (Powers, 2007).

The depression cone created by dewatering operations has the collateral effect of generating non-uniform changes in the effective stress acting on the soil, with the greatest effective stress increase in the proximity of the dewatering well / wellpoint and diminishing outward. Structures located within the radius of influence of a dewatering system may, therefore, suffer the effects of differential settlements, if proper measures are not taken. The possibility of such settlements should be investigated before a dewatering system is designed. Establishing reference hubs on adjacent structures prior to the start of dewatering operations will permit measuring any settlement that occurs during dewatering, and provides a warning of possible distress or failure of a structure that might be affected. Recharge of the groundwater, as illustrated on ***Figure 5-3***, may be necessary to reduce or eliminate distress to adjacent structures. Alternatively, it may be necessary to use cutoff structures to avoid lowering the groundwater level outside of an excavation. These structures are discussed in ***Section 8-1***.

Settlement can be caused by dewatering mainly through the following factors:

1. Consolidation of soils, especially loose fine grained soils, due to an increase in effective stress (***Section 5.1***).

2. Transportation of soil particles, which can be due to critical gradients resulting in piping or heave (***Section 5.2***) or improperly designed or constructed dewatering devices (***Section 5.3***).

Settlement Calculations performed for different types of projects are presented in the Appendices as specific examples**.** The settlement calculations presented in the examples are absolute settlements corresponding to the zones of largest water table level drop due to excavation. For challenging projects, where both a deep excavation and close proximity of sensitive structures, use of numerical models is highly recommended. Of special importance is the estimation of the shape of the cone of depression, which will enable the calculation of differential incremental effective stresses in the proximity of the excavation area.

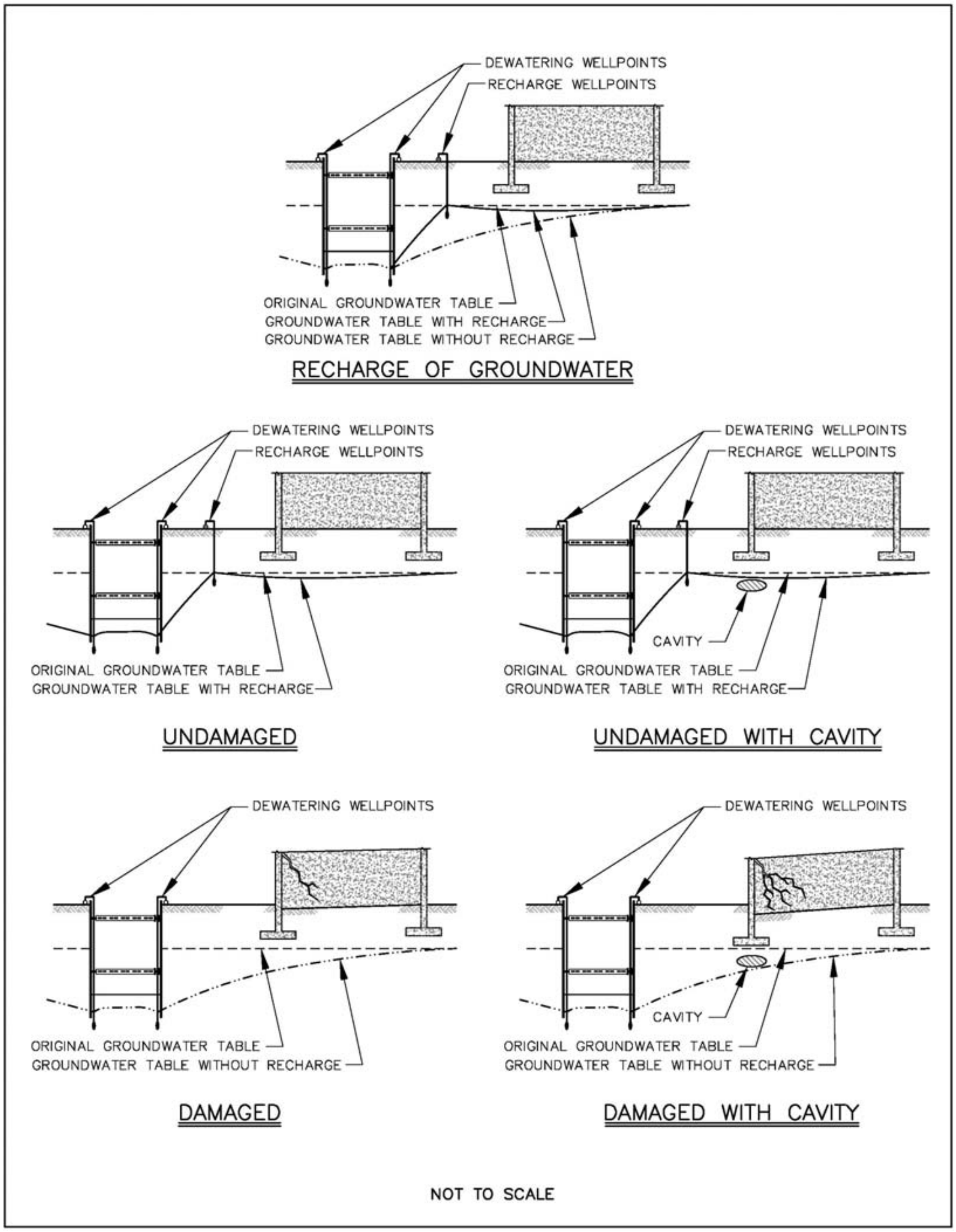


FIGURE 5-3
RECHARGE OF GROUNDWATER TO PREVENT DIFFERENTIAL SETTLEMENT AND COLLAPSE
(Top Figure Adapted from Army, Navy, and Air Force, 1983)

5.6 DISSOLUTION OF SOIL-CEMENTING SALTS AND EVAPORITIC ROCKS

In addition to the fine sediment piping discussed in ***Section 5.3***, dewatering-induced increases in groundwater flow velocities can lead to increased leaching of cementing salts in a soil mass, and thereby an overall reduction in soil strength, as demonstrated by field and laboratory testing (Al-Sanad and Al-Bader, 1990; Al-Sanad et al., 1990; Ismael, 1993). Soil strength reductions and eventual soil settlement or collapse owing to this “chemical piping” effect has generally been linked to soil void ratio increases (see, for example, Karakouzian et al., 1996). However, dewatering-induced subsidence has also been attributed to inter-particle salt bond dissolution and the subsequent collapse of granular soil structure (i.e., hydrocollapse) (Gutiérrez and Cooper, 2002; Gutiérrez, 2014).

Particularly serious dewatering-induced chemical effects can also occur in carbonate rocks and evaporites such as limestone, halite, and gypsum, where localized groundwater flow increases and subsequent chemical reaction rate increases can potentially lead to the formation of solution-widened joints and/or fissures (Fookes et al., 1985; Cooper, 1988). More localized dissolution can even result in significant cavity development. Cavities formed by dissolution can eventually collapse, triggering catastrophic surface failures, or, less dramatically, subsidence in the overlying ground surface. In either case, surface subsidence may be accentuated by mechanical piping (***Section 5.3***).

Dewatering-induced leaching/dissolution is generally confined to the salt-rich soils or soluble strata from which groundwater is being drawn, and is frequently enhanced at or near well or wellpoint locations (or at the location of other groundwater control structures). Nonetheless, it should be noted that dewatering can induce chemical piping over a large area, particularly along regional-scale preferential groundwater flowpaths (i.e., along major joints or fractures). In such cases, wider-scale surface settlement or subsidence could result from an overall thinning of area carbonate or evaporite rock masses (or layers) by dissolution.

It is also important to note that groundwater in the ADM generally maintains elevated calcium, sodium, magnesium, sulfate, and chloride concentrations, but is not necessarily saturated with respect to these ions. Accordingly, the solubility of calcite, gypsum, or any other salts at a given dewatering location can be strongly affected by the ionic strength of the pumped (flowing) water, and by the conditions (temperature, etc.) under which dissolution make take place. Moreover, other factors including water leakage from service pipes, drains, and sewers, and/or over-irrigation may influence groundwater flows and thus confound overall leaching/dissolution patterns of gypsum and/or salt dissolution.

In general, gypsum dissolves about one hundred times faster than limestone and about one thousand times slower than salt (halite) (Cooper and Calow, 1998). Under natural conditions, adjacent to a river or in a cave, gypsum can dissolve at a rate of 1 m per year with a water flow rate of about 1 meter per second across the rockface (Cooper and Calow, 1998).

Dewatering operations generate changes in the conditions of groundwater flow, especially in fractured or fragmented portions of the rock. In Abu Dhabi City, this is a concern with gypsum layers. Even though seepage velocities through intact rock are in general expected to be low as to preclude rapid dissolution of gypsum, fractured or fragmented portions of gypsum could have the potential to create preferential paths for water to flow at seepage velocities substantially larger than the average intact rock seepage velocity. This is of special concern in dewatering operations that require cutoff structures (***Section 8-1***) to be socketed in gypsum.

It is impractical to preclude socketing of cutoff structures into gypsum rock. In situations where cutoff walls need to be socketed in gypsum, the key to effective dewatering design and operation is to ensure close monitoring of the flow rates, as well as the chemical composition of the discharge water. Dissolution of gypsum can be mitigated by reducing the seepage velocity (by pumping very slowly) to avoid cavity and piping phenomena. Monitoring devices are available in the industry, which can record electrical conductivity (***Section 5.3***). If the electrical conductivity of discharge waters remains constant during the operation, it can be inferred that the gypsum remains intact.

6.0 ADDITIONAL RISKS RELATED TO GROUNDWATER AND DEWATERING

Dewatering, or lack thereof, can have other important impacts on excavations, personnel, and nearby structures as discussed in this Section.

6.1 SLOPE FAILURE IN EXCAVATIONS

Groundwater is often a key factor in the failure of slopes in soil or rock. When groundwater levels increase, pore pressures also increase, which generally decreases the stability of natural or excavated slopes. If groundwater levels are not considered in the analysis of natural or excavated slopes or groundwater levels increase above design values, this can lead to failure of the slope. Increasing the effective stress along with potential slip surfaces in slopes, by reducing pore pressures through dewatering, is a common method of improving the stability of slopes. Cracking in a slope, indicating slope instability is shown on ***Figure 6-1***.

FIGURE 6-1
CRACKING DUE TO SLOPE INSTABILITY

6.2 SHORING COLLAPSE

Shoring for excavations should be designed for the highest groundwater levels that could be experienced during the design life of the Project. When groundwater levels adjacent to shoring or retaining walls are high, the water pressures acting on the shoring can be the majority of the horizontal force experienced by the shoring structure. Therefore, increases in groundwater levels due to events, such as rainfall, changes in conditions due to construction, shutdown or failure of dewatering measures, or other factors, can lead to forces exceeding the strength of shoring and subsequent collapse. If shoring is designed with drains to reduce water pressures, such as a temporary or permanent retaining wall with weepholes, the drains can become plugged over time and lead to increases in groundwater levels and failure of the shoring or wall. The retaining/shoring structure should be designed considering the dewatering application in the Project site. Refer to design standards and manuals such as given in ***Section 8.1*** for proper consideration of groundwater related loads.

6.3 FLOODING DUE TO EQUIPMENT FAILURE

When active measures, such as pumping, are used to reduce groundwater levels, the dewatering measures can become ineffective through temporary or long term shutdown of the equipment. The shutdown can be due to intentional measures, such as maintenance or unintentional factors like loss of power or failure of a generator or pump. Dewatering systems can also become less effective with time, due to clogging of screens or other factors.

Although it is not a direct equipment failure, dewatering systems can also be overwhelmed by increases in groundwater levels or inflows beyond the design basis due to rainfall, pipe breaks, flooding, or other factors. As discussed in ***Section 4.0***, flows from rainfall and runoff need to be considered when designing dewatering systems. When dewatering systems are overwhelmed or shutdown, groundwater or surface water levels can rapidly increase and flood excavations or underground structures.

6.4 ELECTRICAL SHOCK HAZARD

Electrical equipment is often required for dewatering projects, and because most dewatering projects are temporary, the electrical codes for permanent construction may not fully apply for temporary installations. The electrical design and requirements for dewatering systems are outside the scope of this Document, and a trained and qualified individual should always be involved in the design and construction of the electrical components of a dewatering project. All electrical equipment should be effectively grounded; inadequate grounding presents a serious hazard.

7.0 DISPOSAL OF PRODUCED GROUNDWATER

This Section provides guidance on disposal of water that is produced during dewatering operations in Abu Dhabi City. Depending on the quality of the water, on-site retention and/or treatment may be required before transporting the water offsite, discharging water to the stormwater sewer system, or discharging water to adjacent land or bodies of water.

7.1 POTENTIAL IMPACTS OF WATER DISPOSAL

Water produced from dewatering operations may contain pollutants that, if discharged to a storm drainage system or natural water course, would adversely impact the water quality of the receiving water. In these Guidelines, pollutants are classified into two groups:

1. Sediment: Sediment is the most common pollutant associated with dewatering operations. Additional information is provided in ***Section 7.2***.
2. Other pollutants**:** This includes all other pollutants, which may be considered harmful to the surface and ground water system. These pollutants tend to be site-specific and are often associated with current or past use of the construction site or adjacent land.

The quality of the water that may be produced from dewatering operations may determine which options for dewatering are feasible for the site. For example, a groundwater cutoff may be required at sites with contaminated groundwater to reduce or eliminate the need for discharge of water during construction. The following parameters, at a minimum, should be evaluated as part of the dewatering system design:

- Origin of water; ground water/cofferdam/accumulated precipitation.
- Chemical characteristics of the pumped water; investigate and review the subsurface water condition and assess any reason to suspect that the pumped water could be polluted by something other than sediment.
- Total quantity of water and proposed discharge rates.
- Expected duration of the dewatering.
- Total estimated discharge.

The opportunity to reuse the pumped ground water may be evaluated. Reusing of water produced by dewatering activities may reduce the need for imported water and reduce the external impact of the Project when compared to direct discharge to the environment. On-site reuse may include applications, such as dust suppression, earthwork, compaction, vegetation establishment/reutilization, and plant/vehicle washing.

7.2 ON-SITE WATER RETENTION

Accumulated water may be retained on-site in a temporary retention pond or in a tank for evaporation, infiltration into the soil, or for other applications, such as dust control, or other construction-related activities. In general, the following guidelines for on-site water retention should be followed:

- Retention ponds should be located so that the infiltration is not recharging the dewatered area or a liner should be provided to prevent infiltration.
- The water should be free of pollutants other than sediment. Guidelines for sizing a sediment retention pond (Desilting Basin) are provided below.
- Retained water should not be reused near inlets or other areas, where it may be inadvertently discharged from the site.

7.2.1 Sediment Treatment

If water produced from dewatering operations is being retained on-site (for infiltration, evaporation, dust control, etc.), sediment treatment may not be required. However, if effluent is being discharged, treatment may be required. There are a variety of sediment treatment technologies available, such as Desilting Basins, Sediment Traps, Weir Tanks, Dewatering Tanks, Gravity Bag Filters, Pressurized Bag Filters, and Cartridge Filters.

A desilting basin is a temporary basin with a controlled release structure formed by excavation and/or construction of an embankment to detain sediment-laden runoff and allow sediment to settle out before discharging. The required desilting basin size is based on the dewatering discharges. ***Table 7-1*** provides general guidance for sizing a basin for a range of discharge flow rates. The calculations used to determine the required surface area are based on a given target particle size to be removed (with an associated settling velocity). Certain design criteria were assumed: 0.015 mm target particle size, a continuous flow rate through the basin, assuming flow in equals to flow out, depth between 1 to 1.5 m and full storage capacity (including a 65 m^3 sediment storage zone).

TABLE 7-1
REQUIRED SIZE OF DESILTING BASIN
(Caltrans, 2001)

FLOW RATE (m^3/s)	REQUIRED SURFACE AREA (m^2)	LENGTH/WIDTH = 2:1	
		LENGTH (m)	WIDTH (m)
0.0016	12.25	4.95	2.48
0.0032	24.51	7.00	3.50
0.0063	49.01	9.90	4.95
0.0095	73.52	12.13	6.06
0.0126	98.03	14.00	7.00
0.0158	122.53	15.65	7.83
0.0189	147.04	17.15	8.57
0.0221	171.54	18.52	9.26
0.0252	196.05	19.80	9.90
0.0284	220.56	21.00	10.50
0.0315	245.06	22.14	11.07

7.3 DISCHARGE TO STORMWATER SYSTEMS

If water produced through dewatering is to be discharged to the stormwater network, a permit is required as described in ***Section 7.5***. Form No. EM-7.2 is required to request permission to discharge groundwater to the stormwater system, and the Infrastructure and Municipal Assets Sector is responsible for issuing permits to discharge water.

The Internal Road Division and Infrastructure (IRI) of ADM have specific requirements regarding the design of the settlement tanks and their size. Requestors of discharge permission must contact IRI to obtain their latest requirements in this regard.

7.4 OFF-SITE TRANSPORT

As discussed in ***Section 7.1***, off-site transport of water produced by dewatering operations may be required when water is contaminated with pollutants, other than sediment and additional treatment. Off-site transport may also be required when there is no space on-site for retention and disposal, and there is no infrastructure available to discharge the water to the stormwater system. The need to transport water off-site should be evaluated on a case by case basis.

7.5 WATER DISCHARGE PERMITS

In Abu Dhabi City, the dewatering and the subsequent discharge of groundwater for Building and Infrastructure projects require compliance with the following steps:

1. Obtain a permit for dewatering prior to implementing any dewatering related works.
2. Obtain permission to discharge the groundwater in the storm water networks prior to actual discharge.

Consultants and contractors registered in the Electronic Permitting System of ADM (commonly known as CDP) can apply online to obtain the necessary permit and permission.

7.5.1 Procedures for Obtaining the Permit and Permission

1. Submit an application online through the CDP.
2. Upload the required documents and drawings.
3. Follow up on application status through the CDP.
4. Obtain the permit or the permission through customer service counters.

7.5.2 Service Application Forms and Required Documents

The required documentation, application forms, and presentations for the application steps for each permit are available on ADM web site www.adm.gov.ae in accordance with the following sequence as shown below:

7.5.2.1 Dewatering Permit

Documentation Center/Documents/Construction Permit/Geotechnical Unit.

7.5.2.2 Permission for Discharge of Groundwater

Documentation Center / Documents / Municipal Infrastructure & Assets / Infrastructure & Services Coordination / Permissions and NOC Certificates

7.6 MANAGEMENT OPTIONS AT CONTAMINATED SITES

Although not frequently encountered within the urban areas of Abu Dhabi City, contamination in the form of petroleum products, organic chemicals, solvents, and biological agents are common in developed or industrialized areas. Handling, treating, and disposing of these contaminants present a significant challenge when dewatering of a contaminated site is required. As discussed in ***Section 11.6***, it is important to evaluate groundwater quality and potential contaminants when planning a dewatering system or excavation. In general, three methods of dealing with contaminated dewatering effluent are used:

1. Exclusion – Where the contaminant plume is prevented from migrating during pumping by the installation of cutoff walls that block flow or by modification of the soil properties via grouting, freezing, or other methods. This method is applicable for sites where the locations of the contaminants are able to be closely defined, and where disturbance of the contaminated material is not required.

2. On-Site Treatment and Discharge – Where dewatering effluent is pumped through an on-site treatment facility, cleaned to an acceptable level, and subsequently discharged to a surface body of water, storm sewer, or sanitary sewer. This method requires expert knowledge of the contaminant and available treatment methodology and may require a full scale treatment test program prior to the start of production of dewatering to ensure the system has enough capacity to adequately treat the effluent. When on-site treatment systems are planned, extra effort should be made to quantify the volume of effluent that must be handled, typically via one or more pumping tests.

3. On-Site Storage and Off-Site Treatment – Where dewatering effluent is pumped into tanks and periodically removed from the site for treatment, and discharged at an offsite facility. This method may be in appropriate short Project durations or for small sites without room for an on-site treatment facility.

8.0 RISK MITIGATION MEASURES

This Section discusses additional measures that can be taken to reduce the risks associated with excavations below the groundwater table, including cutoff structures and soil, or rock conditioning or treatment through grouting or other means.

As discussed in ***Section 5.3***, the current practice to avoid removal of fine materials in ADM is to monitor the rate of water table drawdown, rather than pumping rates. Skilled dewatering contractors in ADM fine tune the rate of drawdown by closely monitoring turbidity of the outflowing waters. The optimal rate of drawdown has been found to be 0.5 m/day. Turbidity is not allowed to exceed 50 NTU. Monitoring of the rate of drawdown during dewatering should be made in piezometers installed halfway between dewatering wells, as specified in ***Table 12-2***. This point corresponds to the crest of the cone of depression between wells.

As described in ***Section 5.3***, monitoring of turbidity and water levels may be performed by using real time, remote data access systems, which are available in the monitoring device industry. Data can be retrieved and analyzed at any moment, which facilitates monitoring of drawdown rates and turbidity levels. These systems also offer the possibility of automatically increasing data logging during a specific event, offering flexibility for data acquisition depending on site-specific conditions during dewatering operations. It is highly recommended that automatic systems be employed in challenging dewatering projects.

The recommended spacing of monitoring wells for Abu Dhabi City is as given in ***Table 12-2***.

8.1 GROUNDWATER CUTOFF STRUCTURES

Cutoffs are required for excavations where the groundwater inflow is expected to be too high to be safely or efficiently managed and needed to provide redundancy in a dewatering system in the event of a loss of dewatering capacity. Groundwater cutoffs are physical barriers to groundwater flow, and may consist of walls made of steel or plastic sheet piles; concrete, soil-bentonite, cement-bentonite, or soil-cement-bentonite walls constructed by excavating a trench and backfilling with impermeable material; jet grouted walls; deep soil mixed walls; and cutoff walls installed in rock with secant piles or the overlapping slurry panels of a diaphragm wall. While it is the responsibility of the excavation contractor to decide the most constructible option for a cutoff wall, the permeability of the solution must be considered as well. For example, secant or diaphragm walls allow less excavation to be open at once, but the presence of joints between the piles or panels introduces a possibility for leakage. A slurry trench on the other hand, is placed continuously with no joints, so only the permeability of the wall itself is considered. Advantages and limitations relating to stiffness, volume of spoils, noise, and effect to nearby structures is further discussed in Chapter 21 of Powers, et al. (2007).

A cutoff wall should be designed by following well-established design standards and manuals such as Eurocode 7 (EN 1997, 2007), Strom and Ebeling, (2001), CIRIA C580 (2003), and Xanthakos (1994). Input, assumptions, the design code followed, analysis procedure and output should be properly documented. Seasonal changes and any fluctuations of groundwater levels or adverse changes due to dewatering should be considered in the design of the walls.

Cutoff walls may require a tie-back system for structural stability. In Abu Dhabi City, tie-backs are not allowed to extend under neighboring plots and internal bracing is required. Tie-backs may be allowed to extend below adjacent utilities and road corridors. However, special no objection certificates (NOC) would be required from both planning and main coordination section (Town Planning Sector), and from coordination of government relation services (Infrastructure and Municipal Assets Sector) during the permitting process of the shoring system (***Section 13.5***).

Typical methods used to analyze a cutoff wall tie-back system usually include beam on rigid support method, beam on inelastic supports analysis, and linear and nonlinear soil-structure finite element analysis. In general, the continuous beam on rigid support method, with staged excavation analysis, will provide reasonable estimates of anchor forces and wall bending moments. This type of construction sequencing analysis is often followed by a beam on elastic or inelastic foundation (Winkler) construction sequencing analysis to verify anchor forces (Strom and Ebeling, 2001). The behavior of multi-anchored systems may be strongly influenced by factors such as the sequence of excavation and installation of anchors, fluctuations in the water table, and the nonlinear stress-strain behavior of soils. In order to accurately obtain the magnitudes of stresses and deformations in the structure and the surrounding soil, it would be necessary to perform soil-structure interaction analyses that model the construction and operation stages of the system.

If conditions warrant, it is preferable to socket the cutoff wall into competent rock. This will ensure a better isolation for groundwater inflow. In addition, the cutoff walls socketed into competent rock is inherently stable. Because soil mass retained by the wall will be bearing on the rock surface, potential failure surfaces through the rock mass will not induce low factors of safety. It is important to note that measures should be taken when it is absolutely necessary to socket these structures into soluble strata, such as gypsum. As described in ***Section 5.6***, the key to effective dewatering design and operation when socketing cutoff structures into gypsum is to ensure close monitoring of the flow rates, as well as the chemical composition of the discharge water. The main parameter to be tracked is electrical conductivity. Monitoring devices are discussed in ***Section 5.3***.

8.1.1 Sheet Pile Walls

The use of sheet pile walls requires specific ground conditions where the tip of the sheet piles does not penetrate in hard rock or soluble rock like gypsum. Too often, the heavy driving of sheet piles can cause issues with interlocking and proper sealing at the tip. Therefore, in Abu Dhabi City, the use of sheet piles is not recommended for cut-off in high risk areas.

Sheet pile walls are the simplest type of groundwater cutoff and are very commonly used to provide excavation support. Because sheet pile walls have numerous vertical joints, where the sheets fit together (interlocks), the effectiveness of sheet piles as a water barrier can be significantly reduced if the interlocks between piles do not fit tightly or are not provided with additional treatment. Additionally, driving piles in cobbly, very dense, soils with other obstructions, or very hard rock, may cause the piles to detach at depth, leaving a window in the cutoff that may not be able to be identified during installation. Interlock treatments may include bituminous or hydrophilic joint sealants applied to the interlocks prior to driving the sheets, or grout injections performed after driving the piles. The post grouted method is more reliable for water tightness but requires a significant additional level of effort. Sheet pile cutoffs are not recommended for soils with significant boulders, rubble, or other obstructions, due to the tendency of the interlocks failing. If sheets are to be driven to an irregular rock surface, the designer should understand that significant windows at the base of the wall are likely due to the inability of the sheets to uniformly penetrate the rock. Sheet pile cutoffs are generally most effective in loose to medium dense uniform soils without significant obstructions, preferably tipped into a clay strata or Baynunah Formation (siltstone/mudstone) such as rock layer observed in Shahkbout City. Typical sheet pile cutoff walls, during installation, are shown on ***Figures 8-1 and 8-2***.

FIGURE 8-1
TYPICAL SHEET PILE CUTOFF DURING CONSTRUCTION

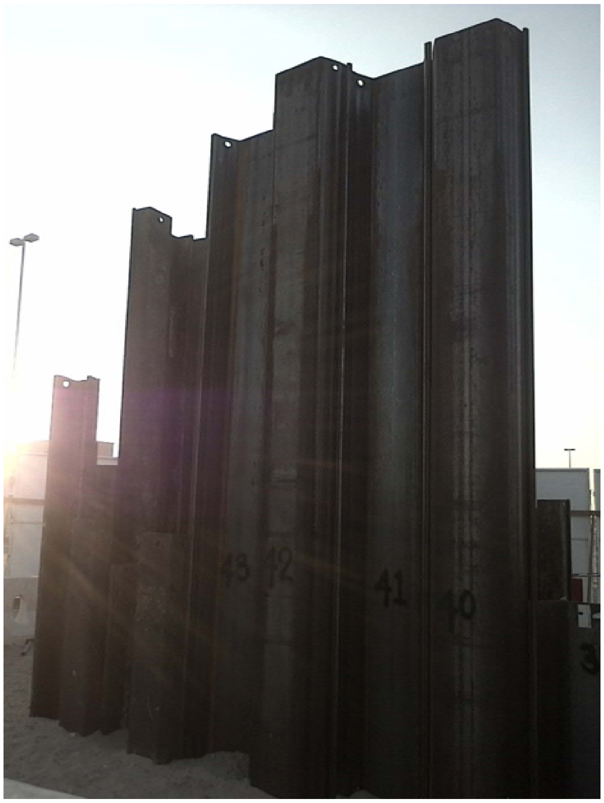


FIGURE 8-2
SHEET PILE CUTOFF FOR CONSTRUCTION OF DRIVING PIT IN SHAKHBOUT CITY, 2013

8.1.2 Trenched Walls

Trenched walls are very common, and typical cost effective cutoffs are frequently used to limit groundwater flow or contain contaminated groundwater. These walls may be excavated using modified excavators fitted with “long reach” booms down to depths around 30 m, or may be excavated using craned mounted hydraulic or mechanical clamshell excavators to greater depths as shown on ***Figure 8-3***.

FIGURE 8-3
LONG REACH EXCAVATOR

The type of backfill selected for trenched walls depends on the required permeability, strength of the wall, and the type of soil that is excavated from the trench. For simple groundwater control walls in soils that have a reasonable amount of fine particles, the wall may be backfilled with native soils mixed with bentonite clay in slurry, producing a wall with low shear strength and low permeability. For sites where the soils have insufficient fine particles or where some strength is desired in the final wall cement and bentonite may be added to excavated soils and used for backfill. Bentonite doses are typically less than five percent by weight of slurry, while cement doses are typically less than two percent by weight of slurry. Backfill for these slurry walls may be placed by replacing material into the trench with an excavator or by pumping slurry through a tremie. Thorough quality control is required in the backfill phase to ensure continuity of the wall. Typical causes for a non-homogenous or leaky wall include:

- High groundwater table or insufficient slurry head
- Sudden slurry losses through pervious materials
- Improper socketing the cutoff wall into soluble rock, such as gypsum
- Spalling of the excavation sidewall
- Inadequate excavation/cleaning of base of wall
- Improperly or poorly mixed backfill
- Free dropping of backfill into a trench instead of using a tremie
- Encapsulation of slurry pockets in the backfill
- Entrapped sediment
- Stockpiling of excavation spoil near the trench
- Unstable soils, such as soft clay and peat

Considerations on the stability of slurry trenches can be found in the literature (e.g., Nash, 1974; Wong, 1984; and Fox, 2004).

For cutoff walls that are required to provide structural support to the excavation or other structures, concrete backfill with or without reinforcement is often employed. Similar to trenched slurry walls, a panel is excavated under slurry and backfilled with concrete via tremie from the bottom up. Slurry is collected as it is displaced by the concrete backfill and used in subsequent panel excavation. Cutoff wall panels may be excavated using excavators, clamshells, or hydro mills. Great attention must be paid to the joints between panels to ensure that the joints do not leak excessively. Typically, subsequent panels are excavated one or more wall thicknesses into the older concrete to prevent a cold joint from acting as a leakage path in the finished wall. ***Figure 8-4*** shows the typical construction progression for a slurry wall built with concrete backfill.

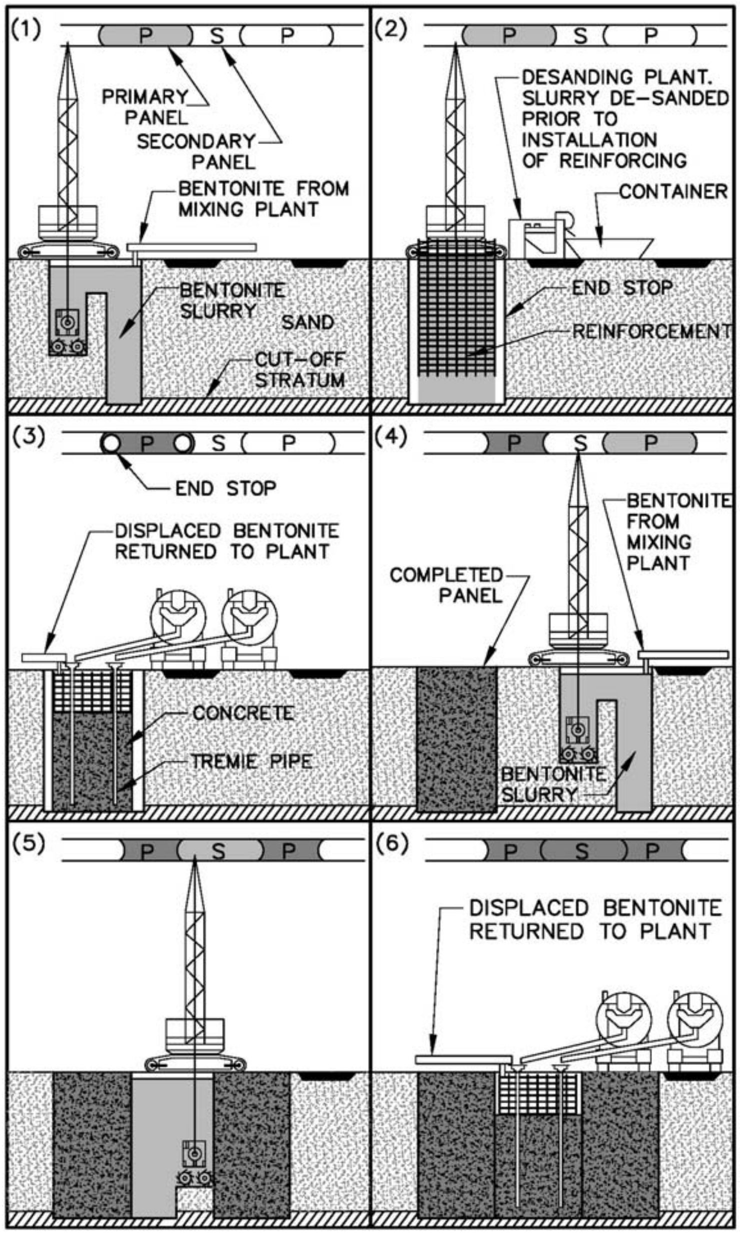


FIGURE 8-4
TYPICAL PANEL SLURRY WALL CONSTRUCTION
(Powers, et al., 2007)

Deep Soil Mixed (DSM) walls are a combination of trenched slurry walls and panel slurry walls. DSM walls are constructed using specially designed machines that mix the native soil with the cement or other binder in-situ as they advance or are removed. The machine is advanced to the required depth, the tool is removed, the machine moves to the next overlapping panel, and so on. DSM machines are available in several configurations, including multiple augers acting together and hydromill type systems.

Jet grouting can be used to create nearly any desired cutoff wall geometry and it is a very useful method in urban settings, where the working area is limited, preventing the use of methods that create large amounts of spoils or that require large mixing basins. Jet grouted cutoffs are created by injecting cement grout, air, water, or combinations of the three at very high pressures into the soil using a drill rig. The high pressure jets remove the soil structure and allow for very thorough mixing of the binder materials and existing soils, resulting in a soil cement element being constructed. High levels of quality control are required for jet grouting to ensure that uniform geometry is being applied to the full column, to ensure the verticality (or other orientation) of the element to limit the existence of windows, and to ensure that the proper mix of binder materials is being injected. Jet grouted walls can be engineered to be self-supporting or may be anchored when they are required to provide structural support.

Groundwater cutoff walls required to penetrate the rock are usually constructed as concrete backfilled panel walls (diaphragm walls) or as secant pile walls. The installation of diaphragm walls can induce closure of the trench. These movements tend to be localized, but can be significant within 5 to 10m of the wall (Clough and O'Rourke, 1990). As described in Section 8.1, care needs to be exercised in the construction process of diaphragm walls, especially by ensuring the proper placement of the stabilizing fluid (slurry).

In terms of trench stability, secant pile walls pose no challenge. With diaphragm walls, on the other hand, care needs to be taken in the construction process, especially by ensuring the proper placement of the stabilizing fluid (slurry). Trench stability is mostly provided by the fluid weight of the slurry and the arching action of the soil around the trench (Richards, 2006). In addition to proper placement of slurry in the trench, guidewalls shall be employed in diaphragm wall construction. They provide a template for wall excavation and panel layout, support the top of the trench, restrain the endstops, serve as a platform to hang the reinforcement, support the tremie pipes, hold down the cage during concreting, and provide reaction for jacking out some types of endstops. Guidewalls are reinforced concrete typically 1.2 to 1.5 m deep. The top of the guidewalls should be at least 1.2 m above the groundwater table to allow for construction in the dry and to allow for slurry level to be 1 m above groundwater table (Richards, 2006).

8.1.3 Secant Pile Walls

Alternative to the diaphragm walls, where groundwater seepage is not a concern secant piles can be used as a cheaper support system. Piles are advanced using air rotary hammers or very heavy rotary bits in primary holes. When the desired depth is reached, the hole is cleaned and backfilled with concrete using a tremie. After the primary piles are backfilled and the concrete has sufficiently set, secondary piles are drilled, overlapping the primary piles to complete the wall. Depending on the structural demand on the wall, reinforcing cages or steel beams may be installed in the secondary piles. ***Figure 8-5*** shows the typical progression of secant pile wall construction.

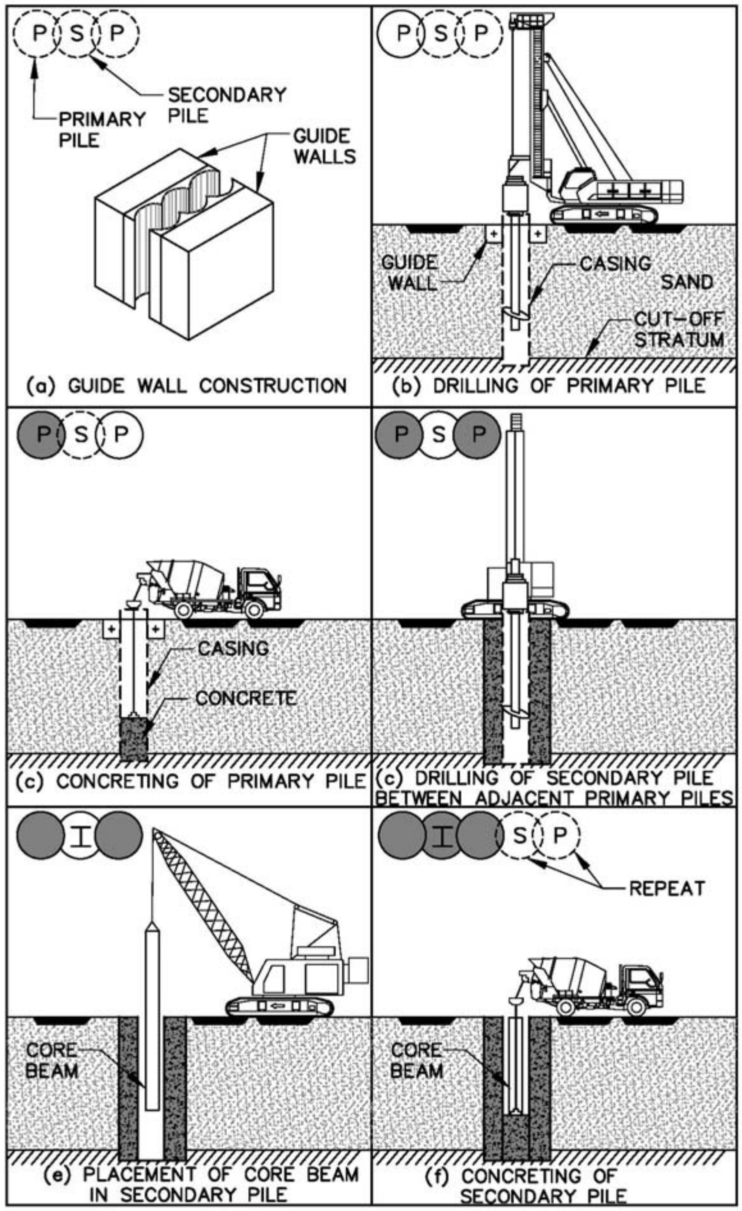


FIGURE 8-5
SECANT PILE CONSTRUCTION PROGRESSION
(Powers, et al., 2007)

8.2 SOIL CONDITIONING

In some soils, modifications to the in-situ soil properties with permeation grouting may reduce the hydraulic conductivity, increase the strength and eliminate, or reduce the need for structural cutoffs and dewatering. Permeation grouting may be performed with particulate grouts, which are typically cement slurry based, or with chemical grouts, which are chemical solutions. Soil conditioning by permeation grouting may also be used to provide containment of environmental contaminants, to close windows in structural cutoffs, such as sheet pile walls, to seal localized areas of high hydraulic conductivity, such as gravel beds, or to increase stand up time in tunneling or other applications.

The effectiveness of soil conditioning by permeation grouting is inversely proportional to the fineness of a soil, the viscosity of the grout, directly proportional to the grouting pressure. For example, open graded gravel and sand deposits may be rendered nearly completely impermeable by permeation grouting, while silt and clay deposits may not be improved at all. In order to evaluate a soil's amenability to grouting and to select permeation grouting technique, in-situ permeability test data, and grain size analyses should be available. Relative density of a soil may also impact the success of permeation grouting, possibly requiring high pressures to facilitate injection. The typical industry rule of thumb for evaluating the "groutability" of a soil using permeation techniques is:

1. Soils with permeability between 10^{-1} and 10^{-3} centimeters per second (cm/s) are easily groutable.
2. Soils with permeability between 10^{-3} and 10^{-4} cm/s are moderately groutable.
3. Soils with permeability lower than 10^{-5} cm/s are not groutable.

Figure 8-6 shows some typical ranges of applicability of permeation grout materials for various ranges of hydraulic conductivity. ***Table 3-5*** shows typical permeability values encountered in the Abu Dhabi.

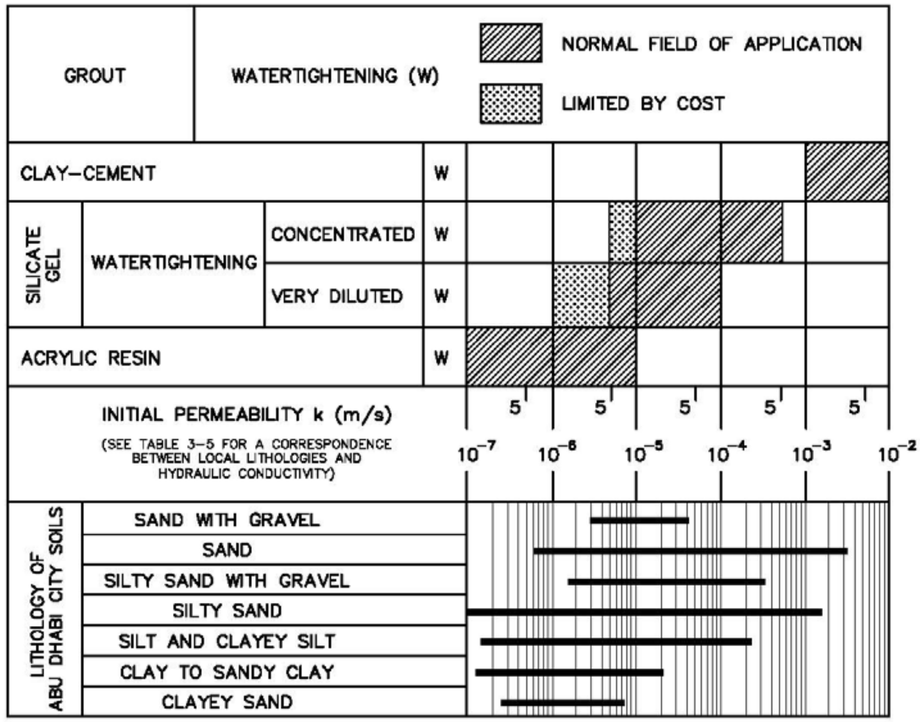


FIGURE 8-6
PERMEATION GROUT AMENABILITY VS. HYDRAULIC CONDUCTIVITY
(Adapted from Powers, et al., 2007)

Particulate or chemical grouts may be injected directly into a soil mass through an open borehole, from the end of a drill casing, or may be injected through the sleeve port pipes (tube a machettes), if repeated applications are desired. When selecting a permeation grouting material, the grout mix for particulate grouts or the composition of the chemical grout must be selected to ensure that the grout will achieve sufficient penetration, without causing hydrofracturing of the soil mass. This will sufficiently fill and bond the pore space in the soil mass and will not suffer from chemical attack, or other adverse reactions to contaminants or other substances in the soil.

8.3 GROUTING OF ROCK FRACTURES

Some rock types can be impermeable when intact, such as fine-grained limestones and mudstones, well-cemented granular rocks, or crystalline rocks. In ADM, these include units such as the intact interbedded marine mudstones and crystalline gypsum of the Gachsaran Formation or well cemented layers of the Ghayathi, Baynunah, Hili, or Barzaman formations. Impermeability of rock leads to concentration of groundwater flow in fractures, bedding planes, and other zones of weakness. These concentrated flows can result in enhancement of the size and length of the zones of weakness leading to formation of voids, particularly in soft, easily eroded, marine mudstones and siltstones or soluble gypsum units. Concentrated flows can also result in heave or boils in the base of excavations with thin soil layers above the rock, or may result in excessive flows into excavations performed to the rock.

Relatively clean fractures and voids in the rock masses are typically groutable using cement-based particulate grouts. Defects, which have extensive infill, clayey weathered material, or very large voids, may require application of one of the previously discussed cutoff wall construction methods to provide long term seepage reduction. In highly permeable rock masses, where cutoffs are to be installed, the existing defects may be pre-grouted to prevent catastrophic losses of drilling slurry and subsequent hole collapse.

Rock grouting techniques and mix designs have improved significantly in recent years. In the past, very thin grout mixes were used in an attempt to increase the "penetrability," or travel distance along a defect of the grout mix. Previously used high water had cement ratio grouts that commonly experienced up to 90 percent volume reduction during the curing process as water bled from the mix, resulting in 90 percent of the grouted defect remaining open.

Today, additives, such as long chain polymers, bentonite, superplasticizers, and other admixtures allow high density, stable (<5 percent bleed) grouts that resist the effects of pressure filtration and exhibit excellent penetrability. Cementitious grouts used to seal rock fractures should have the following characteristics:

1. Minimal Bleed (<5 percent) – Bleed is the tendency of cement particles to fall out of suspension as the grout cures, resulting in a volumetric loss with time. Modern grouts can be readily formulated to exhibit negligible bleed, ensuring full fracture filling.

2. High Resistance to Pressure Filtration – Pressure filtration is the tendency of the water in the grout mix to be pushed out of the mix as the grout travels through defects, resulting in blocking of the defect as the solids in the mix accumulate. The use of polymer additives (welan gum, diuatan gum, and others) is recommended to ensure that homogenous product with maximum penetrability is injected.

3. Appropriate Viscosity – A simplified definition of grout viscosity is that it is a measure of the grout's resistance to flow by its own internal friction. Viscosity of the grout mix must be appropriate to the size of defects and compatible with acceptable injection time. Low viscosity mixes are appropriate to small/narrow defects while higher viscosity formulations may be used in larger defects that are easier to push grout into.

4. Thixotrophy – Grout mixes should be thixotrophic, meaning they should be cohesive and homogenous when at rest, and should readily flow when energy is applied to them via pumping. Thixotrophic mixes resist segregation while preserving penetrability of the grout.

5. Durability – The grout mix should exhibit sufficient density, homogeneity, and strength after curing to provide a durable seal in the injected defects.

After grout holes are drilled, they are typically water pressure tested prior to grout injection. Water pressure testing serves to further characterize the subsurface conditions at the site and ensures that the rock mass is prewetted, helping reduce grout mix water loss into the rock mass during grout injection. The water pressure tests identify areas of high hydraulic conductivity and provide a baseline for comparison to tests performed after grouting. Water pressure testing does not always provide a clear indication of the expected grout take, but is a useful planning and verification tool.

Water pressure tests and grout injections should be monitored and controlled by a computer-based real-time monitoring system. These systems monitor pressure, flow, stage depth, and other information depending on the setup and serve to accurately record pay items (grout volume injected, and pumping time), and allow an experienced operator to ensure the efficiency of the grout program.

9.0 PRE-CONSTRUCTION PLANNING

Prior to beginning the construction, it is critical to evaluate subsurface conditions, including groundwater levels and conditions, and thoroughly plan the dewatering program that will be required to complete the Project successfully.

9.1 DETERMINING DEWATERING NEEDS AND TIME SCHEDULES

The location, geometry, type of excavation, type of soil to be excavated, and the duration of dewatering are important considerations in the selection and design of a dewatering system. For most granular soils, such as sandy soils, the groundwater table during construction should be maintained at least 1 m below the slopes and the bottom of an excavation in order to ensure "dry" working conditions. It may need to be maintained at lower depths for silts (1.5 to 3 m below subgrade) to prevent water pumping to the surface or causing wet and spongy conditions in the excavation. Where the bottom of an excavation is underlain by a less pervious stratum, like soft carbonate rock interbedded with gypsum that is underlain by a pervious formation under artesian pressure, the upward pressure or seepage may cause wet conditions or even heave at the bottom of the excavation without a proper dewatering system, even though the excavated pit and slopes may be dry.

An effective dewatering system may be required to allow construction of subsurface structures founded in, or underlain by, strata below the water table by:

1. Intercepting seepage that would otherwise emerge from the slopes or bottom of an excavation.
2. Increasing the stability of excavated slopes and preventing the loss of material from the slopes or bottom of the excavation.
3. Reducing or controlling lateral loads on retaining walls, structures, or cofferdams.
4. Eliminating the need for, or reducing, air pressure in tunneling.
5. Improving the excavation and backfill characteristics of sandy soils.

As discussed in ***Sections 5.0 and 6.0***, uncontrolled or improperly controlled dewatering can cause piping, heave, slope instability, settlement, dissolution of soluble minerals, and other impacts, which can significantly impact the Project being constructed and other nearby structures. Therefore, subsurface construction should not be done without appropriate control of the groundwater and (subsurface) hydrostatic pressure.

The construction schedule must be determined and evaluated before proceeding with the design of a dewatering system. The time required to achieve the required level of the dewatering depends on the soil type, size, and volume of the excavation, method(s) of dewatering and the design of the dewatering system. Dewatering in fine grained soils may require a significantly longer time than dewatering in coarser grained soils. The number, size, spacing, and depth of wells or other dewatering measures, and the rate at which water must be removed to achieve the required groundwater drawdown or pressure relief, must be determined and adjusted to meet the construction scheduling. In some cases, such as construction of a utility line, it may be necessary to perform dewatering in zones along the alignment of the work zone in stages, with the dewatered zone moving along with and ahead of the excavation or work zone. In other cases, it may be possible to dewater the entire alignment or larger portions of the work zone at once.

Depending on the Project requirements, the design should include instrumentation, such as piezometers, as discussed in ***Section 11***, to monitor and calibrate the dewatering system. The dewatering system may also require adjustments based on the initial performance of the system.

9.2 IDENTIFICATION OF PERMEABLE ZONES

Section 11 of these Guidelines describes geotechnical investigation programs related to designing the dewatering system. Based on borehole logging, monitoring of piezometers and wells, and geophysical measurements, potential permeable zones are identified. Depending upon the Project requirements, additional field tests, such as borehole seepage and pumping tests may be performed to evaluate the quantity of the water likely to be encountered during dewatering operations.

In the analysis of any dewatering system, the source of seepage must be determined, and the boundaries and seepage flow characteristics of geologic and soil formations at and adjacent to the site must be generalized into a form that can be analyzed. In some cases, the dewatering system and soil, and groundwater flow conditions can be generalized into rather simple configurations. For example, the source of seepage can be modeled as a line or circle; the aquifer as a homogeneous, isotropic formation of uniform thickness; and the dewatering system as one or two parallel lines or circles of wells or wellpoints.

9.3 IDENTIFICATION OF ZONES OF POTENTIAL COLLAPSE

The scope of the geotechnical investigation and laboratory testing for a project should include evaluation of potential zones of cavity collapse. The formation of cavities involves natural processes of erosion or gradual removal of slightly soluble bedrock (such as limestone, gypsum, or rock salt) by percolating water, the collapse of a cave roof, or a lowering of the water table. The geotechnical investigation should identify the existence of the collapsible or soluble materials. If such material exists, which is likely in the ADM, the dewatering system and excavation should be designed to prevent the collapse of potential cavities, which could be influenced by the dewatering. The geotechnical investigation should be designed based on the location of the Project along the suggested scope in ***Table 11-3***.

9.4 EVALUATION OF STRUCTURES AND INFRASTRUCTURE IN SURROUNDING AREAS

There is a potential risk that dewatering may result in undesirable consequences, such as settlement, collapse of cavities, dissolution of evaporites, and fine material removal/migration. As discussed in ***Section 5.0***, when the water table is lowered, it increases the effective stress in the soil mass. Compressible soils and loose granular soils have the potential to compress and consolidate when the water table is lowered and the effective stress is increased. If dewatering is carried out properly so that no loss of material occurs due to open pumping and there is no pumping of fine particles from the wells, then settlement is not likely to occur in competent soils. If weak soils are present in the vicinity, such as normally consolidated clays, silts, peats, or loose granular deposits, and uncontrolled/unengineered fills then settlement or consolidation is a possibility during dewatering.

If the geotechnical investigations identify weak soils that may be subject to settlement or consolidation, a general survey of the foundation within the radius of the influence area should be undertaken and a detailed analysis should be performed.

Although not largely applicable to the Abu Dhabi environment, it is important to note that groundwater supplies may also suffer temporary or have a long-term impact from nearby dewatering. Temporary impacts may include reduction in well capacity; possible long-term impacts include saltwater intrusion or accelerated migration of contaminated plumes of groundwater.

The design of the dewatering system should consider potential impacts of the dewatering on nearby structures and mitigating factors should be included in the design, if required. As a minimum, monitoring instrumentation, such as flow meters on the discharge outlet and piezometers and settlement gages, should be installed at critical locations and base line data recorded prior to dewatering. Instrument readings should be recorded in a periodic basis during dewatering to evaluate the performance of the system and its impact on nearby structures.

Changes in groundwater flow during dewatering are a risk trigger. It could indicate enlargement of fractures, formation of pipes within the soil mass, or dissolution of rocks leading to enlargement of cavities and subsequent instabilities.

Variation in groundwater flow from that estimated during design, whether under transient or steady state conditions, could also reflect incorrect assumptions regarding soil conditions and the potential presence of subsurface features that may warrant either shut down of the dewatering system or additional investigation to evaluate the anomaly.

Most dewatering flow calculations are made under steady state conditions, while initially and for a significant period of time, transient conditions occur, whereby the initial flow could be significantly larger than that estimated under steady state conditions. Experience and judgment is therefore required during the evaluation of measured ground flow to differentiate the cause of groundwater increase.

The supervision of dewatering activities by experienced staff and field personnel is therefore one of the key elements of a successful dewatering project.

9.5 PARAMETERS IMPACTING THE DEWATERING DESIGN ACTIVITIES

Required level of detail in analysis, design, geotechnical investigation, and monitoring for dewatering works are impacted by several parameters. Economical and risk to human life based on the consequence of the damage that will result from the dewatering related failures will also influence the scope and level of detail of the dewatering works. Characteristics of neighboring structures, dimension and type of dewatering works and soil conditions are the major determining factors. The potential impact of dewatering under local geologic and construction conditions is discussed throughout the Guidelines. In these Guidelines, geologic and hydrogeological factors, proximity of structures, depth of excavation, and the type of dewatering and excavation support system are considered in the determination of minimum level of required during design and construction of the dewatering systems. Scopes of the dewatering design, geotechnical investigation, monitoring program, or type of analysis approach for dewatering calculations are determined based on these parameters. ***Table 9-1*** presents the factors used in determination of detail level of dewatering design and monitoring activities. This Table is used to determine the extent of the dewatering design scope (***Section 10.0***), geotechnical investigations (***Section 11.0***), and monitoring program (***Section 12.0***).

Existence of soil with fine materials, loose or soft soils, uncontrolled fill, salt layers or geologic units with carbonated material, solution cavities may present a risk during dewatering process as explained in this document. ADM initiated a project to identify the possible geological and hydrogeological risks and publish maps presenting the level of risk potential. These maps can be accessed by the ADM website.

Proximity and type of nearby structures are other key factors to determine the extent of the dewatering design and monitoring system. This factor is grouped into three categories:
1) existence of sensitive or large structures nearby which forms the high risk potential group;
2) existence of structures not included in Item No. 1); and the last group 3) includes the cases where there is no nearby structure.

TABLE 9-1
FACTORS USED TO IDENTIFY THE DETAIL LEVEL OF DEWATERING RELATED WORKS

FACTOR	GROUP	DESCRIPTION
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3m
	Medium	3m-10m
	Deep	>10m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors

10.0 DEWATERING DESIGN

The objectives of a dewatering system are generally to allow safe construction of a project by:

1. Lowering the water table and intercepting influencing seepage which would otherwise enter the excavation and interfere with the work.
2. Improve the stability of excavated slopes.
3. Prevent heave in the bottom of excavation.
4. Reduce lateral pressures on temporary sheeting and bracing exerted by the soils from outside the shoring.

As discussed in ***Section 4.0***, groundwater can be controlled by surface drains, wells, cutoff walls, other methods, or combinations of methods. This Section of the Guidelines describes design methods, which are normally applicable for lowering the groundwater by pumping, various wellpoint methods, or deep wells.

Dewatering systems may be designed by one of the following methods:

1. Analytical Methods
2. Flow Net Analysis
3. Numerical Modeling

10.1 INPUT PARAMETERS REQUIRED FOR DEWATERING DESIGN

10.1.1 Hydraulic Conductivity and Permeability

Permeability is defined as the facility for water flow through a soil mass. In most soils, the flow of water through the void spaces can be considered laminar and the discharge velocity (v) is proportional to hydraulic gradient (i), where i is defined as h/l, h is the hydraulic head loss, and l is the distance travelled; thus,

$v \propto i$ or $v \propto h/l$

$$v = ki \qquad \text{(Equation 10-1)}$$

k is the constant of proportionality and is defined as the conductivity of the materials, through which the water is flowing. In general terms, the hydraulic conductivity is also known as the coefficient of permeability of the materials. In this equation, v is the discharge velocity of water based on the cross-sectional area of the soil. However, the actual velocity of water, as it passes through the void spaces in the soil, is greater than the flow velocity. The permeability of saturated soils can be determined by the following methods:

1. In a laboratory by a Constant-Head or Falling-Head test.
2. In the field through a Pumping Test (ASTM, 2008b, 2008c, 2010a, 2010d; BS5930; BS ISO 14686; U.S. Bureau of Reclamation, 1995), packer test (ASTM, 2008d; USACE, 1980), or slug tests (ASTM, 2008a, 2010b).
3. Through empirical relationships.

Permeability varies widely for different soils and rock masses. Some typical values for saturated soils are given in ***Table 10-1***; however, conditions are different for every site, and the values provided below are for information only. The permeability of unsaturated soils is lower and increases rapidly with degree of saturation. Refer to ***Table 3-5*** for values of hydraulic conductivity for various lithologies.

TABLE 10-1
TYPICAL VALUES OF PERMEABILITY OF SATURATED SOILS
(Refer to ***Table 3-5*** for values of hydraulic conductivity for various lithologies in Abu Dhabi)

SOIL TYPE	PERMEABILITY (k) (cm/s)
Clean gravel	100 - 1
Coarse sand	1.0 – 0.01
Fine Sand	0.01 – 0.001
Silty clay	0.001 – 0.00001
Clay	< 0.000001

10.1.2 Transmissivity

The transmissivity (T) of an aquifer is defined as the quantity of water that flows through the aquifer; thus,

$$T = kB \qquad \text{(Equation 10-2)}$$

where,
B is the thickness of the aquifer.

Transmissivity is an important factor in determining the quantity of water that must be pumped on a dewatering project. It is a very useful concept in the analysis of aquifers, especially complex natural aquifers. When transmissivity is determined from a pumping test, it is an equivalent isotropic transmissivity that defines how the natural aquifer of interest will perform. The thickness B of the aquifer can be estimated from soil borings or inferred from the geology, so the equivalent permeability (k) of the soil can be computed using the relationship defined above.

Field tests for the determination of hydraulic properties of an aquifer are highly encouraged. A reliable estimation of hydraulic conductivity through field tests will lead to an optimum design and operation of a dewatering system, which is translated in time and money savings. A variety of technical problems can be successfully avoided if proper field test results are available to the dewatering design consultant. Three main field tests are considered in these guidelines, in order of project importance: pumping tests, Packer tests, and slug tests. ***Table 11-3*** gives recommended field tests as a function of hazard category, excavation depth, and excavation method.

For pumping tests, the British Standard BS14686 may be followed. This standard gives a complete outline of pumping tests including durations of step tests and water level measurement intervals.

10.1.3 Aquifer Storage and Depletion

The initial head H in the aquifer is normally inferred from observations during test borings. More reliable values are obtained from piezometers or observation wells that have been designed and constructed with prior knowledge of the stratification so that they can be set at the appropriate depth and be screened, filtered, and sealed as per the dewatering requirements. To establish a dewatered or pressure-relieved condition, it is necessary to pump the water stored in the aquifer as the head is lowered to the desired level. Before equilibrium can be reached, some quantity of water must be pumped in addition to the water that will be pumped for a steady-state flow requirement. For confined aquifers, the quantity of water stored is generally small, compared to the quantity for unconfined aquifers.

10.2 Radial Flow in Unconfined, Confined, and Artesian Aquifers

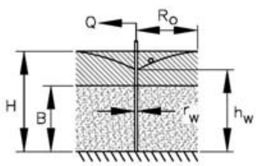
The most significant unknowns for any dewatering system are the total quantity of water, Q that must be pumped to accomplish the desired goal and the quantity of the water, Q_w that can be expected from an individual well or wellpoint. Q and Q_w are based on the decisions regarding spacing, design, and construction of wells or wellpoints, and on the pumps and pumping system used. The analytical methods for dewatering normally assume radial flow in an aquifer under steady-state conditions. The formulas summarized in ***Table 10.2*** are normally used to evaluate the performance of a dewatering system using analytical methods.

Unconfined conditions should be encountered in any of the unconsolidated deposits (Quaternary sands, gravels, or fill material) and in any bedrock formations located up near the ground surface. If bedrock aquifers are located below fine-grained sediments, siltstone, claystone, or mudstone, then there is a high probability that the deeper aquifers will be confined or semi-confined. Confining conditions might be encountered at depths greater than about 20 m.

TABLE 10-2
SUMMARY OF ANALYTICAL MODELS (STEADY STATE CONDITIONS)

(Modified from Powers, et al., 2007)

MODEL | BASIC EQUATION | METRIC UNITS[b]

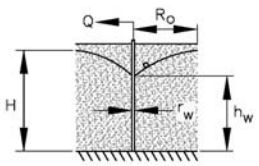


RADIAL FLOW, CONFINED AQUIFER

$$Q_w = \frac{2\pi KB(H - h_w)}{\ln R_o / r_w}$$

$$Q_w = \frac{KB(H - h_w)}{2.65 \times 10^{-6} \ln R_o / r_w}$$

K=HYDRAULIC CONDUCTIVITY

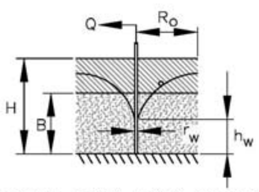


RADIAL FLOW, WATER TABLE AQUIFER

$$Q_w = \frac{\pi K(H^2 - h_w^2)}{\ln R_o / r_w}$$

$$Q_w = \frac{K(H^2 - h_w^2)}{5.31 \times 10^{-6} \ln R_o / r_w}$$

K=HYDRAULIC CONDUCTIVITY

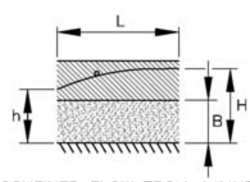


RADIAL FLOW, MIXED AQUIFER

$$Q_w = \frac{\pi K(2BH - B^2 - h_w^2)}{\ln R_o / r_w}$$

$$Q_w = \frac{K(2BH - B^2 - h_w^2)}{5.31 \times 10^{-6} \ln R_o / r_w}$$

K=HYDRAULIC CONDUCTIVITY

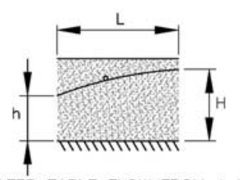


CONFINED FLOW FROM A LINE SOURCE TO A DRAINAGE TRENCH

$$\frac{Q}{x} = \frac{KB(H - h)}{L}$$

$$\frac{Q}{x} = \frac{KB(H - h)}{1.67 \times 10^{-5} L}$$

x=UNIT LENGTH OF TRENCH, FOR FLOW FROM 2 SIDES, USE TWICE THE INDICATED VALUE
K=HYDRAULIC CONDUCTIVITY

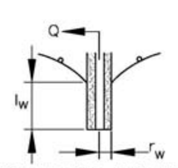


WATER TABLE FLOW FROM A LINE SOURCE TO A DRAINAGE TRENCH

$$\frac{Q}{x} = \frac{K(H^2 - h^2)}{2L}$$

$$\frac{Q}{x} = \frac{K(H^2 - h^2)}{3.34 \times 10^{-5} L}$$

x=UNIT LENGTH OF TRENCH, FOR FLOW FROM 2 SIDES, USE TWICE THE INDICATED VALUE
K=HYDRAULIC CONDUCTIVITY



RECOMMENDED FLOW PER UNIT LENGTH OF WET BOREHOLE (SICHART)

$$Q = 2\pi l_w r_w C\sqrt{K}$$

$$Q_w = 24.91 l_w r_w \sqrt{K}$$

r_w IN mm
l_w IN mm

C=EMPIRICAL COEFFICIENT

[b] EXCEPT WHERE NOTED: Q IN L/min; H, B, R_o, r_w IN m; K IN m/sec

10.2.1 Radial Flow in a Confined Aquifer

The equation given on ***Figure 10-1*** is normally used for estimating flow from a well of radius r_w that fully penetrates a confined aquifer of permeability k and thickness B and that is pumping at a rate Q_w. At a radius of influence, R_0 from the well, a limited source of water under head H communicates along the cylindrical surface, represented by ab. Beyond the radius of influence, there is no draw down due to pumping. Pumping at the constant rate Q_w reduces the head at r_w to h_w. In this equilibrium situation (***Figure 10.1***):

$$Qw = \frac{2\pi kB(H-h_w)}{\ln R_0/r_w} \qquad \text{(Equation 10-3)}$$

The drawdown H-h at any distance r from the well will be

$$H\text{-}h = \frac{Q_w}{2\pi kB}(\ln R_0/r_w) \qquad \text{(Equation 10-4)}$$

The radius of influence may be reliably estimated from a pumping test as explained by Powers et al., (2007). Lacking results from a pumping test can be approximated from following empirical relationship:

$$R0 = 3000(H\text{-}h)\sqrt{k} \qquad \text{(Equation 10-5)}$$

where,
H-h is in feet and K is in meters per second.

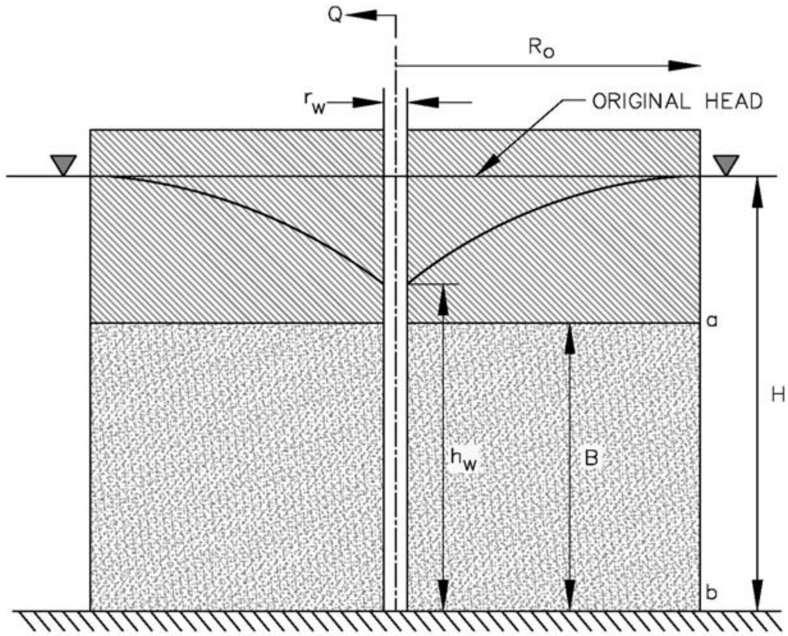


FIGURE 10-1
EQUILIBRIUM RADIAL FLOW TO A FRICTIONLESS WELL IN A CONFINED AQUIFER
(Powers, et al., 2007)

10.2.2 Radial Flow in an Unconfined Aquifer

Flow in an unconfined aquifer (also known as Water Table Aquifer) is more complex since the saturated thickness, and therefore, transmissivity, decreases closer to the well. Furthermore, because of complex boundary conditions at the phreatic surface, water table problems are theoretically indeterminate. However, good approximations of the flow can be estimated from the following relationship (***Figure 10.2***):

$$Qw = \frac{\pi k(H^2 - h_w{}^2)}{\ln R_0/r_w} \quad \text{(Equation 10-6)}$$

The height h of the phreatic surface at a distance r from the well, when r is greater than H (where H is the original saturated thickness), may be estimated as follows:

$$h = \sqrt{H^2 - Q_w \ln \frac{R_0}{r} / \pi k} \quad \text{(Equation 10-7)}$$

This relationship may not give satisfactory solutions for h, where r is less than approximately 1.0H.

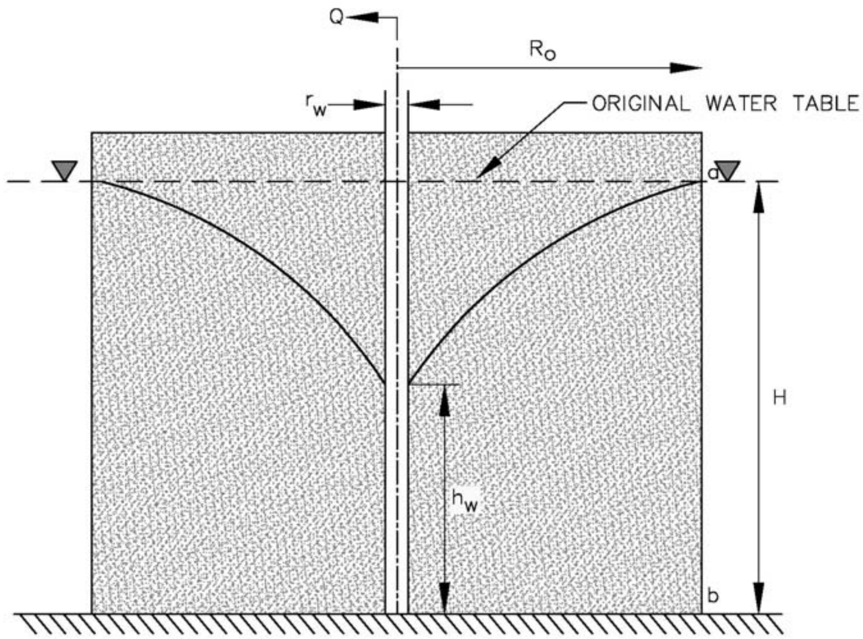


FIGURE 10-2
EQUILIBRIUM RADIAL FLOW TO A FRICTIONLESS WELL IN A WATER TABLE AQUIFER
(Powers, et al., 2007)

10.2.3 Radial Flow in an Artesian Aquifer

For artesian flow, consider a pervious stratum of thickness B, bounded above and below by impervious strata as shown on ***Figure 10.1*** for the confined case, and assume that the seepage enters through the pervious stratum. Further, consider that the water is pumped continuously, but that during pumping the water level is at or above the top of the pervious stratum. Under these conditions the flow is "confined" or "artesian," because the head h at every point in the previous stratum will be at an elevation equal to or above the top of this stratum. In nature, these conditions are approximated, where a line of very closely spaced wells is installed near and parallel to the bank of the river, in which the pervious stratum is exposed. The relationships given for radial flow in confined aquifers can be used for the artesian condition.

10.2.4 Analysis of a System Consisting of a Series of Wells

Dewatering systems with multiple wells are sometimes analyzed by assuming the entire system acts as a single large well of radius r_s. This assumption is valid for a circular system of closely-spaced wells, as on ***Figure 10.3(a)***. Rectangular systems as on ***Figure 10.3(b)*** can also be assumed to act as a circular system of the same enclosed area:

$$r_s = \sqrt{\frac{ab}{\pi}} \qquad \text{(Equation 10-8)}$$

Some analysts prefer to consider a rectangular system to act as a circular system with the same perimeter:

$$r_s = \frac{a+b}{\pi} \qquad \text{(Equation 10-9)}$$

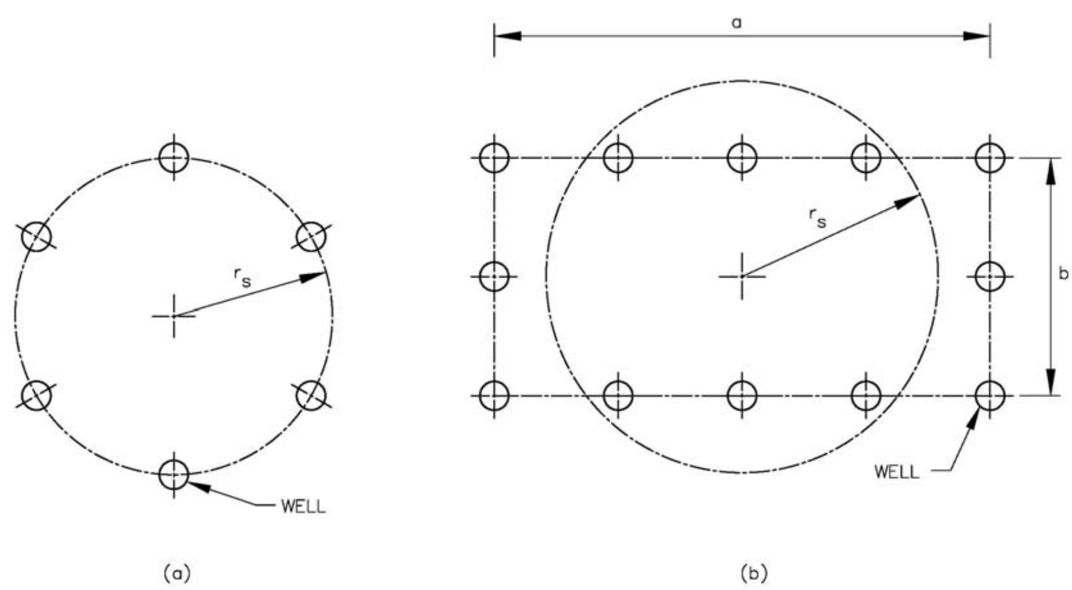


FIGURE 10-3
APPLICATION OF EQUIVALENT RADIUS CONCEPTS (A) CIRCULAR SYSTEM (B) RECTANGULAR SYSTEM
(Powers, et al., 2007)

The relationships above give reasonable approximations when the wells are spaced closely, when R_0 is great in relation to r_s, and when the ratio of a/b is less than about 1.5. If the wells are widely spaced, the actual Q will be significantly higher than the estimated for an equivalent well.

For a long narrow system, where the ratio of a/b is large, a combined analytical model can be constructed using equations for circular systems and rectangular systems. ***Figure10-4*** shows such a system of closely-spaced wells for dewatering a trench excavation of length *x*.

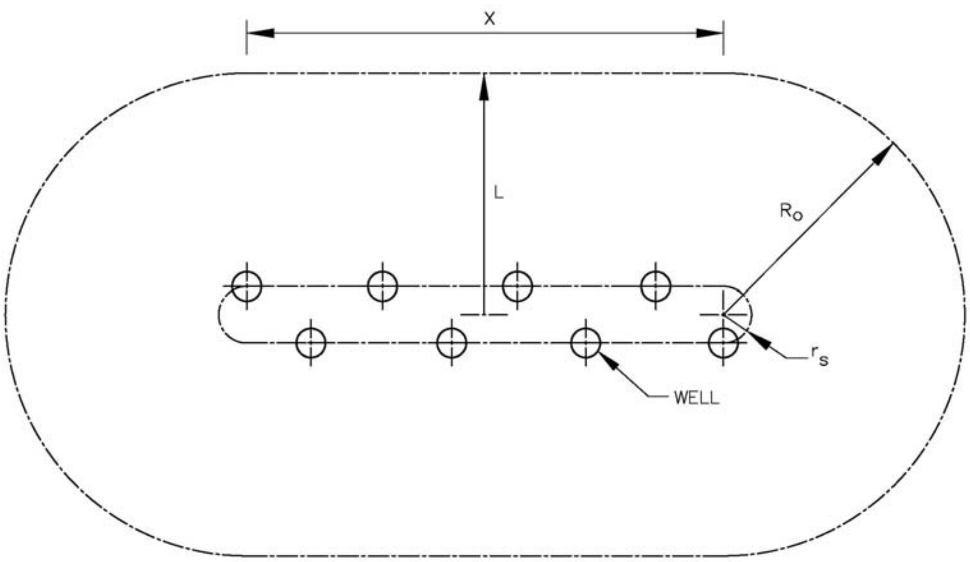


FIGURE 10-4
APPROXIMATE ANALYSIS OF LONG NARROW SYSTEM
(Powers, et al., 2007)

Analytical models, as described in this Section, for the groundwater flow system as a single aquifer or system of aquifers, and confining units generally involve certain simplifying assumptions; primary among these are the condition of a homogeneous and isotropic aquifer. This is necessary to simplify groundwater flow to a one or two dimensional problem. A single value of transmissivity is also usually employed in these analyses. Analytical models offer the advantage of ease of use and a relatively quick solution. However, because of their simplifying assumptions, analytical models may become unreliable when aquifer heterogeneity, anisotropy, or other complexities exist.

10.3 FLOW NET ANALYSIS

The use of graphical representations of flow through soil (flow nets) is of great assistance in designing certain types of dewatering and pressure relief well systems. Furthermore, it is generally easier to obtain flow net solutions rather than analytical solutions. The purpose of this Section is to discuss briefly the basic relationships to be maintained in a flow net and the equations for the seepage flow.

Flow nets may be constructed to represent the seepage problem in plan view, sectional view, or both, depending upon the needs of the Project. When the horizontal permeability, k_h, is different than the vertical permeability k_v of the soil, the flow net section must first be transformed in a ratio of $\sqrt{k_v/k_h}$ to represent equal permeability in both the directions. The flow nets then can be constructed following the general guidelines provided in geotechnical engineering text books, such as Das, 2002.

From the flow net, the discharge q per unit width and the head at any point can be determined from the following equations:

$$q = k\,(H-h_e)\,\frac{Nf}{Nd} \qquad \text{(Equation 10-10)}$$

$$h = (H-h_e)\,\frac{Nd}{N} \qquad \text{(Equation 10-11)}$$

where,
q = unit discharge
k = coefficient of permeability of the soil
N_f = number of flow channels in the net
N_d = number of equipotential drops between full head and head h_e at the point of flow exit
N = number of equipotential drops from exit to point at which head h is desired.

Several commercially available programs, such as SEEP/W, are also available, which can be used for constructing flow nets. Example output from SEEP/W is shown on ***Figure 10-5.***

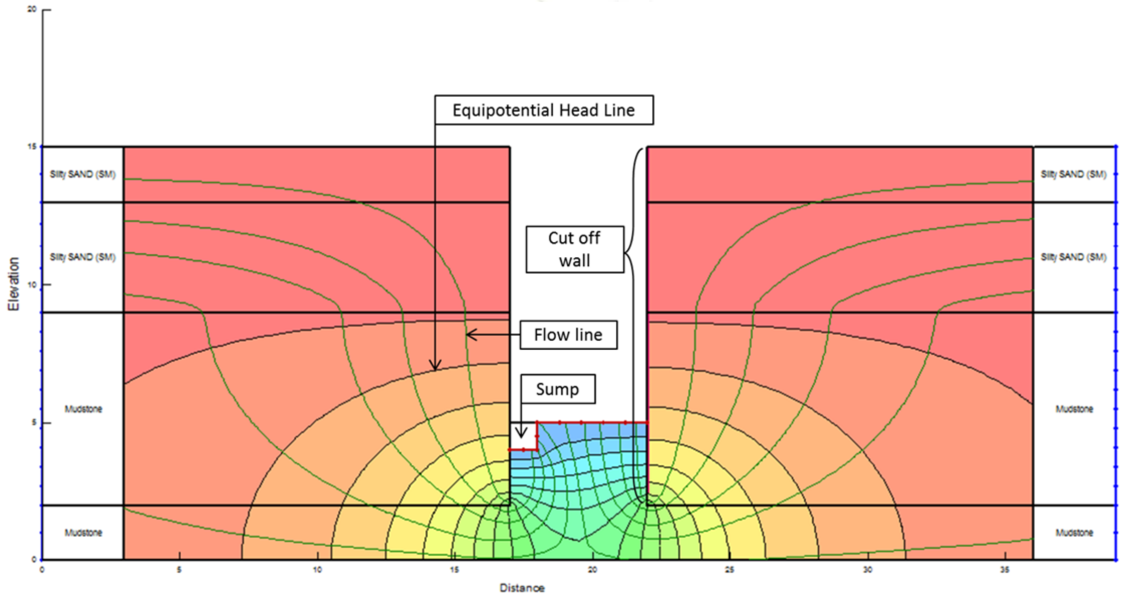


FIGURE 10-5
EXAMPLE FLOWNET ANALYSIS OUTPUT

Flow nets can be useful tools when designing dewatering systems, especially where complicated boundary conditions are present. However, flow nets are primarily used for two dimensional flow problems and can give erroneous results if used to analyze problems, which have important three dimensional variations. In ordinary practice today, flow nets may be used for rough preliminary assessments of a complex problem. For more complex problems, numerical groundwater modeling, as briefly described in the next Section, may be required to more accurately analyze the problem.

10.4 NUMERICAL MODELS

This Section briefly describes how numerical modeling can be used to achieve approximate solutions for dewatering problems. Numerical models can accommodate aquifer heterogeneity, anisotropy, complex and irregular boundary condition, and transient and steady-state flow simulations. Two dimensional or three dimensional, transient and steady-state, confined or unconfined models are possible that can consider both vertical and horizontal components of flow. The most frequently employed numerical models are finite difference and finite element models. Finite element methods offer certain inherent advantages, such as the ability to better simulate irregularly shaped or moving boundaries.

The following steps should be followed in model design and applications:

A. Justify the need and purpose

B. Development of the conceptual model

C. Development of the computer model

D. Verification and evaluation of the input data

E. Calibration of the model

F. Prediction and parametric analysis

Well-documented and extensively tested groundwater flow models, such as MODFLOW as developed by the U.S. Geological Survey, are available within the public domain, which can be used to model dewatering system design. Anderson and Woessner (1992) is also a good reference for groundwater modeling using numerical methods. Although groundwater modeling allows evaluation of complex problems in dewatering, it is not without limitations, such as:

1. Expert hydrogeologists are required to create the conceptual geologic and hydrogeologic model, to select appropriate input parameters, and to interpret the results and output.
2. A model is only an approximation of the real groundwater system.
3. High-powered mathematics and complex graphics do not make up for poor data or poor understanding of the dynamics of groundwater flow.
4. Models need to be calibrated and confirmed using field data.
5. A calibrated model is only one of a number of possible solutions to the given data.

Table 10-3 provides the recommended scope of dewatering analysis and design based on the level of hazard of the Project, the proximity of sensitive structures to the dewatering site, the depth of excavation, and the type of dewatering and excavation techniques selected. ADM initiated a project to identify the possible geological and hydrogeological risks and publish maps presenting the level of risk potential. These maps can be accessed by ADM's website.

TABLE 10-3
DEWATERING DESIGN SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	A	Analytical Solution
	B	Flow Net
	C	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required

TABLE 10-3
DEWATERING DESIGN SCOPE
(CONTINUED)

HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	PUMPING CAPACITY ANALYSIS[1]	SETTLEMENT ANALYSIS	THIRD PARTY REVIEW
C	1,2	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	c	I	X
	3	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	b	I	
B	1,2	i	Shallow	a	I	
		i, ii, iii	Medium	b	I	X
			Deep	c	II	X
	3	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	b	II	
A	1,2	i	Shallow	c	I	X
		ii, iii		c	I	X
		i, ii, iii	Medium	c	II	X
			Deep	c	II	X
	3	i	Shallow	a	I	
		ii, iii		a	I	
		i	Medium	b	I	X
		ii, iii		b	I	X
		i	Deep	c	II	X
		ii, iii		c	II	X

Notes:

[1] Simplified hand calculations are recommended when numerical models are developed. Hand calculations can provide useful checks of more advanced models.

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DEPARTMENT OF MUNICIPAL AFFAIRS

11.0 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations are performed on most projects to provide information on subsurface conditions for the design of the foundations, excavations, lateral load requirements, and other considerations. As discussed in ***Section 13.6*** and ***Appendix F***, starting January 2, 2014, all geotechnical investigations in Abu Dhabi City must be permitted.

A geotechnical investigation should also be designed to evaluate the groundwater conditions and potential need for dewatering. Foundation and groundwater investigations are sometimes considered separate entities; however, defining groundwater conditions are a key part of understanding subsurface conditions. Geotechnical investigations should generally be designed to evaluate the following:

1. Subsurface stratigraphy – Accurate classification of the materials encountered in the explorations, with care taken to identify the contacts, where materials change significantly. As discussed in other sections of these Guidelines, identification of these horizons is a critical aspect of the design of a dewatering program.

2. Pore pressures, including potential variations across the site and variations with time or changing conditions, such as precipitation.

3. Grain size and density of granular materials – Gradation and density can be used to estimate key parameters used in dewatering system design using published correlations.

4. Shear strength parameters and compressibility of fine grained or cohesive materials – These parameters may be determined using field or laboratory testing, and may be used to evaluate excavation stability, potential dewatering induced settlements, and other factors.

5. Obstructions – Existing utilities, fill materials, and other debris that may impact the performance or installation of the dewatering system.

6. Hydraulic Conductivity (Permeability) – The hydraulic conductivity of all the materials encountered in the exploration should be evaluated. These values may be determined based on correlations to density and material classification or based on field or lab testing programs on undisturbed or remolded samples.

7. Classification of aquifers – Including their location, extents, and depth; whether they are perched, unconfined, or confined; and where sources of recharge, such as bodies of water, utilities, and areas of concentrated runoff exist.

8. Precipitation information and topography to develop an understanding of expected site runoff conditions.

9. Environmental factors, such as groundwater or soil contamination, existence of wetlands, corrosivity of soils, etc.

General guidance on the number of borings and laboratory testing that may be required for projects is provided in ***Table 11-3***. This Table is provided as general guidance only and is not a substitute for engineering judgment and experience. Each project should be evaluated thoroughly on a case by case basis with a thorough understanding of the project requirements.

11.1 BOREHOLE DRILLING, LOGGING, IN-SITU TESTING, AND SAMPLING

The drilling of relatively small diameter geotechnical boreholes is the most common geotechnical exploration technique for most projects. Borings are most commonly drilled vertical; however, inclined borings may be required to develop a better understanding of site geology, especially in rock. The depth and inclination of the borings should be carefully selected prior to performance of the field investigation to ensure that the needs of the designer will be met and that adequate information on subsurface conditions is obtained. The use of standardized exploration plans that base the drilling quantities (depth and number of holes) on the total area to be developed, or the minimum number of borings required for permitting, is discouraged because subsurface conditions can vary widely even on small sites, and the amount and type of information required will vary with the type and complexity of the Project.

Numerous drilling techniques are available with availability typically dictated by local preference and subsurface conditions. The most common geotechnical drilling techniques in soil are the use of hollow stem augers (HSA), mud rotary drilling, and the use of cone penetration testing (CPT). Other methods are also used, such as the excavation of test pits, the use of bucket augers to drill larger diameter exploratory holes, and various geophysical techniques. With the exception of CPT drilling, these methods facilitate collection of a sample at depth and the performance of in-situ testing. Each of these methods has its own limitations and the drilling methods used should be carefully chosen. ***Table 11-1*** summarizes several common geotechnical exploration techniques.

TABLE 11-1
GEOTECHNICAL EXPLORATION TECHNIQUES

DRILLING TECHNIQUE	DESCRIPTION	LIMITATIONS OF TECHNIQUE
Wash Boring	Drill bit and water jet are used to advance boring, with cuttings lifted to top of hole by wash water. Samples collected in wash water at surface or by downhole method, such as the Standard Penetration Test (SPT).	Jet tends to disturb soil structure, removing fine particles and loosening soil matrix. Not recommended for sampling but suitable for quickly advancing holes for piezometers or other instrumentation.
Hollow Stem Auger	Hollow auger (outer) is rotated into ground with a leading bit (inner) plugging the hole in the lowest flight of augers. Samples collected by pulling the inner and inserting SPT or other sampler.	Effective above the water table, below the water table borehole instability is common. Difficult to advance augers through cobbles and boulders.
Wash Rotary (Mud Rotary)	Hollow drill rods advance a rotating bit with side oriented water/mud jets that help to cut the borehole walls and lift cuttings to the surface. Samples recovered by pulling drill rods from boring and dropping SPT or other sampler into boring. Steel casing or bentonite drilling mud (or both) typically used to maintain borehole stability.	Suited to a wide range of conditions; however, drilling can be very slow in soils with high gravel/cobble content.
Direct Push	Small diameter tools are hydraulically pushed into the ground to create a borehole. Tooling can be configured to allow for continuous sampling or SPT testing, but generally the probe is fitted with an instrumented cone tip for CPT testing.	Method is generally economical and quick. Penetrating hard/dense and gravelly soils can be difficult or impossible. In stiff soils the reaction force required to advance the probe can be very high, requiring additional anchoring of the rig.
Bucket Auger	Used to drill large diameter borings when advantageous, able to drill through large gravels and cobbles to obtain representative samples.	Method is expensive and no in-situ testing is possible.
Test Pit	Excavators or hand tools used to excavate pits for collection of bulk samples and direct observation/logging of stratigraphy.	Method can be expensive, depth limited by available equipment and excavation safety concerns.

Qualified drilling subcontractors should be experienced in the local geology and hydrogeology, and the proposed exploratory techniques. Details that may seem small at the time of drilling, such as drilling fluid pressure changes or water losses, the existence of thin cemented or gravelly lenses, and other incidental conditions can be very important to understanding the site hydrogeologic setting. To ensure a reliable record is produced, borings should be logged by a qualified geologist or engineer who has a thorough understanding of the objectives of the exploratory program, is knowledgeable about the proposed drilling and sampling techniques and local subsurface conditions, and who is capable of accurately logging the materials and groundwater conditions encountered in the borings. The borehole logger is responsible for maintaining and producing clear logs of the subsurface conditions and ensuring that proper drilling procedures and boring abandonment procedures are employed.

11.1.1 Closure of Boreholes

Upon completion of drilling and testing, the boreholes should be permanently abandoned. Use a standard cement-bentonite grout mixture to abandon the boreholes. This mixture can be as follows:

- Maximum of 60 liters (L) of water
- Per 100 kilogram (kg) of Portland cement
- No more than 5 percent by weight of bentonite powder to reduce shrinkage

The cement-bentonite grout will be placed from the bottom of each borehole to the ground surface by pumping through a tremie pipe. Boreholes will be filled with grout completely to the ground surface. Then surface casing will be removed and the grout will be re-filled up to the ground surface. Twenty-four hours later, additional grout will be added if necessary to account for grout shrinkage.

In-situ testing methods are performed in the field as the boring is advanced. These methods typically provide the most cost effective quantitative subsurface data, but are susceptible to poor data quality if they are not properly performed or if they are employed in unsuitable conditions. The most common in-situ test in geotechnical exploration programs is the SPT. SPT testing consists of driving a split barrel sampler 457 mm into the soil, with a 63.5 kg hammer allowed to drop 762 mm. The hammer may be operated manually with a rope and cathead or properly calibrated automatic hammers may be used. The number of hammer blows required to advance the sampler 30.48 cm is recorded (referred to as the N-value), and published correlation tables are used to estimate the in-situ parameters of the soil. CPT testing is another very common technique and can be much faster to perform than SPT sampling in the right soil conditions. However, because a sample is not recovered in CPT holes, often additional SPT boreholes are required to collect samples to correlate the CPT results and to obtain material for lab testing. ***Table 11-2*** summarizes common sampling and in-situ testing techniques and the materials, in which they are appropriate.

TABLE 11-2
TYPICAL DRILLING, SAMPLING, AND FIELD TESTING TECHNIQUES

Sampler Type or Field Test Method	Sample Type (Disturbed/ Undisturbed)	Penetration Achieved By	Appropriate Soil Types	Inappropriate Soil Types	Engineering Properties Determined based on Field Results (1)
Split barrel (split spoon)/ Standard Penetration Test (SPT)	Disturbed	Hammer Driven	Sands, silts, clays	Very gravelly/ cobbly or cemented soils, rock	D_r, ϕ, γ, c
Continuous Auger	Disturbed - Grab sample from auger	Rotation of auger	Cohesive soils	Granular soils	Lab testing
Bulk Sample (hand sample)	Disturbed - Grab sample collected from test pits or hand auger borings	Shovel, excavator, etc.	Any excavatable soil	None	Lab testing, typically grain size, Proctor testing
Block Sample (sample cut from soil mass)	Undisturbed - Sample carved from in-situ soil mass	Excavation and cutting	Cohesive, relatively stiff soils	Granular soils	Lab testing
Thin walled sampler (Shelby Tube)	Undisturbed	Pushed with drill rig	Clays, silts, clayey sands, fine grained/ cohesive soils	Granular soils, very hard or cemented soils	Lab testing – typically triaxial testing and consolidation testing
Continuous Push Sample	Partially disturbed	Pushed with drill rig, plastic sleeve inside of barrel collects sample	Sands, silts, clays	Gravelly/ cobbly soils, very hard or cemented soils	Lab testing

Note:

(1) D_r = relative density, ϕ = friction angle, c = cohesion, s_u = undrained shear strength, OCR = overconsolidation ration, E = elastic modulus

11.2 Piezometers and Monitoring Wells

The primary tools for understanding and monitoring groundwater conditions and dewatering system performance are piezometers and monitoring wells. Monitoring wells consist of a screened interval placed in a borehole open to the entire aquifer, and are suitable for relatively homogenous subsurface conditions, or for monitoring in shallow boreholes. Because they are under the influence of the full depth of the borehole, readings from monitoring wells that penetrate confining layers depict the average head between aquifer zones. This average head may over or under estimate the actual conditions in the zone of interest. Piezometers provide more specific information and consist of a screened interval isolated to a specific zone of the aquifer. Accurate monitoring systems may need to consist of multiple piezometers screened at different elevations and isolated from each other to develop an accurate understanding of site conditions. ***Figure 11-1*** shows a typical piezometer installation for measuring the head in isolated aquifer zones.

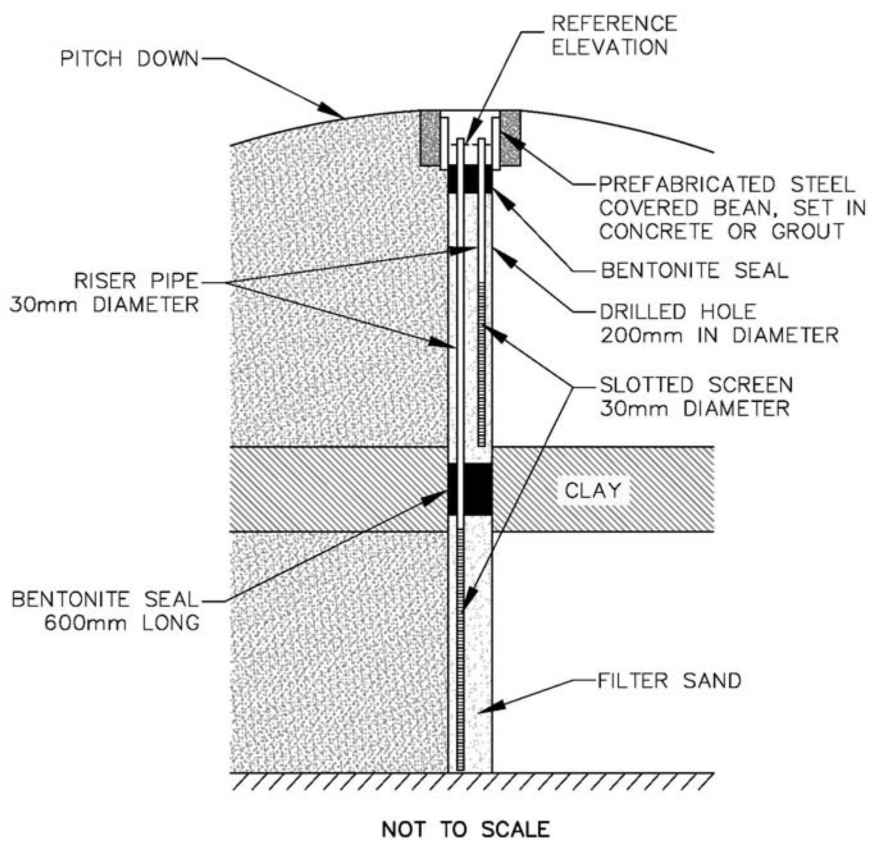

FIGURE 11-1
MULTIPLE PIEZOMETERS IN SINGLE BOREHOLE

Piezometers and monitoring wells are typically constructed as open standpipe instruments. The standpipes allow access to the water level in the piezometers for measurement with electric probes or in place sensors, and are typically protected at the surface by a metal enclosure surrounded by a concrete pad. The installation consists of a pipe provided with a screened interval at its end, inserted into the borehole at the desired elevation. The annulus between the screened interval and borehole wall is then filled with filter sand to some distance above the screen. If the screened interval is required to be isolated from the higher portions of the aquifer, the filter zone is sealed off using bentonite pellets and cement-bentonite grout to prevent migration of water in the vertical direction. In some cases, it may be desirable to then install other piezometers at higher intervals in the same hole. Successful installation of multiple standpipe piezometers requires very careful placement of the plugs and filters and often communication between the zones intended to be isolated occurs due to construction errors. For this reason, if site conditions permit, it is recommended that open standpipe piezometers installed at different elevations be installed in different holes. When automated readings are desirable, vibrating wire piezometers are often installed in the casing of open standpipe piezometers.

Because they rely on flow of water in and out of the screen, open standpipe piezometers are most effective in soils, which are at least moderately hydraulically conductive. The slow draining nature of fine grained soils tends to greatly increase the response time of standpipe instruments, which can limit the usefulness of the data in construction dewatering. In these cases, the use of vibrating wire piezometers to measure groundwater levels is appropriate. Vibrating wire instruments are extremely responsive and have the added benefit of being able to be easily connected to a datalogger, greatly increasing the number of readings that can be easily collected. Vibrating wire piezometers consist of a metal housing containing a diaphragm exposed to the water pressure in the surrounding material with a wire attached to the diaphragm and fixed at the other end. The wire is "plucked" by exciting a coil around the wire and the resulting frequency of the wire's vibration is measured. The frequency can be correlated to the pressure against the diaphragm and a resulting head calculated. Vibrating wire piezometers may be installed in sand filters similar to standpipe piezometers, or may be installed in boreholes completely backfilled with an appropriate cement-bentonite grout. Because the vibrating wire piezometer requires only infinitesimal changes to the water in contact with the diaphragm to register a change in pressure, microcracks in the cured grout allow sufficient horizontal conductivity to allow for effective measurements to be taken. The grouted option is preferable because it allows for multiple instruments to be installed in a single borehole easily and without concern for the quality of the seal between instruments. The vertical conductivity of the grout is orders of magnitude lower than the horizontal conductivity, so an effective separation of readings is maintained. ***Figure 11-2*** shows typical piezometer installations, including a Casagrande piezometer, an open standpipe piezometer, and a multistage vibrating wire piezometer installation using direct burial in grout.

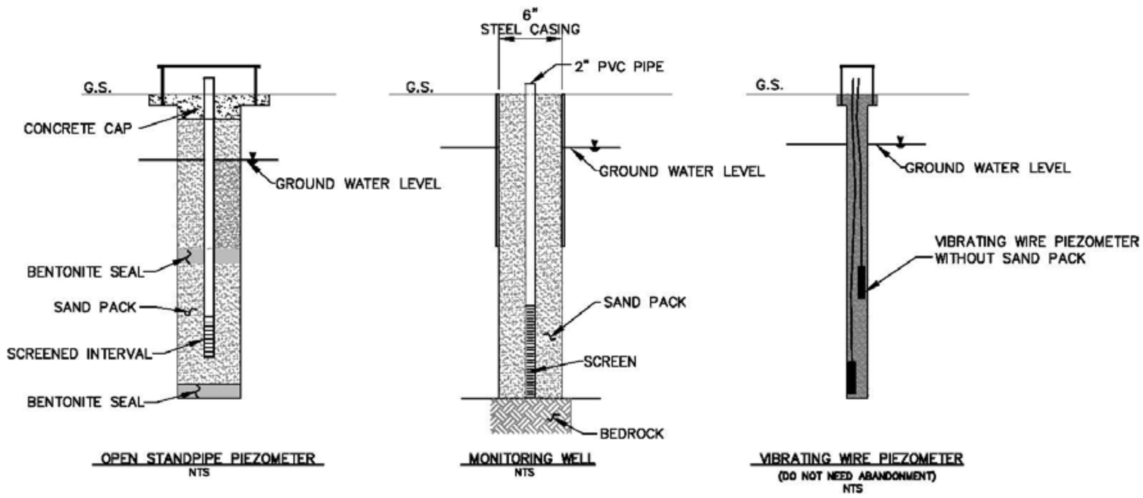

FIGURE 11-2
TYPICAL PIEZOMETER INSTALLATIONS

11.2.1 Monitoring Well Installation

The selection of the monitoring wells (MW) will be based on the Project specifics, including the depth of excavation to be dewatered and location of the Project. All Monitoring well Installation should be completed in accordance with ASTM D5092-04(2010)e1 (ASTM, 2010) or BS ISO 5667-22:2010. MW will be installed either above-ground or flush-mount. Above-ground wells are usually designed with a stick-up of approximately one meter above ground surface elevation. In heavily trafficked regions, or areas where a stick-up casing is impractical, it may be necessary to install a “flush mount” well. Flush mount wells have the top of the riser set just below ground surface in a water-proof well vault. When installing the flush mount well, the riser must be covered securely with a watertight cap.

When installing a well in consolidated bedrock, a wellscreen may not be necessary and the well can be completed as open hole. This is only possible when the bedrock in question is competent and little potential exists for cave-in of the well. Placing a well in unconsolidated materials (such as sand, gravel, or silt) requires a wellscreen.

The well depth, diameter, and monitored interval must be tailored to the specific monitoring needs of each investigation. For screened monitoring wells, materials placed within the annular space around the riser pipe generally include a filter pack, a filter pack seal, and a cement bentonite grout (the annular seal).

11.2.2 Monitoring Well Development

Following the well installation, well development should be conducted to stabilize and increase the permeability of the filter pack around the wellscreen, to remove fine-grained sediment from the well that may have entered during or after well construction, and to restore any of the porosity and permeability of the formation, which may have been reduced by drilling operations.

A monitoring well must be properly developed to provide representative data for the geologic unit being characterized. One of the following methods should be performed to develop the well:

- Surge and Pump
- Water (inertia) Pump
- Hand Bailing

No overpumping should be done during well developments. In cases where a well contains excessive sediment or water that is highly turbid, airlifting is an additional procedure that can be performed first to aid in well development. Airlifting is performed by injecting air under pressure into the bottom of the well.

Well development methods should remove impediments to the flow of water from the monitored formation to the monitoring well being developed. Each of the methods includes the measurement of volume of water being extracted from the well. This data is used to determine when a well is developed completely. Well development should be performed no sooner than 24 hours after completion of well installation to allow time for the cement-bentonite grout to harden.

During well development, the cumulative time and volume removed should be measured at least every 10 to 30 minutes. Development will continue until a minimum of five well volumes have been purged from the well. The well volume can be computed in liters (*l*) as:

$$\text{Volume (l): } (\pi\, r^2\, h)/1000 \qquad \text{(Equation 11-1)}$$

Where r is the radius of the well in centimeters (cm), and h is the height of the water column in cm.

11.2.3 Groundwater Level Measurements

During the groundwater level measurements in a monitoring well or borehole, either a pressure transducer or an electric water level indicator should be used. The measurements should be taken after three to five minutes to permit groundwater levels to reach equilibrium.

In case of using clear tubing attached to a flowing well (i.e., water level is above reference point) for instantaneous measurement of groundwater level, the technical should measure the stabilized height of the water column in the tubing (i.e., height above the reference point) with a ruler/measuring tape.

Depending on the Project size and duration, continuous measurement of water level using a pressure transducer should be performed in a non-flowing well (i.e., water level is below reference point). The transducer should be positioned in the water column within a depth range specified by the manufacturer and at least one meter above the bottom of the borehole.

11.3 Borehole Seepage Tests

Borehole seepage tests may be performed in borings or in completed monitoring wells or piezometers, and includes packer tests, rising head, falling head, constant head, and slug test methods. These test methods change the amount of water in the borehole and take measurements of the resulting response of the water level in the borehole to determine the hydraulic conductivity of the test interval. These tests are susceptible to error caused by contamination of the borehole walls with drilling mud or smearing of fine particles during drilling, but can provide very useful data when properly performed.

Falling head tests are performed by removing water from the borehole after the water level has stabilized. The rate of recharge is measured until the stabilized water level is reached again. Rising head tests are performed by adding water to the borehole after the water level has stabilized. The rate of drawdown is measured until it reaches the stabilized water level. Falling and rising head tests are suitable for soils with fairly low hydraulic conductivity. Highly permeable soils make collecting the readings needed for falling and rising head tests very difficult. When highly conductive soils are to be tested, the constant head test should be employed. Constant head permeability testing is performed by measuring the amount of water required to maintain a static water level in the borehole over time.

Slug testing is a modified version of the rising or falling head test. In slug testing, a sudden change of head in the boring is executed by quickly adding or removing a “slug” of water from the water column and measuring the response as the water table returns to its original level. Addition or removal of the slug may be performed with a solid cylinder rapidly withdrawn from the borehole, using compressed air to push the water level down in the borehole or by applying a vacuum to the borehole and suddenly releasing it.

Packer tests consist of isolating specific sections (usually 2 to 3 m) of an open bedrock borehole with inflatable packers (bladders) so that aquifer tests can be conducted in select intervals. A series of such tests allows definition of the vertical distribution of hydraulic conductivity in a bedrock aquifer. Monitoring water levels in nearby wells while performing a packer test can identify permeable intervals within the aquifer. The test is usually performed by measuring the flow rate of water entering the borehole under steady water pressure. Different flow rates are typically measured for two to four different water pressures. From these data, a hydraulic conductivity value can be calculated for the specific rock interval.

11.4 PUMPING TESTS

A properly designed and performed pump test provides the most reliable data on-site with hydrogeologic conditions. For projects where dewatering is expected to be a critical operation, pump testing provides a critical opportunity to refine the design of the dewatering system and reduce the risk of unforeseen circumstances during construction. A full scale pump test should be performed when:

1. Large quantities of water are expected to be pumped.
2. High flow conditions exist that may contribute to ground subsidence, damage to existing structures or utilities, or depletion of an aquifer.
3. Dewatering will be required to lower the groundwater table to near a difficult geologic interface with large changes in hydraulic conductivity (example, sand over rock).
4. In developed areas to help identify sources of concentrated flow, such as buried drains, abandoned, or leaky utilities, etc.

5. In industrial or developed areas to help identify contaminants and develop mitigation concepts.

6. Where permeable soils exists at excavation grade, which may require pressure relief to prevent boils or heave caused by artesian conditions.

7. To verify the continuity of aquitards and aquicludes when they are important to dewatering system design.

The purpose and objectives of the pumping test should be clearly understood prior to performance of the test. Typical objectives include:

1. Determination of various aquifer parameters, including transmissivity, radius of influence, and storage coefficient.

2. Measuring horizontal and vertical flow gradients.

3. Evaluation of installation methods, including type of drilling, material suitability, filter and screen size, and others.

4. The expected yield from a production well.

5. Unexpected conditions, such as artesian conditions, direct communication with sources of concentrated flow, and transient impacts, such as tidal fluctuations.

6. Evaluation of the effect of aquifer anisotropy (K_h/K_v) on pumping performance.

The pumping test system consists of the pumping well and a piezometer array arranged at varying distances from the pumping well. To ensure quality data, the pumping well and piezometers should be installed and monitored with the following best practices.

The pumping well is typically a deep well with an electric submersible pump. In cases where the dewatering to be performed will be shallow and within the limitations of suction lift, the deep well may be replaced with a series of wellpoints or a larger diameter suction well. The pumping well should:

1. Have sufficient capacity to stress the aquifer with a suitable constant flow rate. This may require a large capacity pump and high discharges in conditions with high hydraulic conductivity.

2. Be installed in a borehole of sufficient diameter to contain the well casing, wellscreen, and sufficient filter material.

3. Be provided with well casing and screen with the following characteristics:

   a. Casing and screen must be of sufficient diameter and structural strength to contain the pump and resist the loads induced by dewatering.

   b. Screen open area should be suitable for the expected flow rate and for retention of the filter material.

4. Be provided with a filter material that is as coarse as possible without allowing continuous pumping of fine particles.

5. Be provided with a piezometer in the filter zone outside the well casing to evaluate the loss of screen material and to provide water level information at the pumping well. For pumping tests performed with wellpoints or suction wells, this piezometer should be within 60 cm of the installation.

In simple aquifers, a piezometer array consisting of a single line of instruments spaced logarithmically that are installed in the same aquifer as the pumping well may be suitable for measuring drawdown trends during the pumping test. In more complicated hydrogeologic settings with multiple aquifers, aquicludes, and other features, additional instruments at various elevations may be necessary. The piezometer array should:

1. Be installed and monitored prior to the beginning of the pumping test to establish baseline trends.

2. Be spaced logarithmically from the pumping well, with the first instrument within 3-6 m of the well. The farthest instrument is typically installed around 30 percent of the expected radius of influence of the pumping well from the well.

3. Include one primary line of instruments with the screened interval in the same stratigraphic horizon, as the well is screened. These instruments will be used to develop the properties of the aquifer being pumped.

4. Consist of multiple lines of instruments when flow barriers or boundaries, recharge sources, and multiple aquifers exist or are suspected.

5. Include additional piezometers with the elevation of screened intervals targeted to the various aquifers or points of interest in addition to those on the primary piezometer section.

The pumping rate should be held constant and not allowed to deviate more than ten percent for the duration of the pumping test. The pumping rate should be selected, such that it is high enough to cause significant drawdown and so that the same pump rate can be maintained over the range of heads experienced during the pumping test. The pumping rate and total volume pumped should be monitored and recorded, preferably using a properly calibrated electronic water meter and datalogger.

The pumping test duration must be selected to ensure that the aquifer is adequately stressed, and preferably should run until the system reaches equilibrium while pumping at a constant rate. Reaching equilibrium is not always feasible; however, pumping tests should, at a minimum, run for 24 hours in confined aquifers and for three to seven days in a water table aquifer. Collected data should be plotted and interpreted during the performance of the test to allow for evaluation of the test method and the aquifer conditions. The use of electronic instruments and dataloggers will greatly reduce the level of effort required to plot and interpret the data. After the pumping test is performed, the recovery of the aquifer should be measured for not less than 60 percent of the time spent actively pumping.

Upon completion of the pumping test, logarithmic distance-drawdown plots depicting the radius of influence of the well should be developed and the results evaluated to determine the effects of boundary conditions, barrier boundaries, storage release, and to develop the aquifer parameters.

Table 11-3 provides the recommended scope of field investigation based on the level of hazard of the Project, the proximity of sensitive structures to the dewatering site, the depth of excavation, and the type of dewatering and excavation techniques selected.

TABLE 11-3
FIELD INVESTIGATION SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	no structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required

Field Investigation scope for guidance only. Building Law No. 4, Section 12.0 Geotechnical Investigation Regulations, specifies the minimum number and depth of boreholes. If following the above table yields fewer boreholes than those required by the Regulations, the Regulations will govern. In the area of Shakhbout City, a minimum of six boreholes per plot are required.

TABLE 11-3
FIELD INVESTIGATION SCOPE
(CONTINUED)

HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/DEWATERING TYPE	EXCAVATION DEPTH	BOREHOLE DISTRIBUTION (< 1000 m^2)	BOREHOLE DISTRIBUTION (< 10000 m^2)	VISUAL INSPECTIONS	LAB TESTING[3]	FIELD TESTING	THIRD PARTY REVIEW
C	1,2	i	Shallow	3[1]	5[1,2]	X	1[4]		
		i, ii, iii	Medium	3[1]	5[1,2]	X	1[4]	Slug Test	
			Deep	3[1]	5[1,2]	X	1[4]	Packer Test[5]	X
	3	i	Shallow	3[1]	5[1,2]	X	1[4]		
		i, ii, iii	Medium	3[1]	5[1,2]	X	1[4]	Slug Test	
			Deep	3[1]	5[1,2]	X	1[4]	Packer Test[5]	
B	1,2	i	Shallow	3[1]	5[1,2]	X	1[4]	Slug Test	
		i, ii, iii	Medium	3[1]	5[1,2]	X	1[4]	Packer Test[5]	X
			Deep	3[1]	5[1,2]	X	1[4]	Pumping Test	X
	3	i	Shallow	3[1]	5[1,2]	X	1[4]		
		i, ii, iii	Medium	3[1]	5[1,2]	X	1[4]	Slug Test	
			Deep	3[1]	5[1,2]	X	1[4]	Packer Test[5]	
A	1,2	i	Shallow	3[1]	5[1,2]	X	1[4]	Packer Test[5]	X
		ii, iii		3[1]	5[1,2]	X	1[4]	Packer Test[5]	X
		i, ii, iii	Medium	3[1]	5[1,2]	X	1[4]	Pumping Test	X
			Deep	3[1]	5[1,2]	X	1[4]	Pumping Test	X
	3	i	Shallow	3[1]	5[1,2]	X	1[4]	Slug Test	
		ii, iii		3[1]	5[1,2]	X	1[4]	Slug Test	
		i	Medium	3[1]	5[1,2]	X	1[4]	Packer Test[5]	X
		ii, iii		3[1]	5[1,2]	X	1[4]	Packer Test[5]	X
		i	Deep	3[1]	5[1,2]	X	1[4]	Pumping Test	X
		ii, iii		3[1]	5[1,2]	X	1[4]	Pumping Test	X

TABLE 11-3
FIELD INVESTIGATION SCOPE
(CONTINUED)

Notes:

[1] Two thirds of the boreholes should be up to 1.5 x depth of excavation and the remaining boreholes up to 2 x depth of excavation.

[2] One borehole each at the corners and one at approximate center location or at a spacing not exceeding 50 m c/c. For soil and ground water testing refer to ***Section 11***.

[3] Sieve analysis and Atterberg Limits (Soil Classification, e.g., USCS).

[4] One test per geologic layer (based on geologist's description) but no less than one test per 3 m of depth.

[5] Packer tests are performed in rock formations only. If not applicable, a slug test is recommended.

12.0 DEWATERING OPERATIONS

The successful operation of any dewatering system requires that it be properly installed, tested, maintained, and continuously monitored and evaluated. This Section discusses the installation and monitoring of dewatering applications.

12.1 INSTALLATION AND TESTING

Principal installation features of various types of dewatering or groundwater control systems are presented in ***Section 4.0*** of these Guidelines. The following paragraphs contain a brief summary regarding the suitability of the different dewatering installation methods.

12.2 WELL INSTALLATION

The various methods used for construction of dewatering wells are capable of producing wells with varying well losses or efficiencies. The most common well installation methods, generally in order of decreasing efficiency, are jetting, reverse circulation rotary drilling, dual rotary drilling, mud rotary drilling bucket auger drilling, and hollow stem auger drilling. ***Sections 4.0 and 11.0*** provide a summary of these methods of deep well installation.

The jetting method utilizes non-recirculated clean water, i.e., no drilling mud, so that a cleaner hole of superior quality is generally produced. Jetting is best suited for system requiring a substantial number of wells on a relatively close spacing.

The bucket auger method of drilling is very versatile and effective in sand and gravel with particle size up to 75 mm and soft to moderately stiff clay. It provides a good quality hole if clean water is used to prevent caving. Bentonite is not recommended because of the difficulty in removing the resulting mud cake from the side of the hole.

Rotary drills using circulating fluid to remove the cuttings from the hole are effective for holes of small to moderate diameter and almost to any depth. Mud or polymers are used for supporting the hole. Reverse circulation rotary drilling is sometimes used in dewatering. The flow is reverse direction from conventional drilling, hence the name. A minimum of about 3 m of the water head above the water table is recommended. Because of cost and difficulties associated with the reverse drilling, it is not widely used. However, it provides superior holes compared to conventional rotary drilling. Cased boreholes can be used where the ground conditions preclude other techniques or where the ground is highly permeable.

12.3 OPERATION

Details on operation of different systems are provided in this Section.

12.3.1 Wellpoints

- The proper performance of a wellpoint system requires continuous maintenance of a steady, high vacuum. After the system is installed, the header line and all joints should be tested for leaks by closing all swing-joint and pump suction valves, filling the header with water under a pressure of 70 to 100 kilopascal (kPa), and checking the line for leaks. The next step is to start the wellpoint pump with the pump suction valve closed. The vacuum should rise to a steady 63 to 67 cm of mercury. If the vacuum on the pump is less than this height, there may be air leakage or worn parts in the pump itself. If the vacuum at the pump is satisfactory, the gate valve on the suction side of the pump may be opened and the vacuum applied to the header, with the wellpoint swing-joint valves still closed. If the pump creates a steady vacuum of 63 cm or more in the line, the header line may be considered tight. The swing-joint valves are then opened and the vacuum is applied to the wellpoints. If a low, unsteady vacuum develops, leaks may be present in the wellpoint riser pipes, or the water table has been lowered to the screen in some wellpoints so that air is entering the system through one or more wellpoint screens. One method of eliminating air entering the system through the wellpoints is to use a riser pipe 7 m or more in length. If the soil formation requires the use of a shorter riser pipe, entry of air into the system can be prevented by partially closing the main valve between the pump and the header or by adjusting the valves in the swing connections until air entering the system is stopped. This method is commonly used for controlling air entry and is known as tuning the system; the pump operator should do this daily.
- A wellpoint leaking air will frequently cause an audible throbbing or bumping in the swing-joint connection, which can be felt at the swing joint. The throbbing or bumping is caused by intermittent charges of water hitting the elbow at the top of the riser pipe. In warm weather, wellpoints that are functioning properly feel cool and will sweat due to condensation in a humid atmosphere. A wellpoint that is not sweating or that feels warm may be drawing air through the ground, or it may be clogged and not functioning. Likewise, in very cold weather, properly functioning wellpoints will feel warm to the touch of the hand compared with the temperature of the atmosphere. Vacuum wellpoints disconnected from the header pipe can admit air to the aquifer and may affect adjacent wellpoints. Disconnected vacuum wellpoints with riser pipes shorter than 7 m should be capped.

- Wellpoint headers, swing connections, and riser pipes should be protected from damage by construction equipment. Access roads should cross header lines with bridges over the header to prevent damage to the headers or riser connections and to provide access for tuning and operating the system.

12.3.2 Deep Wells

Optimum performance of a deep-well system requires continuous uninterrupted operation of all wells. If the pumps produce excessive drawdowns in the wells, it is preferable to regulate the flow from all of the wells to match the flow to the system, rather than reduce the number of wells operating, and thus create an uneven drawdown in the dewatered area. The discharge of the wells may be regulated by varying the pump speed or by varying the discharge pressure head by means of a gate valve installed in the discharge lines. Uncontrolled discharge of the wells may also produce excessive drawdowns within the well, causing undesirable surging and uneven performance of the pumps.

12.3.3 Pumps

Pumps, motors, and engines should always be operated and maintained in accordance with the manufacturer's directions. Standby pumps and power units in operating conditions should be provided for the system. As discussed in ***Section 6.0***, the possibility of system breakdowns needs to be considered. Standby equipment may be required to operate during breakdown of a pumping unit or during periods of routine maintenance of the regular dewatering equipment. All standby equipment should be periodically operated to ensure that it is ready to function in event of a breakdown of the regular equipment. Automatic starters, clutches, and valves may be included in the standby system if the dewatering requirements so dictate. Signal lights or warning buzzers may be desirable to indicate, respectively, the operation or breakdown of a pumping unit. If control of the groundwater is critical to safety of the excavation or foundation, appropriate operating personnel should be on duty at all times. Where gravity flow conditions exist that allow the water table to be lowered an appreciable amount below the bottom of the excavation and the recovery of the water table is slow, the system may be pumped only part time, but this procedure is rarely possible or desirable. Such an operating procedure should not be attempted without first carefully observing the rate of rise of the groundwater table at critical locations in the excavations and analyzing the data with regard to existing soil formations and the status of the excavation. ***Table 12-1*** gives examples of pump types commonly used in dewatering operations (Powers et al., 2007).

TABLE 12-1
TYPICAL PUMPS USED IN DEWATERING
(Powers et al., 2007)

PUMP TYPE	POWER (hp)	POWER (kW)	FLOW RATE CAPACITY (gpm)	FLOW RATE CAPACITY (l/min)	TYPICAL APPLICATION	COMMENTS	ADVANTAGES	DISADVANTAGES
Contractor's submersible pump	Up to over 100	Up to over 74.6	Up to over 2000	Up to over 7570	Sumps, shallow wells		Environmentally safe oil; capable of handling large solids content	Sensitive to wear from sharp-grained sand; large diameter well casings and screens needed
Hydraulic submersible pump	Up to 1000	Up to 746	30 – 18000	115 - 68000		Must be powered by hydraulic power pack; sumping applications	Several thousand kPa of pressure; pumps fluids with considerable solids; do not require electrical distribution system	Efficiency is less than 50 percent
Turbine submersible pumps	Up to several hundred hp	Up to several hundred kW	Up to 1500	Up to 5678		Should not be installed in well until well is fully developed	Slender; Can be used in small-diameter wells; High efficiency (70% - 80%); environmentally safe oil	Rapid wear when handling abrasive sand
Vertical lineshaft pumps	Up to over 1000	Up to over 746	Up to over 10000	Up to over 37854	With turbine-type pump ends, used for moderate to high volumes and heads in deep wells and as vertical wellpoint pumps; with mixed flow and propeller type pump ends, they can be used to pump large volumes at low heads	Well must be plumb		

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TABLE 12-1
TYPICAL PUMPS USED IN DEWATERING
(Powers et. al, 2007)
CONTINUED)

PUMP TYPE	POWER (hp)	POWER (kW)	FLOW RATE CAPACITY (gpm)	FLOW RATE CAPACITY (l/min)	TYPICAL APPLICATION	COMMENTS	ADVANTAGES	DISADVANTAGES
Wellpoint pumps	20 to 250	15 to 185	Up to over 5000	Up to over 18930	Wellpoints	Employ a centrifugal unit to pump water, a vacuum unit to pump air, and a chamber with a float valve to separate the air from the water		Subject to cavitation
Jetting pumps			200 to 3000	800 to 12000	Installation of wellpoints, wells, sand drains, bearing piles, and steel sheet piling and other application requiring water under pressure	Pressures from 415 to 2275 kPa		

12.3.4 Surface Water Control

Ditches, dikes, sumps, and pumps for the control of surface water and the protection of dewatering pumps should be maintained throughout construction of the Project. Maintenance of ditches and sumps is of particular importance. Silting of ditches may cause overtopping of dikes and serious erosion of slopes that may clog the sumps and sump pumps. Failure of sump pumps may result in flooding of the dewatering equipment and complete breakdown of the system. Dikes around the top of an excavation to prevent the entry of surface water should be maintained to their design section and grade at all times. Any breaks in slope protection should be promptly repaired.

12.4 MONITORING, CONTROL, AND PERFORMANCE EVALUATION

After a dewatering or groundwater control system is installed, its performance and adequacy should be checked and monitored regularly. The initial groundwater or artesian water table, drawdown at critical locations in the excavation, flow from the system, elevation of the water level in the wells or vacuum at various points in the header, and distance to the “effective” source of seepage, should be measured if possible. These data should be analyzed, and if conditions at the time of test are different than those for which the system was designed, the data should be extrapolated to water levels and source of seepage assumed in design. It is important to evaluate the system as early as possible to determine its adequacy to meet full design requirements.

Testing a dewatering system and monitoring its performance require the installation of piezometers and the setting up of some means for measuring the flow from the system or wells. Pressure and vacuum gages should also be installed at the pumps and in the header lines. For multistage wellpoint systems, the installation and operation of the first stage of wellpoints may offer an opportunity to check the permeability of the pervious strata, radius of influence or distance to the source of seepage, and the head losses in the wellpoint system. Thus, from observations of the drawdown and discharge of the first stage of wellpoints, the adequacy of the design for lower stages may be checked to a degree.

General guidance on monitoring that may be required for projects is provided in ***Table 12-2***. Each project should be evaluated methodically on a case by case basis with a thorough understanding of the project requirements. The following monitoring equipment described in the following sections is generally recommended for successful dewatering operations.

TABLE 12-2
DEWATERING MONITORING SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	no structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Flow Measurement	X	Flow measurement required
		Flow measurement not required

TABLE 12-2
DEWATERING MONITORING SCOPE
(CONTINUED)

Hazard Zone	Proximity of Structures	Excavation/Dewatering Type	Excavation Depth	Water Level / Turbidity / TDS/ Electrical Conductivity	Survey/Deformation	Settlement	Slope Movement	Flow Measurement
C	1,2	i	Shallow					
		i, ii, iii	Medium	4 [1]	1 [2]	1 [2]	1 [2]	X
			Deep	4 [1]	1 [2]	1 [2]	1 [2]	X
	3	i	Shallow					
		i, ii, iii	Medium					
			Deep	4 [1]	1 [2]	1 [2]	1 [2]	X
B	1,2	i	Shallow					
		i, ii, iii	Medium	4 [1]	1 [2]	1 [2]	1 [2]	X
			Deep	4 [1]	1 [2]	1 [2]	1 [2]	X
	3	i	Shallow					
		i, ii, iii	Medium	4 [3]	1 [4]		1 [2]	X
			Deep	4 [3]	1 [4]		1 [2]	X
A	1,2	i	Shallow					
		ii, iii						
		i, ii, iii	Medium	4 [3]	1 [4]	1 [2]	1 [2]	X
			Deep	4 [3]	1 [4]	1 [2]	1 [2]	X
	3	i	Shallow					
		ii, iii						
		i	Medium	4 [3]	1 [4]		1 [2]	X
		ii, iii		4 [3]	1 [4]		1 [2]	X
		i	Deep	4 [3]	1 [4]		1 [2]	X
		ii, iii		4 [3]	1 [4]		1 [2]	X

TABLE 12-2
DEWATERING MONITORING SCOPE
(CONTINUED)

Notes:

[1] One piezometer each at the corners or at a spacing not exceeding 50 m c/c. In addition, one piezometer between wells to monitor drawdown rate at the crest of cone of depression.

[2] One instrument on or near to each sensitive structures/steep section of the slope.

[3] One piezometer each at the corners or at a spacing not exceeding 50 m c/c within 200 m of periphery of the excavation and at 100 m c/c between 200 m to cone of depression. In addition, one piezometer between wells to monitor drawdown rate at the crest of cone of depression.

[4] Install Survey Monument at a spacing not exceeding 50 m c/c within 200 m of periphery of the excavation and at 100 m c/c between 200 m to cone of depression.

12.4.1 Piezometers

Piezometer details and installation are discussed in detail in ***Section 11.2***. If required, the location of piezometers should be selected to produce a complete and reliable picture of the drawdown produced by the dewatering system. Piezometers should be located so they will clearly indicate whether water levels required by specifications are attained at significant locations. The number of piezometers depends on the size and configuration of the excavation and the dewatering system. If the pervious strata are stratified and artesian pressure exists beneath the excavation, piezometer tips, or screened intervals should be located in each significant stratum. Number of required piezometer depending on the project type and location are shown in ***Table 12-2***.

Piezometers should be installed at the edge of and outside the excavation area to determine the shape of the drawdown curve to the dewatering system and the effective source of seepage to be used in evaluating the adequacy of the system. If recharge of the aquifer near the dewatering system is required to prevent settlement of adjacent structures, control piezometers should be installed in these areas. Where the groundwater is likely to cause incrustation of wellscreens, piezometers may be installed at the outer edge of the filter and inside the wellscreen to monitor the head loss through the screen as time progresses. This way, if a significant increase in head loss is noted, cleaning and reconditioning of the screens should be undertaken to improve the efficiency of the system. Provisions for measuring the drawdown in the wells or at the line of wellpoints are desirable from both an operation and evaluation standpoint.

12.4.2 Flow Measurement

Measurement of flow from a dewatering system is desirable (required for to evaluate the performance of the system relative to design predictions). Flow measurements are also useful in recognizing any loss in efficiency of the system, due to incrustation or clogging of the wellpoints or wellscreens.

12.4.3 Settlement

If structures that are part of the Project or structures that are located nearby may be susceptible to damage due to settlement caused by the dewatering, settlement gages should be installed on or around the structure(s) for monitoring settlement and the structures should be visually inspected on a regular basis. The location of settlement gages should be selected to produce a complete and reliable picture of the settlement of the structure produced by the dewatering system. The number of settlement gages depends on the size and configuration of the structure and the excavation and the dewatering system.

12.4.4 Operational Record

Piezometers located within the excavated area should be observed at least once a day, or more frequently, if the situation demands, to ensure that the required drawdown is being maintained. Records should also be maintained to track trends with time. Vacuum and gages (revolutions per minute) on pumps and engines should be checked at least every few hours by the operator as he makes his rounds. Piezometers located outside the excavated area, discharge of the system, and settlement gages may be observed less frequently after the initial pumping test of the completed system is concluded. Piezometer readings, flow measurements, stages of nearby streams or the elevation of the surrounding groundwater, and the number of wells or wellpoints operating should be recorded and plotted throughout the operation of the dewatering system.

13.0 PERMIT APPLICATION PROCESS

ADM has a state-of-the-art internet-based Dewatering Permit Application System as a part of Construction Permitting System (commonly known as CDP). The details of how to start a permit application and getting the approval is explained in ***Sections 13.0 and 14.0***, respectively. The dewatering applications are divided into two main categories: building projects of any size and infrastructure projects, such as roads, pipelines, trenches, and any kind of excavation that will require discharge of water (which changes the water ground water regime). In this Section, first the internet-based permit system has been introduced and required documentation for application is summarized. The following sub-sections discuss the specific requirements for applying for dewatering permit for building and infrastructure projects.

13.1 ELECTRONIC PERMIT SYSTEM

The internet-based electronic permit system for both building and infrastructure projects can be accessed from ADM internet site http://www.adm.gov.ae, under "Construction and Infrastructure Permits" or specifically by using the following link: https://cdp001.dma.ae/cdpabudhabi/cdppublicportal/loginpage.aspx. ***Figure 13-1*** shows the components of the CDP. ***Figure 13-2*** shows the hierarchical structure of the CDP application.

In order to start the online application process, the applicant must register to the ADM website and then register with the CDP to obtain username and password. A comprehensive presentation on the details of the online dewatering permit application process is given in ***Appendix C***. The following sections summarize the overall procedure.

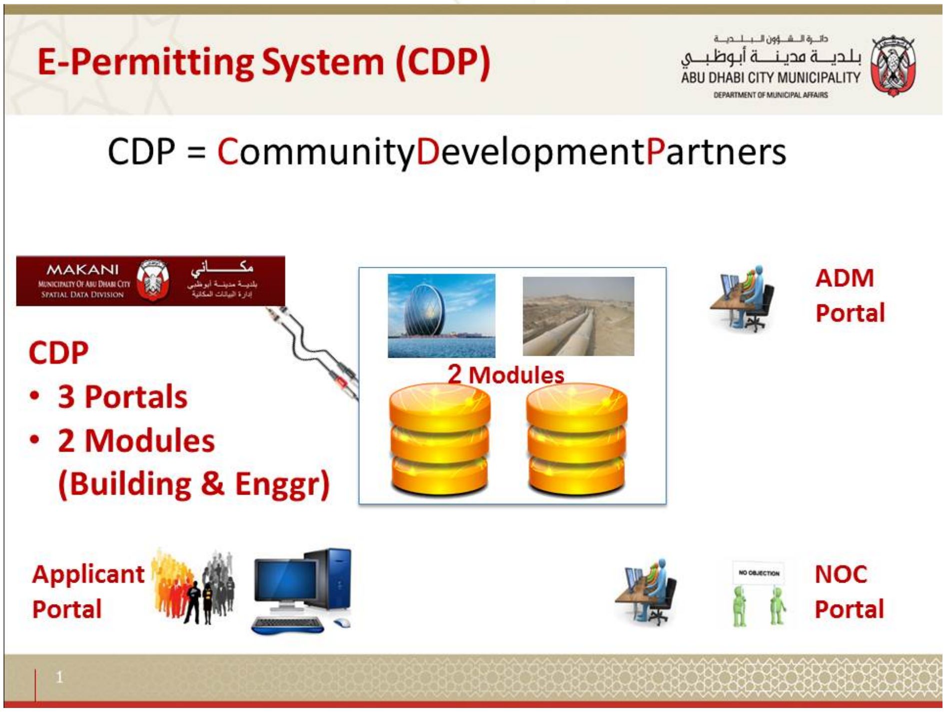

FIGURE 13-1
COMPONENTS OF ADM E-PERMIT SYSTEM (CDP)

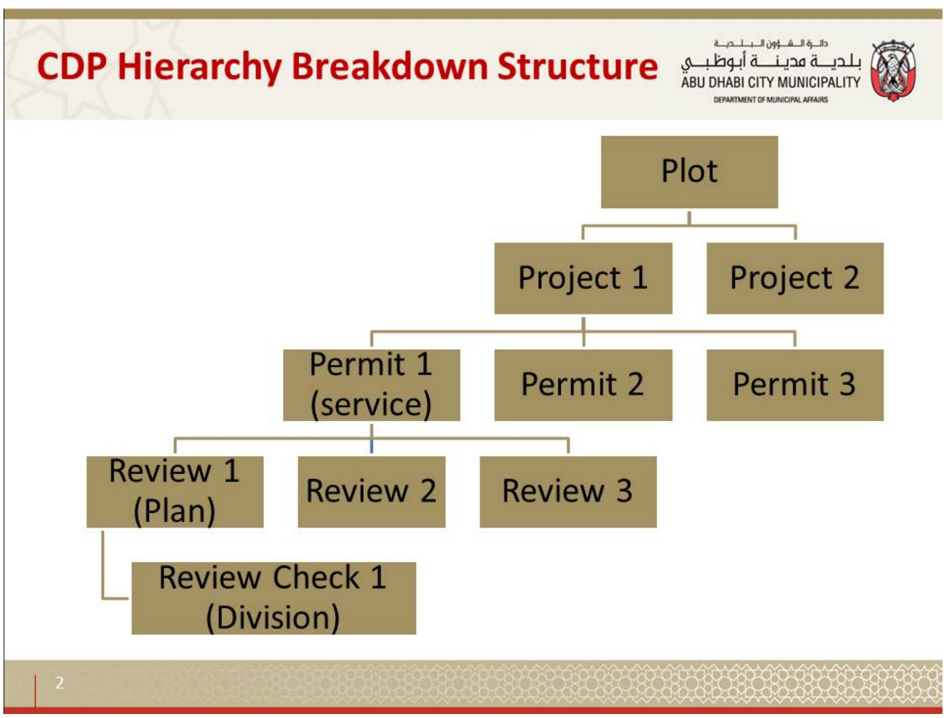

FIGURE 13-2
CDP HIERARCHY BREAKDOWN STRUCTURE

13.2 REQUIRED DOCUMENTATION

The first document to complete is the Dewatering permit application form (CDP-002), which is presented in ***Appendix B***. This is an electronic form (in PDF format) that includes general Project related information, such as Project Type, Location, and a check list to ensure the necessary documents are prepared and uploaded as per ADM's requirements. The form required to be signed and stamped by the Consultant and the Contractor of the Project. The form shall be uploaded together with necessary attachments. There are three main attachment categories:

- Geotechnical Report that meet the Dewatering Guidelines requirements.
- Dewatering Method Statement and other technical documents, such as design calculations, drawings, and monitoring plan.
- Health Safety and Environmental Division (HSE) Risk Assessment.

The following information is required in permit application documents at a minimum:

1. Permit Form
   a. Contractor and Subcontractor details.
   b. Introduction – Providing background, type, and location of the Project.
2. Geotechnical Report
   a. Geotechnical evaluation, including description of site conditions, subsurface conditions, and key geotechnical parameters, such as thickness and relative density of the soil cover, grain size analysis representing the whole soil section, the coefficient of permeability of the soil, and groundwater level.
3. Dewatering Method Statement
   a. Design calculations, including design of dewatering system for different load cases and evaluation of other potential impacts, including settlement, slope stability, and other impacts. Assumptions should be thoroughly documented. Some examples of the content are number of wells; calculation of the anticipated settlement and zone of influence performed by using standard references; slope stability calculations (if needed); a detailed calculation based on the permeability coefficient of the soil, showing rate of flow and quantity of water to be discharged and details of monitoring system design; and its schedule and template of the record workbook.

b. Construction Method – Providing purpose, scope, design basis, definitions, proposed structure, description, and method of dewatering and dewatering system, including method of installation, operation, maintenance, monitoring, and decommissioning. Following are some examples of the content: type of shoring system, depth and dimensions of the excavation, detailed layout drawings with coordinates and ground elevations, the protective measures taken to prevent damage to the existing structures, and duration of proposed dewatering work.

c. Resources – equipment and materials required for installation and operation of the dewatering system.

d. Organizational Chart and listing of the personnel and responsibility.

e. Quality Assurance and quality control management.

f. Attachments – Drawings (minimum plan and section view), calculations, and supporting geotechnical information, including boring logs, field, and laboratory test results.

4. HSE Risk Assessment

a. Evaluation of potential hazards.

b. Environmental Assessment.

c. Health and Safety – Provide general safety, contractor's safety procedure/manual, construction safety procedure and safety hazards, and precautionary provisions and phone numbers.

All application documents shall be reviwed and approved by an independent Third Party Engineering Conculting Company, which is qualified by the ADM for this purpose. Design drawings, calculations, and method statement shall be signed and stamped the by the third party reviewer before submitting to ADM.

Examples of dewatering application documents are presented in ***Appendices A and B***. The examples are categorized based on depth; ***Appendix C*** presents shallow dewatering examples, and ***Appendices D and E*** presents medium deep and deep dewatering projects, respectively.

13.3 APPLICATION STEPS FOR BUILDING PROJECTS

After preparation of all required documents in electronic format are ready for uploading, log-in to the system and select the project. This will bring the "Permits" tab on your screen.

After adding a new permit to the system, select “Dewatering” from the drop down list of “Permit.”

This will bring up the “Add New Permit to Project” window where a short description of the project can be entered. Necessary documents can be uploaded by selecting Plan ID corresponding to “Dewatering” title under “Plans” tab in the “Permit Details” window. The Permit Form, Geotechnical Report, and Dewatering Method Statement shall be uploaded from the attachment pop-up window that will appear when “submit/View A Plan” tab is selected.

Click the “List” tab in order to go back to the “Plans” tab to upload HSE Risk Assessment documents. HSE Risk Assessment documents shall be uploaded by selecting the “Plan ID” corresponding to “HSE-Risk Assessment” title “Plans” tab. The step by step application procedure detailed with screen views from the internet site is described under “Application Steps for Building Projects” section of the ***Appendix C.***

13.4 APPLICATION STEPS FOR INFRASTRUCTURE PROJECTS

Similarly, the details of application procedure for infrastructure projects are described under “Application Steps for Infrastructure Projects” section of the ***Appendix C.*** In order to start the process, log-in to the system and select the project after making sure that all required documents in electronic format is ready for uploading. Infrastructure project permit application starts by selecting “Request New Project” and “New Engineering Permit” menu items from the “My Dashboard,” respectively. This will bring the permit type section window. Select “7- Partial Work” item from the dropdown window.

The application form needs to be completed before proceeding with the rest of the process. Complete the requested information regarding to project details on the “Infrastructure Project Details” page. Project title, description, Sector, Plot, and Parcel ID are among the information needed to be completed. In addition, type of utilities, and its owner, consultant, and contractor of the project input are required at this stage of the application. Approximate project dimensions, nearest street to the project area, planned start and completion dates, total construction cost, and contract No. and contact mobile No. are to be filled. This information can be reviewed under “Engineering” Tab on the “Infrastructure Details” window following the successful submit by clicking the “Submit” button under “Review Application” section.

Project related documents can be uploaded by selection "Plans" tab on "Infrastructure Details" window. Click "Submit/View a Plan" tab in order to view the attachment pop-up window. Uploading the technical documents, such as Geotechnical Report and Dewatering Method Statement files, can be started by selecting "Plan ID" corresponding to "Dewatering NOC" title under the "Plans" tab.

After uploading the technical documents, click the "List" tab in order to go back to the "Plans" tab to upload Risk Assessment documents. Risk Assessment documents shall be uploaded by selecting the "Plan ID" corresponding to "Risk Ass. Infrs" title under the "Plans" tab. The step by step application procedure detailed with screen views from the internet site is described under "Application Steps for Infrastructure Projects" section of the ***Appendix C.***

13.5 PERMITTING OF SHORING SYSTEMS

13.5.1 Building Projects

Shoring systems are permitted after an application is submitted and accepted online for the entire project. Two add-on permits are available in the CDP:
A.3.2: Phased Permit – Excavation/Shoring: this permit can be obtained BEFORE the construction permit is issued for the entire project to expedite the construction.
A.6.1 Structural Shoring Design: when the application for the validation of shoring design is submitted before construction permit is issued.

Detailed requirements for these applications can be obtained from ADM website, Documents Center, Town Planning Sector, or Construction Permit Division.

13.5.2 Infrastructure Projects

The type of shoring systems for infrastructure projects is determined during the review process of the dewatering application. During the development period of this guideline, the technical design, stability, and performance of the excavation are entirely the responsibility of the consultant, contractor, and the infrastructure asset owner without governance by ADM.

13.6 PERMITTING OF GEOTECHNICAL INVESTIGATION

Starting January 2, 2014, all geotechnical investigations in Abu Dhabi City must be permitted by the Construction Permit Division. The requirements and application process for this permit are included in ***Appendix F***.

14.0 REVIEW AND APPROVAL PROCESS

In this Section of the Guidelines, the approval process will be explained in order to inform related parties and to avoid complications. In the first part of the Section, types of required approvals will be summarized. The next two sub-sections describe how to check the status of the application and how to obtain the permit. The field inspection procedure is described in the following sub-section. Non-compliance issues are presented at the end.

14.1 REVIEW PROCESS AND REQUIRED APPROVALS

In correct implementation of the dewatering process causes serious problems in the entire UAE, since engineering properties of soil and rocks, specifically in ADM region are sensitive to ground water changes. Rapid drawdown or frequent fluctuation of ground water level or loss of fine soil due to deawatering causes serious settlement and even structural damage of the nearby structures. These issues are amplified if natural ground has soluble zones, such as halite/salt layers or if manmande fill is not constructed properly. Therefore, ADM scrutinezes each dewatering permit application. Adequate time should be reserved for dewatering permit application and approval in the construction schedule.

All application documents shall be reviewed and approved by an independent Third Party Engineering Consulting Company, which is qualified by the ADM for this purpose. Design drawings, calculations, method statement shall be signed and stamped the by the third party reviewer before submitting to ADM.

The dewatering application review and approval process is managed by a reviewer who is an ADM Engineer. All the submitted documents will be reviewed by the ADM reviewer prior to approval of the permit. Members of Geotechnical Engineering Unit, Division of Health, Safety, and Environment and Town Planning Division review and approve parts of the application. Installation of instrumentation to monitor the dischrage of the water may be requested during the review process. In order to obtain the permit, follow these steps:

- Obtain approval from Third Party Reviewer
- Obtain approval from the CDP
- Obtain approval from the HSE
- Pay applicable fees
- Approval of Notice of Intent by Town Planning (for Infrastructure Projects only)

14.2 STATUS CHECKS

In order to check the status of the permit application, select “Project List” for building projects and “Engineering List” menu item for infrastructure projects under the “My Dashboard” menu list that will appear after logging-in to the CDP system. Status of the application will appear under the “Description” column of the project list.

If the status of the application indicates it is under review, it may have missing information. It may point out the corrections or additional information requested by the reviewer. The status may show the conclusion reached by the reviewer; the permit may be approved or rejected. Alternatively, the reviewer may request a meeting that may require attendance of several parties, such as the Consultant or the Owner of the project, which will be noted under status of the application.

The permit shall not be approved if the application is not in compliance with the engineering, HSE, Quality Assurance, and administrative requirements defined in these Guidelines and required by the reviewer.

14.3 OBTAINING THE PERMIT

Once the approval is issued, the permit can be printed from any customer services counter in ADM centers. The construction can only start after obtaining the permit. The consultant and the contractor are required to inform ADM on planned start and termination date of dewatering. It is also required to notify the ADM before the actual start and the termination dates.

Issuance of a permit shall not remove the responsibility of the consultant and contractor on the dewatering system. The consultant and contractor shall be responsible for any damage to existing structures and services in the vicinity of the excavation.

14.4 FIELD INSPECTION OF DEWATERING WORKS

ADM inspectors might perform a field inspection of the dewatering works in any stage of the construction. Although it is not required, ADM inspectors may give an advance notice before these inspections.

These inspections may focus on the compliance of constructed dewatering system with approved design and procedures. Type and dimension of shoring system, number of wellpoints, filter design, and depths of the wells are among the subject of the inspection. Discharge points, street crossings of dewatering pipes, and location of other equipment might be examined. Based on the number of working dewatering wells, fine particle content in the sediment tanks may be inspected. ADM inspectors may review the monitoring records. Water samples may be taken if needed.

14.5 NON-COMPLIANCE

Non-compliance with the dewatering guidelines and permit requirements shall be subject to penalty. If any non-compliance is observed during the field inspections, ADM has the right to suspend the construction activities and revoke the permit.

15.0 REFERENCES

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APPENDICES

APPENDIX A

APPLICATIONS FORMS

Application Form No. CDP-002
For Dewatering Approval

(For Use by Applicant. Date of form is 11 April 2012)

نموذج رقم CDP-002
لطلب موافقة على سحب المياه الجوفية

(لاستخدام مقدم الطلب. صدرهذا النموذج في ١١ أبريل ٢٠١٢)

رقم S/N	البند Item	إفادة مقدم الطلب Applicant Input			مراجعة الإدارة CPD Review
1.0	**General Information:**			**معلومات عامة:**	
1.1	نوع المشروع Project Type	☐	Residential or commercial structures or a Project Within a Private Plot	مباني أو فلل أو مشروع في ملكية خاصة	☐
		☐	Infrastructure	بنية تحتية	☐
1.2	رقم المشروع في النظام Project No. in CDP				☐

رقم S/N	البند Item	إفادة مقدم الطلب Applicant Input		مراجعة الإدارة CPD Review
2.0	**Location**			**الموقع**
2.1	موقع المشروع Project Location	Area		المنطقة
		Zone		الحوض
		Plot		القسيمة

رقم S/N	المستند Document	النوع Type	مكان التحميل Upload Location	مباني Building (Project within a Plot)	بنية تحتية Infrastructure Project (Project in Public Space)	تحقق مقدم الطلب Applicant's Compliance Check	مراجعة الإدارة CPD Review
3.0	**Submittal Requirements**						**الوثائق المطلوبة**
3.1	Application Form نموذج	PDF	وحدة الهندسة الجيوتكنيكية **Geotech.**	✔	✔	☐	☐
3.2	تقرير فحص التربة المعتمد من الاستشاري موضح فيه: – سماكة طبقات التربة. – التدرج الحبيبي للتربة وتصنيفها. – معامل نفاذية التربة. – منسوب المياه الجوفية Geotechnical Report Approved by Consultant showing as minimum - Subsurface Lithology - Soil Gradation - Hydraulic Conductivity of the various strata affected by the dewatering - Ground Water Level(s)	PDF		✔	✔	☐	☐
3.3	الطريقة المقترحة لسحب المياه ومتضمنة الآتي: Method of Dewatering including the following minimum requirement:	PDF		✔	✔	☐	☐
3.3.1	مخططات تفصيلية للموقع، تبين الإحداثيات ومناسيب الأرض والمياه الجوفية والحفريات. Detailed Plans of the site showing coordinates, Ground elevations, to a suitable scale.	PDF		✔	✔	☐	☐
3.3.2	مخططات تفصيلية تبين أبعاد الحفريات الأفقية والرأسية ونوع طبقات التربة على المخططات الرأسية. Detailed Plans showing Vertical and Horizontal Dimensions of the Excavation and type of substrata.	PDF		✔	✔	☐	☐
3.3.3	مخططات تفصيلية تبين نوع الدعائم الساندة وأبعادها الأفقية والرأسية ونوع طبقات التربة على المخططات الرأسية. Detailed plans showing Vertical and Horizontal Dimensions and type and detail of Soil Retaining System.	PDF		✔	✔	☐	☐

Application Form No. CDP-002
For Dewatering Approval

(For Use by Applicant. Date of form is 11 April 2012)

نموذج رقم CDP-002
لطلب موافقة على سحب المياه الجوفية

(لاستخدام مقدم الطلب. صدرهذا النموذج في ١١ أبريل ٢٠١٢)

رقم S/N	المستند Document	النوع Type	مكان التحميل Upload Location	مباني Building (Project within a Plot)	بنية تحتية Infrastructure Project (Project in Public Space)	تحقق مقدم الطلب Applicant's Compliance Check	مراجعة الإدارة CPD Review
3.3.4	مخططات تفصيلية تبين الطريقة المستخدمة لسحب المياه والتفاصيل التنفيذية لآبار السحب و/ أو نقاط السحب و/أو شفاطات السحب و تصميم الفلاتر لضمان عدم سحب المواد الناعمة. Detailed dewatering method with plans and sketches showing location and detail construction of wells, well points, and sumps filter design to avoid loss of fines.	PDF	وحدة الهندسة الجيوتكنيكية Geotech.	✔	✔	☐	☐
3.3.5	حسابات لمعامل نفاذية التربة ومعدل وكمية السحب الكلية استناداً إلى خصائص التربة بالموقع. Calculations for the hydraulic conductivity of the substrata and amount of discharge and radius of influence based on actual site conditions.	PDF		✔	✔	☐	☐
3.3.6	Duration of Dewatering. الفترة الكلية لعملية السحب.	PDF		✔	✔	☐	☐
3.3.7	حساب مقدار الهبوط المتوقع نتيجة لسحب المياه وتحديد مدى تأثيره في المحيط المجاور. Calculation of settlement due to dewatering within the zone of influence and its effects on adjacent buildings, utilities, roads, and infrastructure located within the zone of influence.	PDF		✔	✔	☐	☐
3.3.8	الوسائل المتبعة لضمان عدم حدوث ضرر بالمنشآت والبنية التحتية والطرق القائمة في نطاق تأثير عملية السحب. Methods that will be used to ensure no damage to buildings, utilities, roads, and infrastructure within the zone of the influence.	PDF		✔	✔	☐	☐
3.4.1	موافقة إدارة التخطيط الحضري وآخر تجديد (إذا كان المشروع غير مسجل في النظام) Approval Letter and all subsequent extensions from Urban Planning Division for infrastructure Projects Route (only if project is not registered in CDP)	PDF		NR	✔	☐	☐
3.4.2	مخطط المسار المعتمد من إدارة التخطيط الحضري (إذا كان المشروع غير مسجل في النظام) Approved Route by Urban Planning Division (only if project is not registered in CDP)	PDF		NR	✔	☐	☐
3.5	جدول تقييم المخاطر Environmental, Health and Safety Risk Assessment Matrix	PDF	إدارة البيئة والصحة والسلامة HES	✔	✔	☐	☐

Legend

✔	Required	O	Optional	NR	Not Required	CPD	Construction Permit Division

Application Form No. CDP-002 For Dewatering Approval

(For Use by Applicant. Date of form is 11 April 2012)

نموذج رقم CDP-002 لطلب موافقة على سحب المياه الجوفية

(لاستخدام مقدم الطلب. صدرهذا النموذج في ١١ أبريل ٢٠١٢)

4.0	Approval Conditions شروط الموافقة الصادرة عن الدائرة
4.1	أن لا ينتج عن سحب المياه الجوفية أية أضرار.
4.2	تقع على المقاول والاستشاري كامل المسؤولية عن صحة تقارير التربة وبيانات المياه الجوفية والمستندات الأخرى المرفقة بالطلب والتي سيتم العمل بموجبها.
4.3	البلدية غير مسؤولة عن أية أضرار من أي نوع تلحق بالأشخاص و/أو بالطرق والمنشآت والمباني والبنية التحتية وخطوط الخدمات والمرافق نتيجة هذه الأعمال.
4.4	يلتزم المقاول والاستشاري بتوفير جميع متطلبات الأمن والسلامة والحماية للسكان والمارة بالقرب من موقع الأعمال وذلك عن طريق وضع الأسيجة الآمنة والإضاءة والحراسة والعلامات التنبيهية والتحذيرية والإرشادية اللازمة حول مواقع الحفريات طوال فترة انجاز الأعمال
4.5	يلتزم المقاول والاستشاري بتوفير واتخاذ كافة الاحتياطات لحماية الطرق والمنشآت والمباني والبنيـــة التحتية وخطوط الخدمات والمرافق المجاورة للحفريات ضمن مجال تأثير السحـــب (Zone of Influence) و كافة ما يلزم من احتياطات لمنع سحب المواد الناعمة من طبقات التربة المجاورة وتطبيق كافة الضوابط والاشتراطات الفنية وكذلك المتعلقة بالبيئة والصحة والسلامة الإنشائية اللازمة والتأكد بصفة مستمرة من الالتزام بها لحين الانتهاء الرسمي من الأعمال.
4.6	يلتزم المقاول والاستشاري بالحصول على موافقة جميع الجهات المعنية على المخططات التنفيذية وإخطار بدء العمل قبل الشروع بالتنفيذ بحسب الإجراءات المتبعة لهذه الأعمال.
4.7	يؤكد كل من المقاول والاستشاري أن المخططات وكافة البيانات وتقارير التربة المقدمة منهما والتي سيتم العمل بموجبها صحيحة ومبنية على دراسات واختبارات وتحريات كافية وأن سحب المياه الجوفية بناء عليها لا يرتب أي أضرار بالتربة ولا المباني القائمة أو الطرق أو البنية التحتية وبأنهما يتحملان على سبيل التضامن كامل المسؤولية عن أية أضرار تنتج عن ذلك.

Declaration	إقــرار
I hereby declare that the information contained in this form and the attached documents, plans and drawings is true and correct in all material particulars and that we will fully comply with the approval conditions of Department of Municipal Affairs.	وبهذا أصرح بأن المعلومات الواردة في هذا النموذج والمستندات المرفقة والمخططات والرسومات حقيقية وصحيحة في جميع التفاصيل، وأننا سوف تمتثل امتثالا تاما لشروط الموافقة الصادرة عن دائرة الشؤون البلدية.

توقيع الاستشاري: Signature of the Consultant:		الختم الرسمي للاستشاري Official Stamp	
Date: التاريخ :			
توقيع المقاول: Signature of the Contractor:		الختم الرسمي للاستشاري Official Stamp	
Date: التاريخ :			

APPENDIX B

SUPPORTING APPLICATION GUIDANCE DOCUMENT

إجراءات تقديم رخصة سحب المياه عبر النظام الإلكتروني

Online Application for Dewatering Permit

Mun. of Abu Dhabi – Town Planning Sector	بلدية مدينة أبو ظبي – قطاع تخطيط المدن
Date: 25/06/2012	التاريخ: 25/06/2012
Duration (minutes): 25	مدة العرض (بالدقيقة): 25
Number of slides: 24	عدد الشرائح (صفحات): 24

Content المحتوى

- Introduction
- Required Approvals
- Required Documentation for Permitting
- Application steps for dewatering permits of Building Projects
- Application steps for dewatering permits of Infrastructure Projects

- مقدمة
- الموافقات المطلوبة
- المستندات والوثائق المطلوبة للترخيص
- خطوات التقديم على رخصة سحب المياه لمشاريع المباني
- خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

رخصـــة بنـــاء

التاريخ: 25/6/2012 **رقم المشروع:** 471112 **رقم الرخصة:** 22424

فئة الرخصة: بناء جديد

نوع الرخصة: سحب مياه جوفية

صاحب العمل السيد مالك القسيمة

طالب الترخيص: استشاري أو مقاول المشروع

Introduction مقدمة

- Application for dewatering permit can be done through the existing Permitting System commonly known as “CDP” for both building and infrastructure projects.
- The objective of this presentation is to demonstrate the steps to be followed for online application of dewatering permit.

- يمكن تقديم طلبات رخصة سحب المياه الجوفية لمشاريع البناء والبنية التحتية عبر النظام الإلكتروني لإدارة تراخيص البناء والمعروف بنظام ”CDP“.
- الهدف من هذا العرض توضيح إجراءات التقديم لهذه الرخصة عبر النظام الإلكتروني لأغراض المشاريع المختلفة.

رخصـــة بنـية تحتية

التاريخ:	25/6/2012	رقم المشروع:	471112	رقم الرخصة:	22424
فئة الرخصة:	تصاريح العمل الجزئي				
نوع الرخصة:	سحب مياه جوفية				
صاحب العمل	الشركة مالكة الأصول				
طالب الترخيص:	استشاري أو مقاول المشروع				

Required Approvals الموافقات المطلوبة

• Geotechnical Engineering Unit • Division of Health, Safety, and Environment • Permit is issued by Customer Service	• وحدة هندسة التربة • إدارة البيئة والصحة والسلامة • يصدر الترخيص من خدمة العملاء

رخصـــة بنـــاء

التاريخ: 25/6/2012 **رقم المشروع:** 471112 **رقم الرخصة:** 22424

فئة الرخصة: بناء جديد

نوع الرخصة: سحب مياه جوفية

صاحب العمل السيد مالك القسيمة

طالب الترخيص: استشاري أو مقاول المشروع

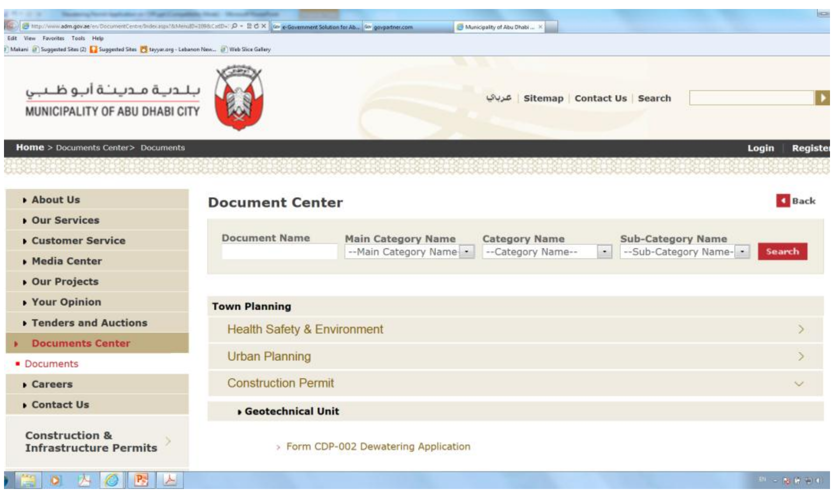
بـلـديـة مـديـنـة أبـو ظـبـي
MUNICIPALITY OF ABU DHABI CITY

المستندات المطلوبة للترخيص
Required Permitting Documentation

- The following is a link to the application form and the required documentation:

http://www.adm.gov.ae//En/DocumentCentre/PDF/582012100941337375 0_ADM_Dewatering%20Application%20Form.pdf

- Or Check ADM web site, Documents Center/Documents/Town Planning/Construction Permit



- يمكن الحصول على طلب الترخيص ومتطلباته عبر الرابط التالي:

http://www.adm.gov.ae//En/DocumentCentre/PDF/582012100941337375 0_ADM_Dewatering%20Application%20Form.pdf

- يرجى زيارة الموقع الإلكتروني لبلدية مدينة أبوظبي مركز الوثائق/الوثائق/تخطيط المدن

رقم S/N	البند Item	إفادة مقدم الطلب Applicant Input				مراجعة الإدارة CPD Review
1.0	General Information:					معلومات عامة:
1.1	نوع المشروع Project Type		Residential or commercial structures or a Project Within a Private Plot	مباني أو فلل أو مشروع في ملكية خاصة		
			Infrastructure	بنية تحتية		
1.2	رقم المشروع في النظام Project No. in CDP					

رقم S/N	البند Item	إفادة مقدم الطلب Applicant Input		مراجعة الإدارة CPD Review
2.0	Location			الموقع
2.1	موقع المشروع Project Location	Area		المنطقة
		Zone		الحوض
		Plot		القسيمة

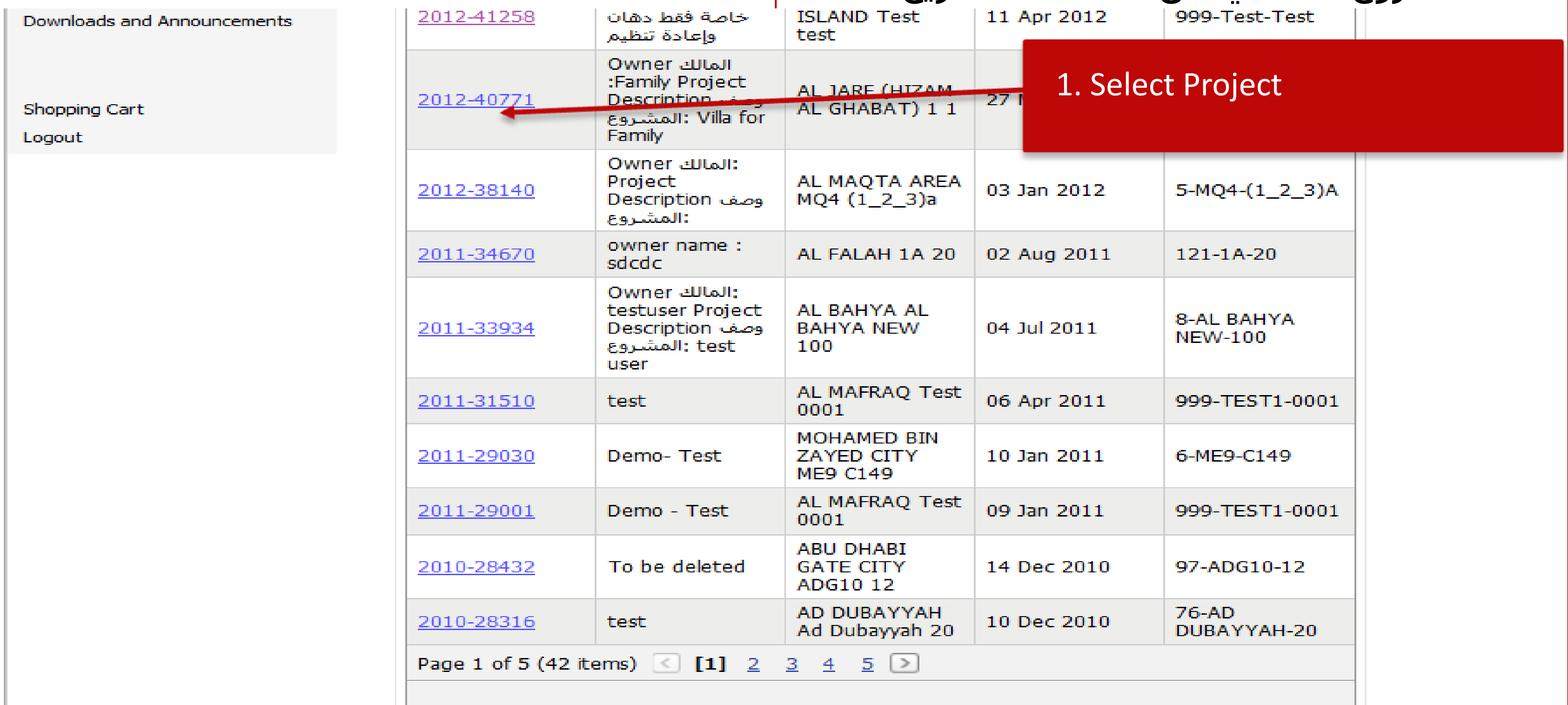
رقم S/N	المستند Document	النوع Type	مكان التحميل Upload Location	مباني Building (Project within a Plot)	بنية تحتية Infrastructure Project (Project in Public Space)	تحقق مقدم الطلب Applicant's Compliance Check	مراجعة الإدارة CPD Review
3.0	Submittal Requirements						الوثائق المطلوبة
3.1	Application Form نموذج	PDF		✔	✔		

بلدية مدينة أبو ظبي
MUNICIPALITY OF ABU DHABI CITY

خطوات التقديم على رخصة سحب المياه لمشاريع المباني
Application Steps for Building Projects

- Following log-in on the system and the preparation of all required submittals, select the project from your list of projects:

- بعد التسجيل وإدخال معلومات الدخول و استيفاء كافة المتطلبات يرجى اختيار المشروع المعني من لائحة المشاريع:



2012-41258	خاصة فقط دهان وإعادة تنظيم	ISLAND Test test	11 Apr 2012	999-Test-Test
2012-40771	Owner المالك :Family Project Description وصف المشروع: Villa for Family	AL JARF (HIZAM AL GHABAT) 1 1	27	
2012-38140	Owner المالك: Project Description وصف المشروع:	AL MAQTA AREA MQ4 (1_2_3)a	03 Jan 2012	5-MQ4-(1_2_3)A
2011-34670	owner name : sdcdc	AL FALAH 1A 20	02 Aug 2011	121-1A-20
2011-33934	Owner المالك: testuser Project Description وصف المشروع: test user	AL BAHYA AL BAHYA NEW 100	04 Jul 2011	8-AL BAHYA NEW-100
2011-31510	test	AL MAFRAQ Test 0001	06 Apr 2011	999-TEST1-0001
2011-29030	Demo- Test	MOHAMED BIN ZAYED CITY ME9 C149	10 Jan 2011	6-ME9-C149
2011-29001	Demo - Test	AL MAFRAQ Test 0001	09 Jan 2011	999-TEST1-0001
2010-28432	To be deleted	ABU DHABI GATE CITY ADG10 12	14 Dec 2010	97-ADG10-12
2010-28316	test	AD DUBAYYAH Ad Dubayyah 20	10 Dec 2010	76-AD DUBAYYAH-20

Page 1 of 5 (42 items) [1] 2 3 4 5

خطوات التقديم على رخصة سحب المياه لمشاريع المباني
Application Steps for Building Projects

- Select “Permits” Tab
- Press “Add New Permit”

- اختر ”الرخص“
- أضغط ”إضافة رخصة جديدة“

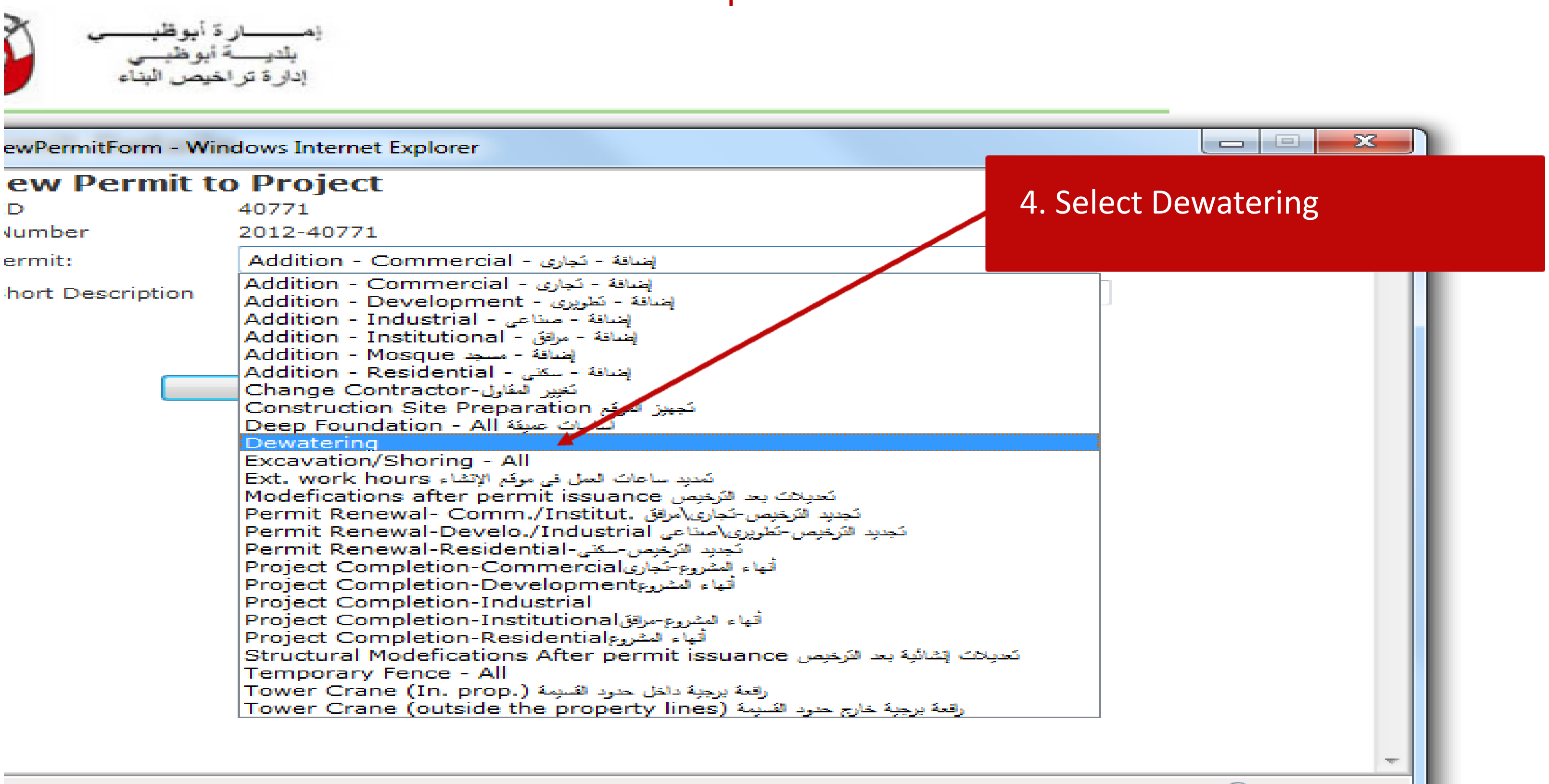
بلـديـة مـديـنـة أبـو ظـبـي
MUNICIPALITY OF ABU DHABI CITY

خطوات التقديم على رخصة سحب المياه لمشاريع المباني

Application Steps for Building Projects

- In the drop-down menu select “Dewatering”

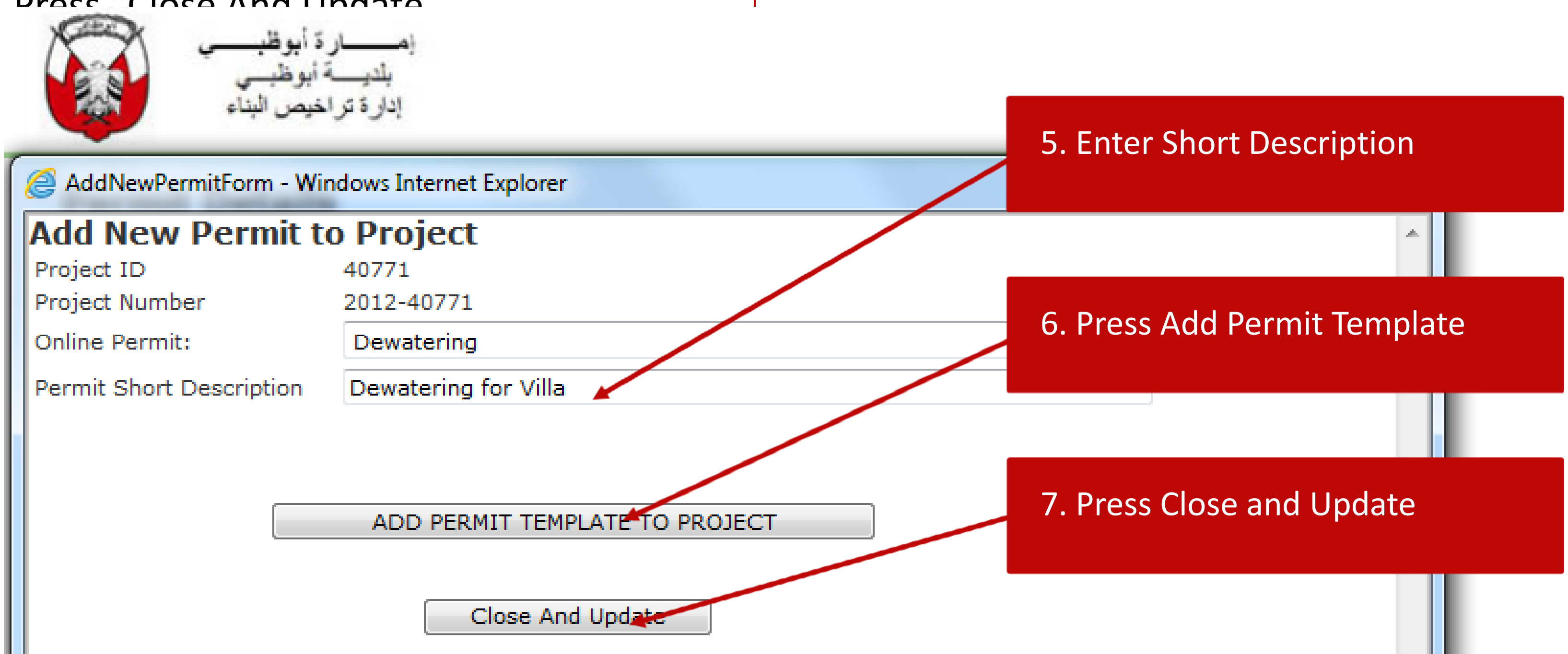
- من الاختيارات المتاحة أختر ”سحب المياه“



خطوات التقديم على رخصة سحب المياه لمشاريع المباني
Application Steps for Building Projects

- Enter Short Description
- Press “Add New Permit Template to Project”
- Press “Close And Update”

- أدخل وصف ملخص للرخصة
- أضغط على “أضف رخصة إلى مشروع”
- إضغط “إغلاق وتحديث”



خطوات التقديم على رخصة سحب المياه لمشاريع المباني
Application Steps for Building Projects

- Select “Plans”

- أختر ”الخطط“

إمـــــارة أبوظبـــي
بلديـــة أبوظبـــي
إدارة تراخيص البناء

Permit Details

Certificate of Completion

News: To get the Certificate of Completion, request from the coordinators to add a Project Completion permit if it's not on the list before requesting the final Final Inspection. Then, upload the required documents to Proj. Completion plan.
مستجد : للحصول على شهادة إتمام المنشأة، عليك طلب إضافة رخصة إتمام المشروع، عند عدم تواجدها، من المنسقين قبل طلب التفتيش النهائي تم رفع المستندات في النظام في قسم إنتهاء المنشأة

Project | Permits | Fees | Inspections | Plans

8. Select “Plans”

Project : 40771

Permit #	Permit Category	Permit Type	Short Desc	Status	Issue Date	Complete Date	
58547	Building	New Building بناء جديد	New Building بناء جديد	Applied For			
62377	Building	Dewatering		Applied For		21 May 2012	

Add New Permit

خطوات التقديم على رخصة سحب المياه لمشاريع المباني
Application Steps for Building Projects

- Press the “Dewatering” Plan ID

- اضغط على معرف خطة "سحب المياه"

Architectural Checklist-Residential

Click here to download the Spreadsheet submittal أضغط على الرابط للحصول على لائحة القواعد المعمارية للسكني

Project | Permits | Fees | Inspections | Plans

9. Press the “Dewatering” Plan ID

Plan ID	Title	Description	Receive Date	Due Date
500180	Arch. Resident.		28 Mar 2012	
500195	As Planned	Go to the Link "UPLOAD THE GIS DROWINGS " above to upload the drowings	28 Mar 2012	31 Mar 2012
500182	Civil D. Villa		28 Mar 2012	
500181	Civil D.Clr.New		28 Mar 2012	
521178	Dewatering		22 May 2012	
500183	Drainage-New		28 Mar 2012	
500184	Electricity-New	No Attachment	28 Mar 2012	
500185	Entrance- Resid		28 Mar 2012	31 Mar 2012

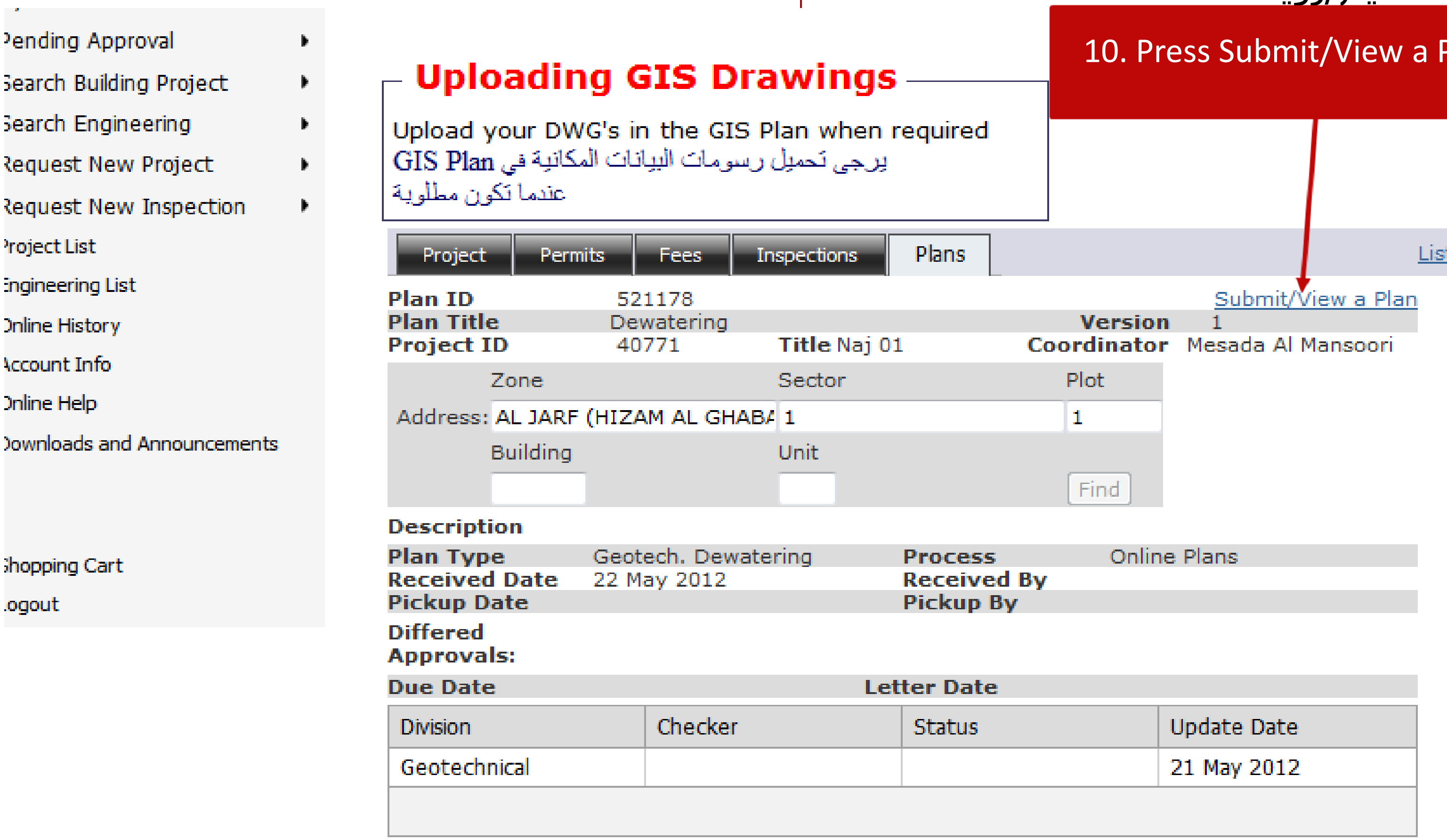
خطوات التقديم على رخصة سحب المياه لمشاريع المباني

Application Steps for Building Projects

- Press "Submit/View a Plan"

- أختر "تقديم/رؤية مخطط"

10. Press Submit/View a Plan

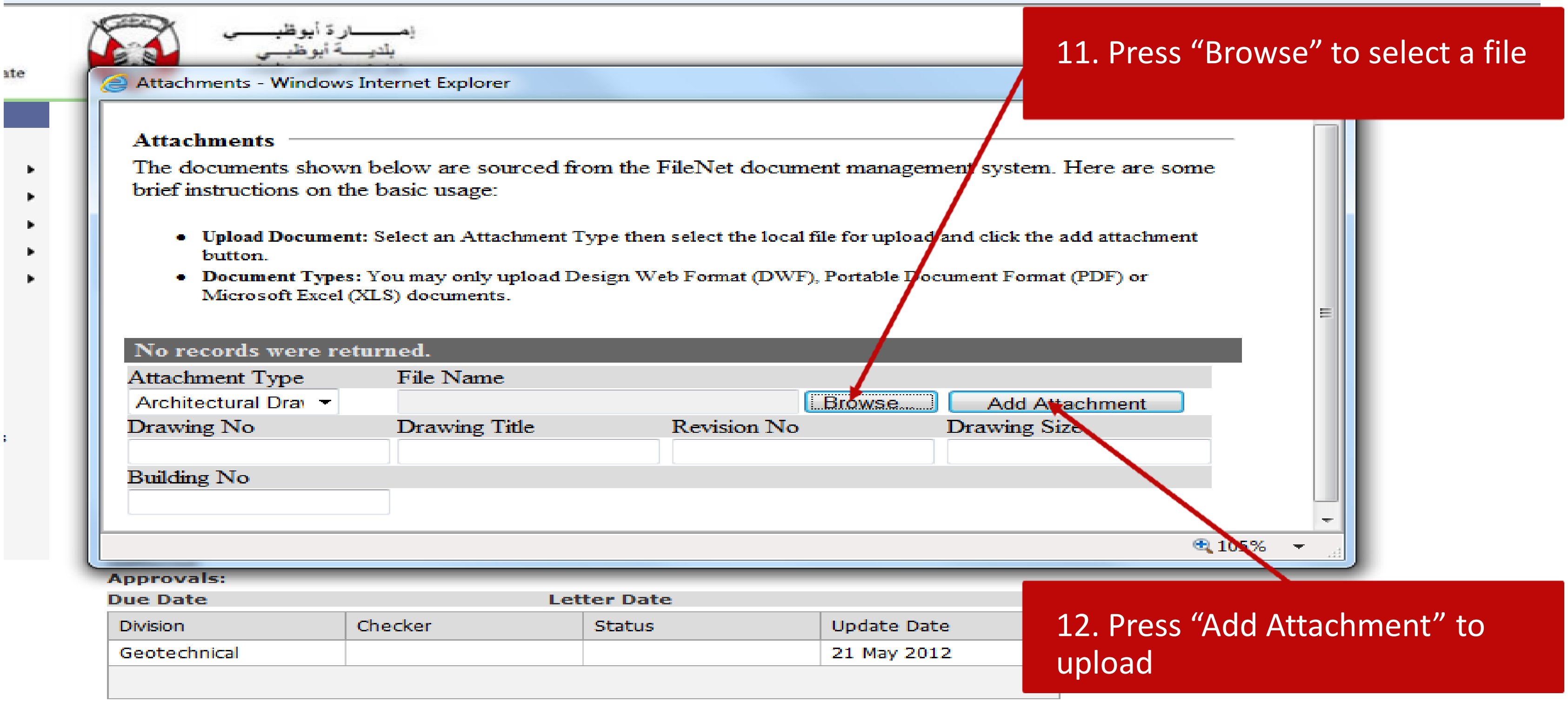


خطوات التقديم على رخصة سحب المياه لمشاريع المباني

Application Steps for Building Projects

- Press :"Browse" to Select file from your PC
- Press "Add Attachment" to upload the file

- اضغط "تصفح" لاختيار ملف
- اضغط "إضافة مرفق" لتحميل الملف



خطوات التقديم على رخصة سحب المياه لمشاريع المباني
Application Steps for Building Projects

• Repeat Steps 11 and 12 until all documents are uploaded • Use "HSE Plan" to upload the HSE Risk Assessment Matrix Hints: 1. Limit the size of each file to a maximum of a few MB (less than 5 MB). 2. Break up large files into smaller files, for example, break up the geotechnical report into: text, borehole logs, lab tests, etc. 3. Use file names that are relevant to the content of the attachment, for example, use Geo_Txt.pdf to refer to the attachment that contains the text of the geotechnical report, etc.	• كرر الخطوات 11 و 12 إلى أن يتم تحميل جميع المرفقات • يتم تحميل «جدول تحليل المخاطر في HSE Plan ملاحظات: 1. حدد حجم كل ملف بما لا يزيد عن عدد قليل من الميغابايت (5 أو أقل) 2. تقسيم الملفات الكبيرة على عدة ملفات أصغر، على سبيل المثال، تقسيم تقرير فحص التربة إلى: النص، وسجلات السبر، الفحوصات المخبرية، الخ. 3. استخدم أسماء ملفات التي لها صلة بمضمون المرفق، على سبيل المثال، استخدم Geo_Txt.pdf للإشارة إلى المرفق الذي يحتوي على نص تقرير فحص التربة، الخ.

بـلـديـة مـديـنـة أبـو ظـبـي
MUNICIPALITY OF ABU DHABI CITY

خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية
Application Steps for Infrastructure Projects

- Following log-in on the system and the preparation of all required submittals, place mouse on “Request New Project” and press “New Engineering Permit”

- بعد التسجيل وإدخال معلومات الدخول و استيفاء كافة المتطلبات أختر ”طلب مشروع جديد” ثم اضغط على ”رخصة بنية تحتية جديدة”

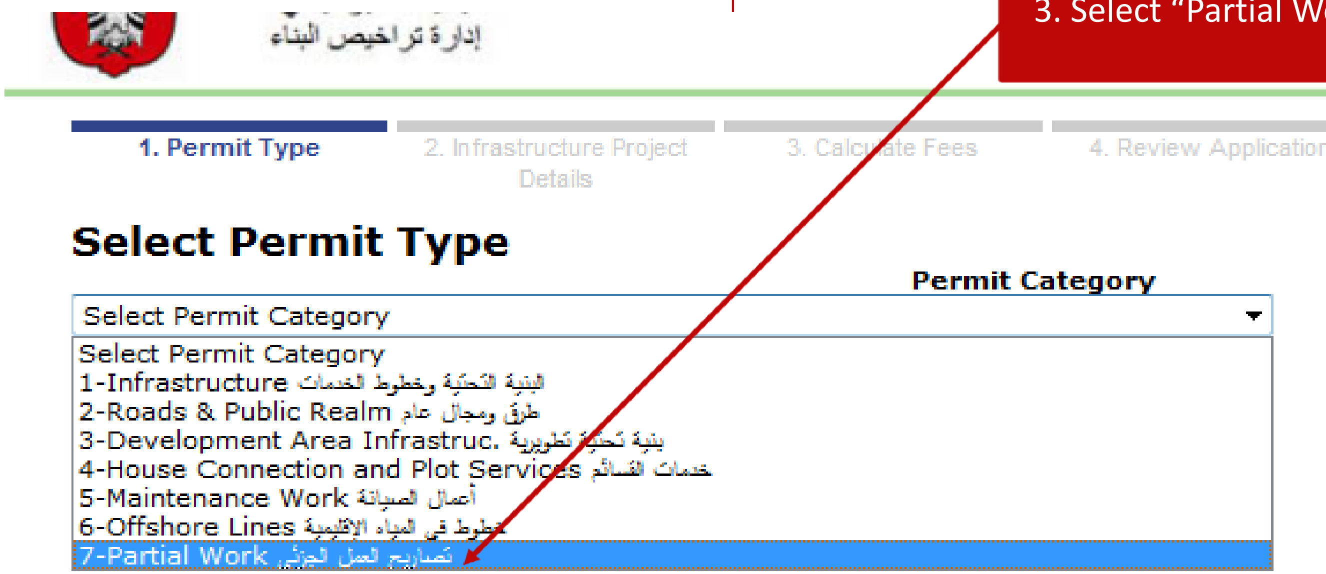
خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects

- From the drop-down menu for Permit Category select “Partial Work”

- من القائمة المنسدلة لفئة الترخيص أختر “تصاريح العمل الجزئي"

3. Select “Partial Work”

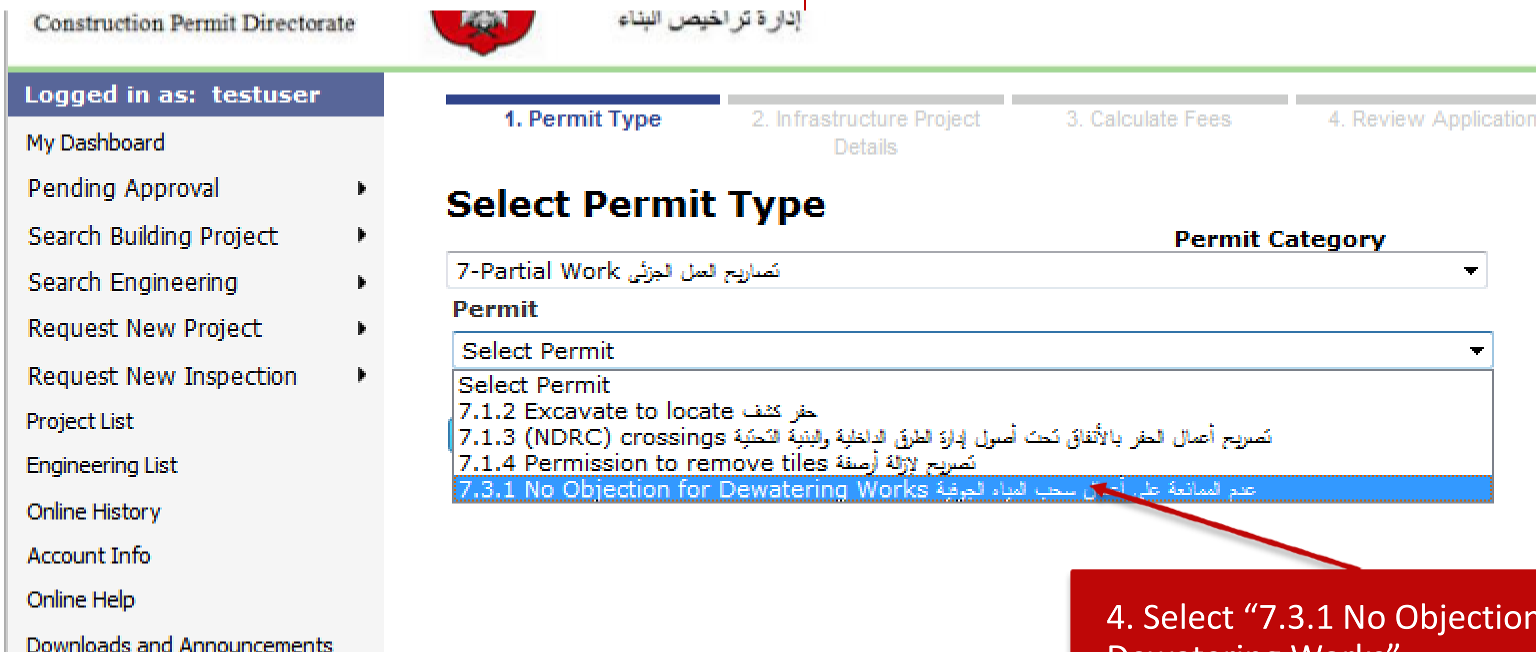


خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects

- From the drop-down of Permit type, select "7.3.1 No Objection for Dewatering Works" then press "next"

- من القائمة المنسدلة لنوع الترخيص أختر "عدم الممانعة على أعمال سحب المياه الجوفية" ثم اضغط على "next"



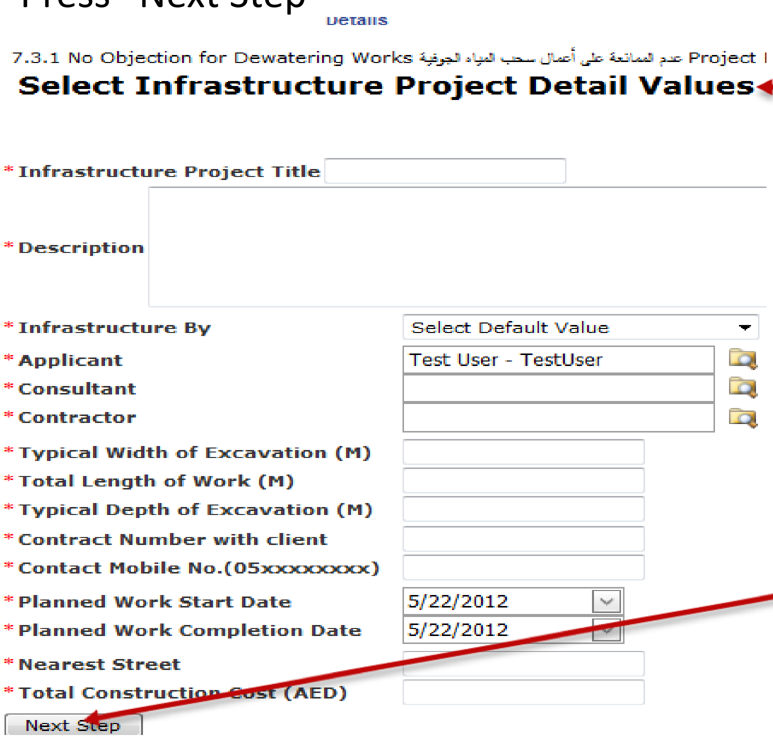
4. Select "7.3.1 No Objection for Dewatering Works"

خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects

- Fill-in the requested information. For explanation, see the next slide
- Press "Next Step"

- املأ المعلومات المطلوبة. ولشرح المطلوب يرجى مراجعة اللوحة القادمة
- اضغط "Next Step"

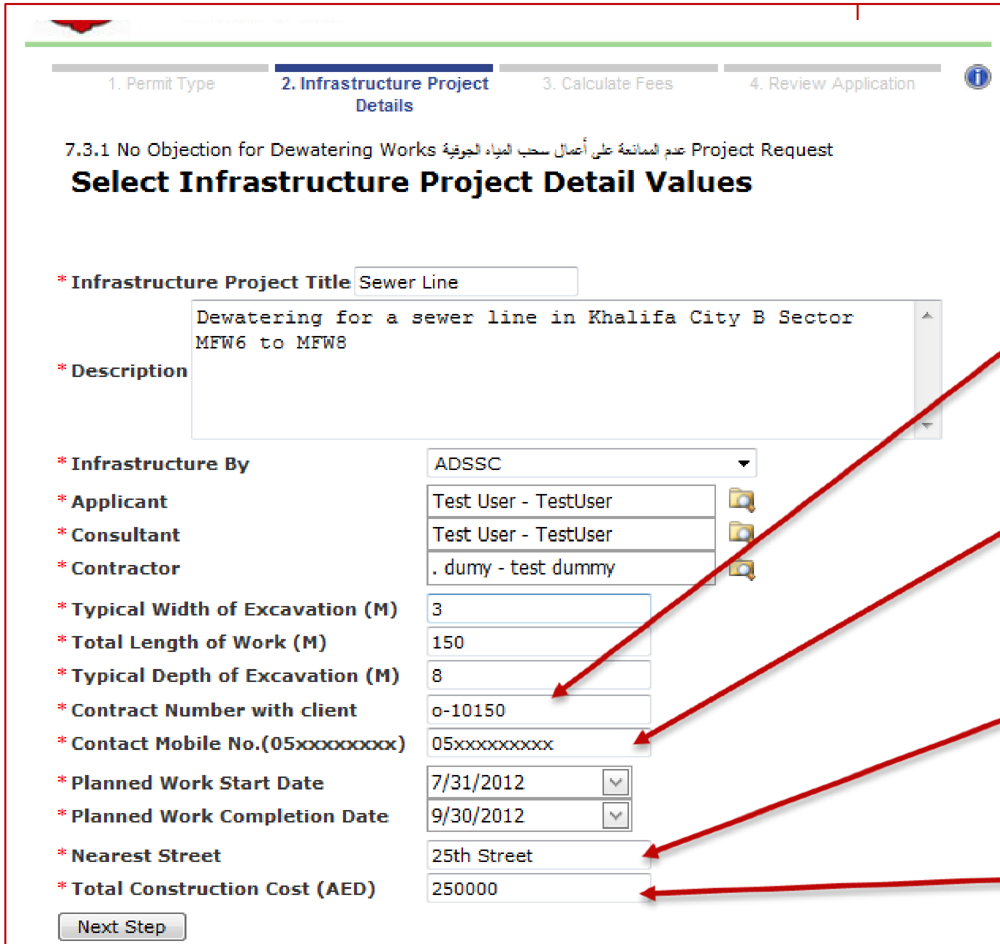


5. Fill-in the requested information

6. Press "Next Step"

خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects



1. Permit Type | 2. Infrastructure Project Details | 3. Calculate Fees | 4. Review Application

7.3.1 No Objection for Dewatering Works عدم الممانعة على أعمال سحب المياه الجوفية Project Request

Select Infrastructure Project Detail Values

* Infrastructure Project Title	Sewer Line
* Description	Dewatering for a sewer line in Khalifa City B Sector MFW6 to MFW8
* Infrastructure By	ADSSC
* Applicant	Test User - TestUser
* Consultant	Test User - TestUser
* Contractor	. dumy - test dummy
* Typical Width of Excavation (M)	3
* Total Length of Work (M)	150
* Typical Depth of Excavation (M)	8
* Contract Number with client	o-10150
* Contact Mobile No.(05xxxxxxxx)	05xxxxxxxxx
* Planned Work Start Date	7/31/2012
* Planned Work Completion Date	9/30/2012
* Nearest Street	25th Street
* Total Construction Cost (AED)	250000

Next Step

مثال للمعلومات المطلوبة

Example of a completed project details

For Main Contractor للمقاول الرئيسي

Contact number of a person that can answer questions

Nearest “Main” Street أقرب طريق رئيسي

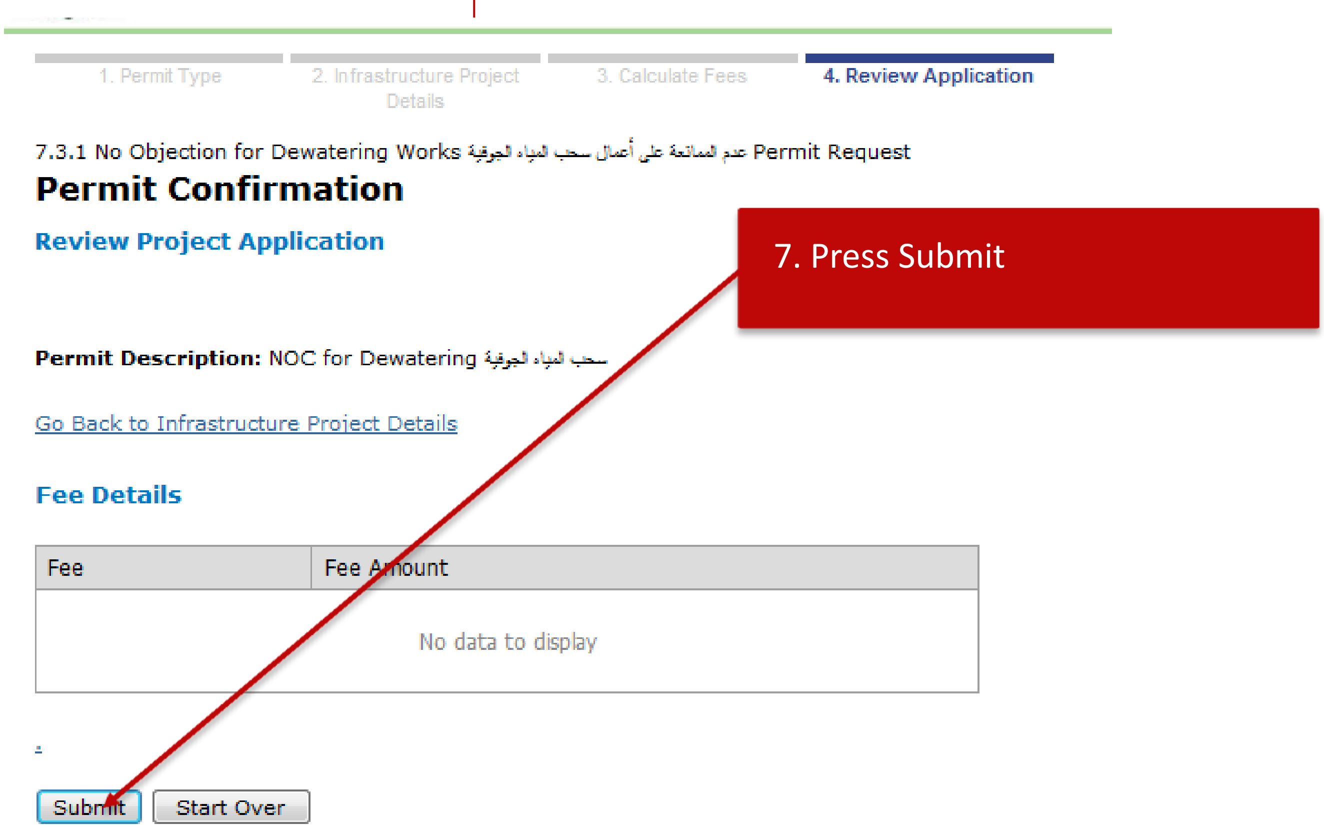
Cost of the dewatering only كلفة أعمال سحب المياه فقط

خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects

- Press “Submit”

- اضغط “Submit”



خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية
Application Steps for Infrastructure Projects

- Press “Plans”

- اضغط ”Plans“

8. Press “Plans”

Infrastructure Details

Engineering | Permits | Fees | Inspections | Plans

Infrastructure Info: **Project Number** 2012-23 **Title** Test

Description Dewatering Approval test

USDM Urban RD Type		**Utility Type**	
Infrastructure By	ADDC	**Public Realm Type**	
Consultant	Test Partners - Utilities Portal test	**Applicant**	Test User - TestUser
Contractor	Portal Test - PortalTest	**Engineer**	
Filing Date	30 Apr 2012	**Completed Date**	
Accepted to Apply Date		**Status**	On Line App
Typical Width of Excavation(M)	5.00	**Total Length of Work**	100.00
Total Depth Of Excavation (M)	7.00		
Property(KV,Dia.,Rd width...etc.)		**Nearest Street**	Salam
Total Construction			

خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects

- Press “Plan ID” for Dewatering NOC

- اضغط على معرف خطة “سحب المياه”"

9. Press the “Dewatering NOC” Plan ID

Plan List

Architectural Checklist-Residential

Click here to download the Spreadsheet submittal أضغط على الرابط للحصول على لائحة القواعد المعمارية للسكني

Engineering | Permits | Fees | Inspections | Plans

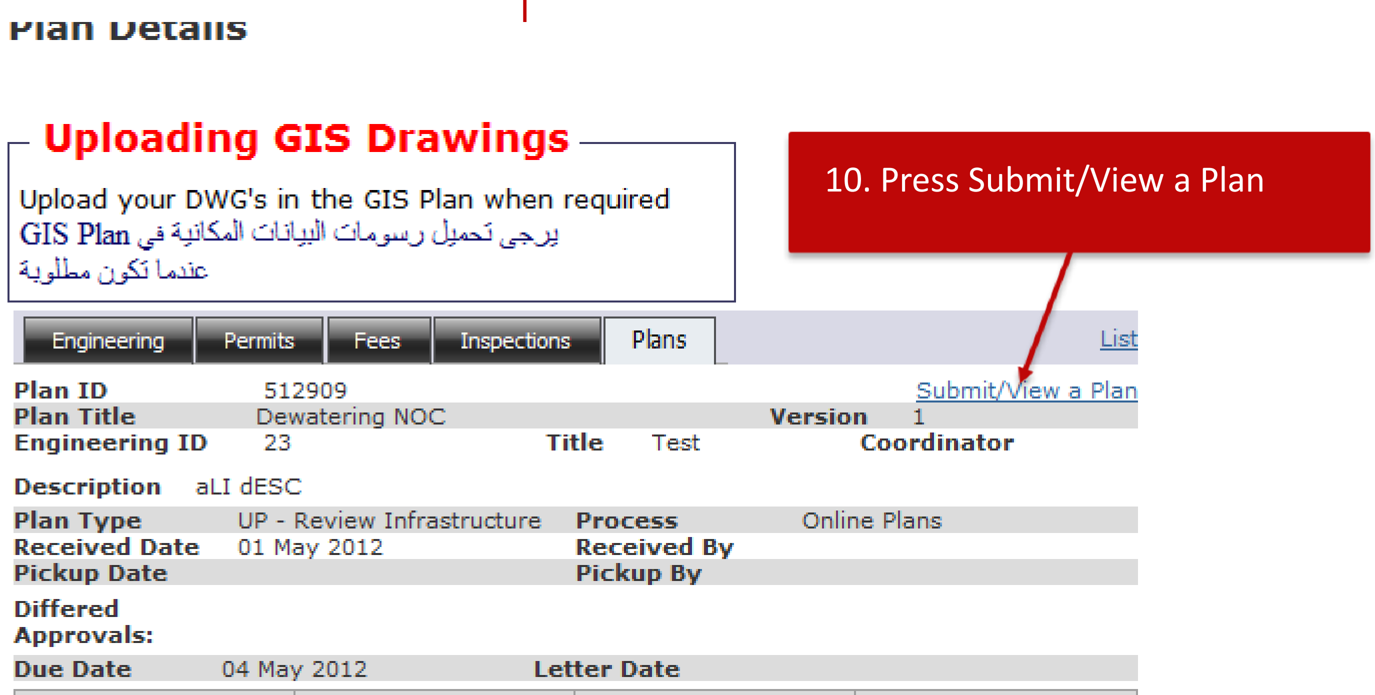
Plan ID	Title	Description	Receive Date	Due Date
512909	Dewatering NOC	aLI dESC	01 May 2012	04 May 2012
512910	Risk Ass. Infrs		04 May 2012	01 May 2012

خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects

- Press “Submit/View a Plan”

- أختر “تقديم/رؤية مخطط”

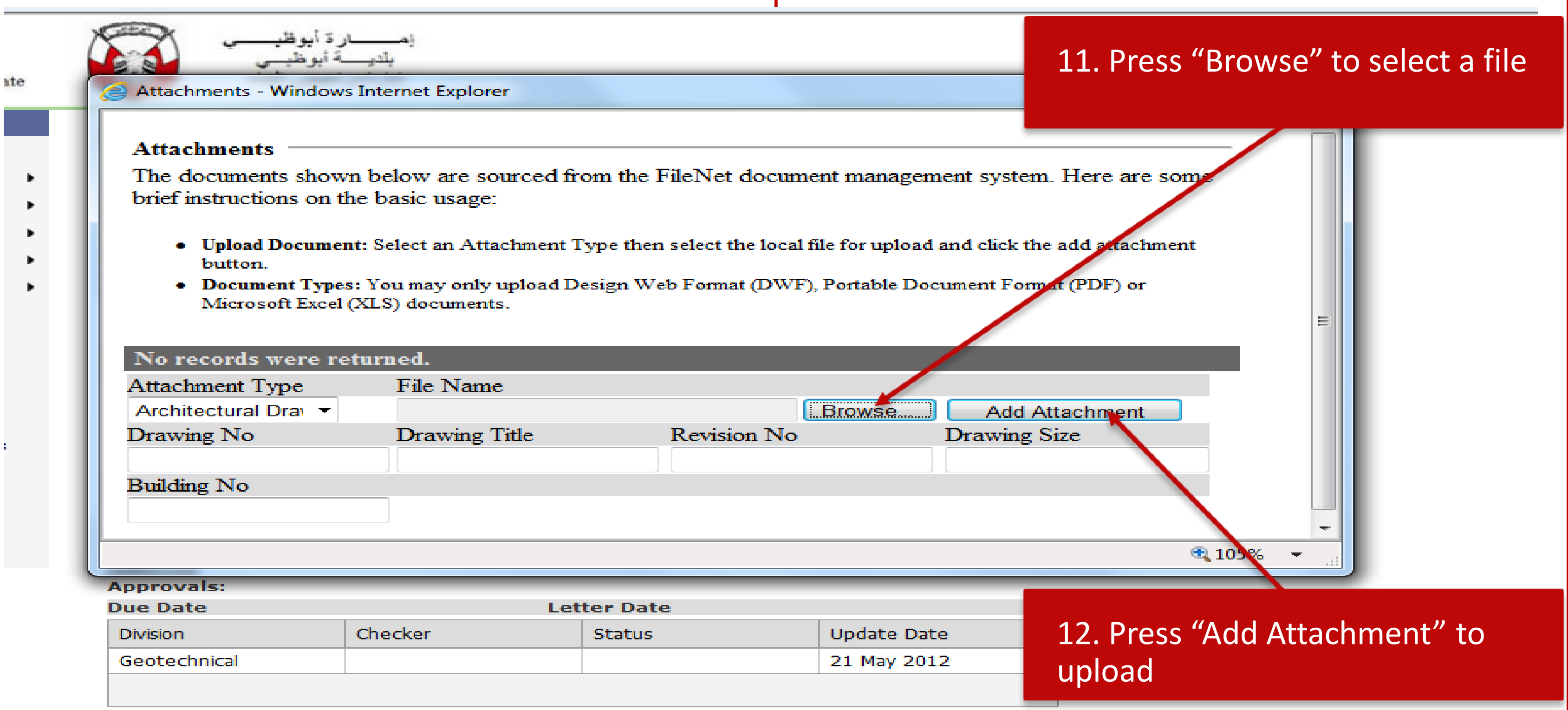


خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية

Application Steps for Infrastructure Projects

- Press :"Browse" to Select file from your PC
- Press "Add Attachment" to upload the file

- اضغط "تصفح" لاختيار ملف
- اضغط "إضافة مرفق" لتحميل الملف



خطوات التقديم على رخصة سحب المياه لمشاريع البنية التحتية
Application Steps for Infrastructure Projects

• Repeat Steps 11 and 12 until all documents are uploaded • Use “Risk Ass. Infrs.” to upload the HSE Risk Assessment Matrix Hints: 1. Limit the size of each file to a maximum of a few MB (less than 5 MB). 2. Break up large files into smaller files, for example, break up the geotechnical report into: text, borehole logs, lab tests, etc. 3. Use file names that are relevant to the content of the attachment, for example, use Geo_Txt.pdf to refer to the attachment that contains the text of the geotechnical report, etc.	• كرر الخطوات 11 و 12 إلى أن يتم تحميل جميع المرفقات • يتم تحميل «جدول تحليل المخاطر في Risk Ass. Infrs. ملاحظات: 1. حدد حجم كل ملف بما لا يزيد عن عدد قليل من الميغابايت (5 أو أقل) 2. تقسيم الملفات الكبيرة على عدة ملفات أصغر، على سبيل المثال، تقسيم تقرير فحص التربة إلى: النص، وسجلات السبر، الفحوصات المخبرية، الخ. 3. استخدم أسماء ملفات التي لها صلة بمضمون المرفق، على سبيل المثال، استخدم Geo_Txt.pdf للإشارة إلى المرفق الذي يحتوي على نص تقرير فحص التربة، الخ.

APPENDIX C

EXAMPLE APPLICATION SUBMITTAL FOR DEWATERING OF SHALLOW EXCAVATION

ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

PROJECT DEWATERING REPORT

SHALLOW DEWATERING PROGRAM FOR CONSTRUCTION OF (*STRUCTURE NAME*)

ADDRESS OF PROJECT
ZONE NO./NAME
SECTOR NO.
PLOT NO.

PROJECT NO. XX
REVISION 1
XX JUNE 2014

CONTRACTOR NAME
CONTRACTOR ADDRESS
CONTRACTOR PHONE CONTACTS
WWW.CONTRACTOR.COM

TABLE OF CONTENTS

PROJECT DEWATERING REPORT

SHALLOW DEWATERING FOR CONSTRUCTION OF (*STRUCTURE NAME*)

1.0 INTRODUCTION

1.1 PROJECT UNDERSTANDING

The Project consists of dewatering of the proposed *(name)* located at *(address of structure)*. The scope involves the dewatering of the proposed site utilizing a single-stage wellpoint dewatering system as per site requirements to enable construction to be carried out in dry, safe conditions.

The dewatering system installation shall be dependent on the site access and working space available. The dewatering system needs to be installed and operational before the excavation begins. After the Method Statement (***Section 4.0***) has been approved by the Abu Dhabi City Municipality (ADM), the Main Contractor should be given approval to begin mobilizing.

1.2 PROJECT ENTITIES: OWNER, CONTRACTOR, AND DEWATERING SUBCONTRACTOR

It is the intention of the Dewatering Subcontractor to coordinate closely with the Main Contractor in terms of mobilization, installation, commissioning, operation, and maintenance of the dewatering system. Relevant project entities are as follows:

Owner:	(Name and Address of Owner)
Main Contractor:	(Main Contractor Name)
Dewatering Subcontractor:	(Dewatering Subcontractor Name)

1.3 HEALTH AND SAFETY PROGRAM

Personnel Health and Safety will be protected under the Main Contractor's and Dewatering Subcontractor's Health and Safety Plans. An outline of hazards in the Risk Analysis for the Project, including Health and Safety, is provided in ***Appendix C3***.

1.4 ENVIRONMENTAL PROTECTION PROGRAM

Owner's environmental guidelines and standards have been considered throughout the design of the proposed system. Environmental protections will be provided in accordance with local regulations and the Dewatering Subcontractor's standard environmental protection plans. Owner shall be responsible for all permits associated with dewatering.

To limit emissions on the site, electric motors shall be used as prime movers where possible allowing for the concentration of diesel prime movers at dedicated control stations.

Diesels and oils shall be restricted within drip trays to prevent contamination of the sand. Bulk storage fuel tanks shall be bundled to contain waste or spillage.

All dewatering system effluent shall pass through a settlement tank before discharge into the approved collection area, or approved stormwater discharge manhole.

An outline of hazards in the Risk Analysis for the Project, including environmental protection, is provided in ***Appendix C3***.

1.5 QUALITY ASSURANCE/QUALITY CONTROL MANAGEMENT PROGRAM

Quality Assurance/Quality Control (QA/QC) Management will be provided under the Dewatering Subcontractor's internal processes and will be the responsibility of the Site Manager.

2.0 SITE SUBSURFACE CONDITIONS

2.1 DESCRIPTION OF SITE GEOLOGIC SETTING AND HYDROGEOLOGY[1]

Generally, Abu Dhabi Emirate is divided into four physiographic regions, each with unique topographic, geologic, geomorphic, and hydrologic characteristics. Namely, these regions include the Al Hajar Mountains, the wadis and alluvial fans along the flanks of the Al Hajar, the vast desert region of sand dunes and inland sabkhas (the largest area within the Abu Dhabi Emirate), and the coastal region that includes sand dunes, coastal sabkhas, lagoons, and tidal zones. Overall, this coastal physiographic region is characterized by the lowest elevations (typically less than 20 meters mean sea level [m msl]) and flattest surfaces in the Emirate, and contains a higher proportion of sabkha (coastal sabkha). The Abu Dhabi City Municipality (ADM) lies primarily within the coastal region but also the desert region in its southern and eastern extents.

Erosion and fluvial transport of sand, gravel, and boulders down the sides of the Al Hajar and in mountain valleys (wadis) have resulted in the formation of extensive outwash sand, gravel deposits, and alluvial fans that empty out onto the desert floor (Kumar et al., 2008). Within the eastern ADM, these alluvial and wadi land surfaces are covered with a vast expanse of desert filled with sand dunes and inland sabkhas (Styles et al., 2006). Along the Arabian Gulf coastline, which includes most of the ADM, the land surface contains sand dunes, beach sand deposits, lagoonal silts and clays, and coastal sabkha deposits (Farrant et al., 2012a).

In the ADM, Precambrian (Proterozoic) basement rocks lay approximately 9 km below ground level (BGL) (Farrant et al., 2012a). Overlying the crystalline basement in the United Arab Emirates (UAE) is a thick sequence of shallow, nearly flat-lying marine sedimentary rocks that include up to 2,500 m of Cambrian to Carboniferous aged primarily clastic rock, over which was deposited approximately 4,300 m of mainly limestone, dolomite, mudstone, and anhydrite of mid-Permian to Cretaceous age (Styles et al., 2006). Following a period of erosion and non-deposition that created a regional unconformity at the end of the Mesozoic Era, significant amounts of shale, mudstone, argillaceous limestone, and anhydrite layers

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[1] This Section should be project specific. As no location data is presented for the example, only a general description of geologic setting is provided here.

were deposited from the Paleogene to mid-Miocene (Farrant et al., 2012a). The Gachsaran Formation is the uppermost of these younger rock strata in the ADM.

The overlying formations represent the shallow groundwater flow system within ADM, and are the most important relative to fresh groundwater supplies in the Emirate and to the geotechnical and engineering problems associated with construction and dewatering in the ADM. Overlying the Gachsaran in the northern and eastern parts of the ADM are the distal ends of wadi and alluvial fan deposits of the Barzaman Formation and the Hili Formation (Farrant et al., 2012b). Barzaman rocks grade laterally to the south and to the west into the Baynunah formation whereas the Hili deposits are younger and cut down through the older Barzaman or Baynunah deposits (Farrant et al., 2012b). These units make up the upper bedrock surface within the ADM.

The late Pleistocene (Quaternary) age, weakly- to well-cemented, aeolian sandstone deposits of the Ghayathi Formation overlies the above-mentioned formations (Farrant et al., 2012b). the Ghayathi is in turn overlain by younger, late Pleistocene to Holocene age unconsolidated dune deposits of the Rub al Khali Formation that includes thick deposits of fine- to medium-grained sands that are carbonate-rich near the coast and quartz-rich inland (Farrant et al., 2012b). Within many inland areas, the younger Rub al Khali sands have been deflated down to the water table, where these flat, low-lying areas frequently becomes cemented with halite and gypsum, forming sabkhas. Along the coast, and extending a few kilometers inland in the ADM, the coastal zone sand dunes, beach sands, sabkha, lagoonal muds, intertidal algal mats, and other near-shore marine deposits of late Pleistocene to Holocene age are collectively referred to as the Abu Dhabi Formation (Farrant et al., 2012b).

It is important to note, however, that much of the ADM includes land areas that have been reclaimed by sabkha infilling, or that have been formed by the artificial expansion of existing islands using fill material. Fill material utilized in the development of this “made ground” is typically dredged sediment from near-shore shipping channels, near-shore sand dunes, or fill material transported from greater distances.

The top of bedrock elevations, within the ADM, range from roughly -20 to 100 m msl. Generally, the lowest top of bedrock surface elevations (-20 to 10 m msl) are located in the western ADM, but rise inland, effectively mimicking surface elevation changes. In the core geotechnical hazard area, bedrock elevations range from approximately -15 to 55 m msl, with top of rock elevations being highest (10 to almost 55 m msl) to the east.

Total thickness of the unconsolidated sediments and fill material (made ground) within the ADM region (i.e., Quaternary aquifer) ranges from 0 to about 24 m, although in most areas, the overburden thickness ranges only from 0 to 10 m. In the core geotechnical hazard zone, the total thickness of unconsolidated sediments is typically 15 m or less.

Interpreted potentiometric surfaces from waterstrike data indicates that groundwater elevations in the western and north-central ADM are relatively flat, ranging from approximately -15 up to about 10 m msl. Groundwater elevations generally increase in an easterly direction, mimicking ground surface elevations with the relatively highest groundwater levels (roughly 70 to 102 m msl) observed in the southeastern ADM. In the westernmost core ADM area, groundwater elevations are relatively flat, ranging from approximately -15 to 5 m msl while in the eastern core area, groundwater elevations appear much higher, between about 5 and 45 m msl.

2.2 DESCRIPTION OF GEOTECHNICAL CONDITIONS

Geotechnical investigations and testing of the subsurface materials and groundwater were performed in accordance to the recommendations given in Field Investigation Scope (Table 11-3) of the ADM Guidelines. The investigation includes three borings to a depth of 25 m, which are much greater than 2x the excavation depth (3 m). This meets the requirements per the ADM guidelines.

FIELD INVESTIGATION SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis

FIELD INVESTIGATION SCOPE (CONTINUED)

KEY		
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspection on boring logs required
		Visual inspection on boring logs not required

EXCERPT FROM TABLE 11-3 OF THE ADM GUIDELINES

HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	BOREHOLE DISTRIBUTION (< 1000 m^2)	VISUAL INSPECTIONS	LAB TESTING[3]	FIELD TESTING	THIRD PARTY REVIEW
C	1,2	**i**	**Shallow**	**3**[1]	**X**	**1**[4]		
		i, ii, iii	Medium	3[1]		1[4]	Slug Test	
			Deep	3[1]		1[4]	Packer Test[5]	X
	3	i	Shallow	3[1]	X	1[4]		
		ii, iii	Medium	3[1]	X	1[4]	Slug Test	
		i, ii, iii	Deep	3[1]		1[4]	Packer Test[5]	X

Notes:

1 Two thirds of the boreholes should be up to 1.5 x depth of excavation and the remaining boreholes up to 2 x depth of excavation.

2 One borehole each at the corners and one at approximate center location or at a spacing not exceeding 50 m c/c. For soil and ground water testing, refer to ***Section 11***.

3 Sieve analysis and Atterberg Limits (Soil Classification, e.g., USCS).

4 One test per geologic layer (based on geologist's description) but no less than one test per 3 m of depth.

5 Packer tests are performed in rock formations only. If not applicable, a slug test is recommended.

The site subsurface is characterized by approximately 7.5 m of medium dense to dense, fine to coarse grained silty sands (SM) underlain by weak, fine grained gypsum, and mudstone. The groundwater table is 0.7 m of the existing ground level.

Details on the Geotechnical Investigation at the site are provided in ***Appendix C4***[2].

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2 For the sake of brevity, only select pages are provided in this example. The full Geotechnical Investigation Report is required in an actual application, to be provided as an Appendix to the application.

3.0 DEWATERING SYSTEM DESIGN

3.1 SUMMARY OF DESIGN CALCULATIONS

The Site is in Hazard Zone C with few structures close enough to be impacted. As only a shallow excavation is necessary (3 m), an analytical solution for pumping capacity and a hand calculation for settlement analysis is required by the Dewatering Design Scope (Table 10-3) in the ADM Guidelines. Design calculations to determine the expected dewatering production flow rate and radius of influence of a single wellpoint system have been performed. These calculations are provided in ***Appendix C2***.

DEWATERING DESIGN SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspection on boring logs required
		Visual inspection on boring logs not required

DEWATERING DESIGN SCOPE (CONTINUED)

EXCERPT FROM TABLE 10-3 OF THE ADM GUIDELINES

HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	PUMPING CAPACITY ANALYSIS[1]	SETTLEMENT ANALYSIS	THIRD PARTY REVIEW
C	1,2	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	c	I	X
	3	i	Shallow	a	I	
		ii, iii	Medium	a	I	
		i, ii, iii	Deep	b	I	X

Note:

[1] Simplified hand calculations are recommended when numerical models are developed. Hand calculations can provide useful checks of more advanced models.

3.2 EVALUATION OF POTENTIAL SITE IMPACTS DUE TO DEWATERING

The Site is expected to have relatively low risk of detrimental effects from a properly designed and operated dewatering system. The following sections discuss several typical site impacts caused by wellpoint dewatering systems.

3.2.1 Settlement and Soil Collapse

Settlement due to soil collapse is possible when dewatering operations cause the migration of fine materials from the surrounding soil through the dewatering system, or where the flow of water through the surrounding soil encounters soluble minerals such as sabkha or gypsum. The primary cause of settlement, however, is the increase in effective stress due to dewatering. Differential settlement is of special concern.

Risk due to migration of fine materials will be mitigated by the presence of the filter zone around the wellpoints and by the presence of the Site Manager, who will inspect flows for signs of turbidity and take appropriate measures if excessive turbidity occurs.

Soluble minerals were not encountered in the geotechnical exploration and are not expected at the site.

Instrumentation for measuring settlement/subsidence, deformation, and slope movement is not required as can be seen from Dewatering Monitoring Scope (Table 12-2) of the ADM Guidelines for this type of project.

DEWATERING MONITORING SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
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	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspection on boring logs required
		Visual inspection on boring logs not required

EXCERPT FROM TABLE 12-2 OF THE ADM GUIDELINES								
HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	WATER LEVEL / TURBIDITY / TDS	SURVEY/ DEFORMATION	SETTLEMENT	SLOPE MOVEMENT	FLOW MEASUREMENT
C	1,2	i	Shallow	-	-	-	-	
		i, ii, iii	Medium	4 [1]	1 [2]	1 [2]	1 [2]	X
			Deep	4 [1]	1 [2]	1 [2]	1 [2]	X
	3	i	Shallow					
		ii, iii	Medium					
		i, ii, iii	Deep	4 [1]	1 [2]	1 [2]	1 [2]	X

Notes:

[1] One piezometer each at the corners or at a spacing not exceeding 50 m c/c.

[2] One instrument on or near to each sensitive structures/steep section of the slope.

The settlement calculation is provided in ***Appendix C2***.

3.2.2 Excavation Slope Stability

Dewatering to levels below the planned excavation elevation is required to maintain slope stability and reduce the chance of boils in the sandy materials at the site. Groundwater levels in the excavation area will be observed and monitored throughout the dewatering process. Excavation will not be performed to a level closer than 0.5 m from the groundwater table. Slope geometry will be 2H: 1V for the entire excavation. Open excavation may be allowed if it remains within the property limits, i.e., the excavation shall be contained within the plot limits and not encroach on neighboring property.

Instrumentation for measuring slope movements is not required by the ADM Guidelines for this type of project.

3.2.3 Migration of Fine Materials

As discussed in ***Section 3.2.1***, migration of fine materials is a primary cause for excessive settlement and soil collapse. In addition, discharge of highly turbid water into the adjacent storm sewer system is prohibited by the site permit. Discharge of highly turbid water will be prevented by the filter zone around the wellpoints as described in ***Section 4.0***, and by daily observation of dewatering discharges. Settlement tanks will be used in conjunction with dewatering pumps.

The filter zone will be designed according to ***Section 4.3*** of the Dewatering Guidelines and properties of the silty sand material surrounding the filter zone. Uniformity Coefficient (C_u) and D_{50} are determined from Sieve Analysis results, such as those presented in ***Appendix E4***. In this case, D_{50} of the silty sand is around 0.11 millimeter (mm) and C_u is approximately 2.3. Since C_u is generally less than 3, the D_{50} of the filter pack should be 4 to 5 times D_{50} of the silty sand, and C_u of the filter pack material must be smaller (more uniform) than C_u of the silty sand.

4.0 CONSTRUCTION METHOD

4.1 PURPOSE, SCOPE, AND DESIGN BASIS

This Method Statement describes the activities and procedures for the dewatering of the proposed structure at (location), Abu Dhabi. The scope involves the dewatering of the proposed site to enable construction to be carried out in dry, safe conditions, utilizing a single-stage wellpoint dewatering system.

The works for the proposed development involve the excavation as per the following information:

- Area for Dewatering: 21 m x 26 m approximately
- Ground Level: +0.00m
- Existing Water Table level: 0.7 m Below Ground Level (BGL)
- Max Excavation level: approximately 3.00 m BGL
- Shoring: Open-cut excavation

4.2 SUMMARY OF PROPOSED DEWATERING SYSTEM

It is proposed to install a single-stage wellpoint system to control the groundwater within the excavation area. Wellpoint dewatering involves the installation of riser pipes with a filter section on the lower portion, connected into a common header pipe from which the water can be pumped by a vacuum assisted pump to a convenient discharge point. Polyvinyl chloride (PVC) riser pipes shall be inserted in predrilled holes, which are widened and cleaned through the use of high pressure water. The jetting pump is connected to a lance through which the high pressure water is injected into the ground, forming a borehole into which the wellpoint can be installed. The lance is rotated to create a gap around the wellpoint, which in turn is filled with suitable filter material.

4.2.1 Physical Arrangement of Dewatering Equipment

Wellpoints shall be installed at 1.00 m intervals around the perimeter of the excavation. The dewatering equipment will be arranged as shown in ***Appendix C1***, ***Figures 1-1 and 1-2***.

4.2.2 Installation Method and Required Equipment

Initial excavation for all structures may be required as per site conditions or the dewatering system will be installed a minimum 0.5 m above existing groundwater level.

As a means of ensuring the integrity of the wellpoint installation, boreholes shall be predrilled to the required depth at1.00 m intervals. Predrilling shall be executed by the use of a self-propelled drilling rig and continuous flight augers. After completion of predrilling, wellpoints, a jetting lance will be inserted and high pressure water shall be injected into the ground through the jetting lance, widening and cleaning the borehole. This development of wells is important to maximize the effectiveness of the system and to produce clean discharge.

Once the wellpoint is installed, the lance is held in this position until the water being ejected out through the top of the borehole runs clean. The borehole and wellpoint surroundings are backfilled with aggregate, 10 centimeters (cm) to 15 cm above the working platform. As the jetting lance is withdrawn from within the borehole, the aggregate level shall reduce by the displacement created by the removal of the lance. This procedure is repeated upon installation of each wellpoint.

Upon completion of the installation of the predetermined number of wellpoints, the wellpoints shall be connected to the 6-inch header pipe network by means of swing pipes, which in turn is connected to the dewatering pump.

4.2.3 Operations, Monitoring, and Maintenance during Construction

Instrumentation for measuring the groundwater level, settlement/subsidence, and slope movements are not required by the ADM Guidelines.

Visual inspection will be performed by the Site Manager during the operations and maintenance of the wellpoint dewatering system and will consist of the following daily checks:

- Dewatering pump operation and condition, including performance of any required maintenance or repairs.
- Daily monitoring of dewatering system effluent to identify excessive turbidity in flows. If excessive turbidity is noted, the Site Manager will adjust the dewatering system to limit the migration of fine materials by throttling or shutting off specific wellpoints.
- Visual inspection of the surrounding area and excavation slopes for signs of excessive settlement, slope instability, or excessive seepage into the excavation.

No overhead or underground obstructions are expected.

REFERENCES

Farrant, A.R., R.A. Ellison, J.W. Merritt, J.E. Merritt, A.J. Newell, J.R. Lee, S.J. Price, R.J. Thomas, and A. Leslie, 2012a, “Geology of the Abu Dhabi 1:100,000 Map Sheet, 100-16, United Arab Emirates,” Ministry of Energy, United Arab Emirates, 69 p.

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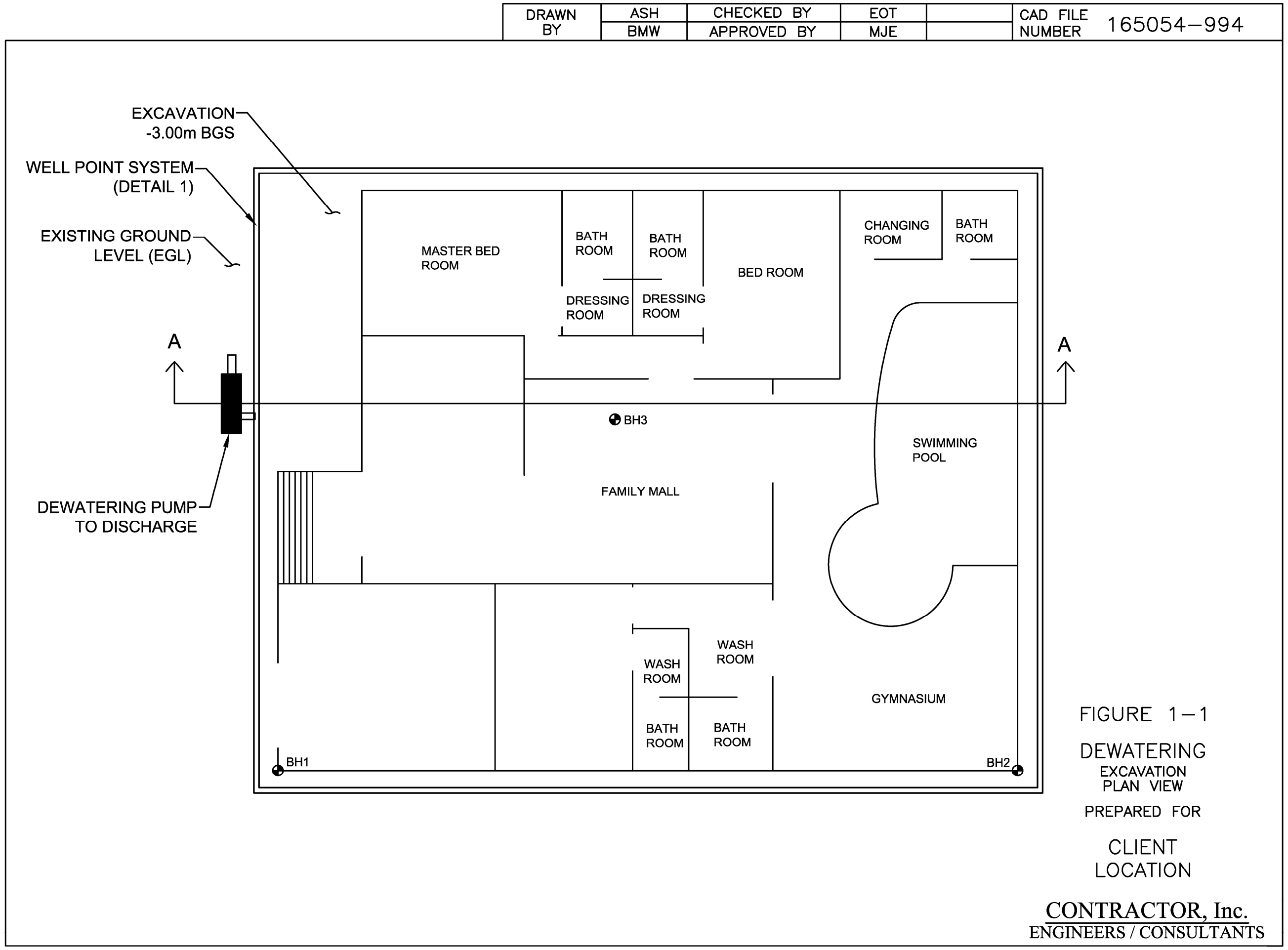
APPENDICES

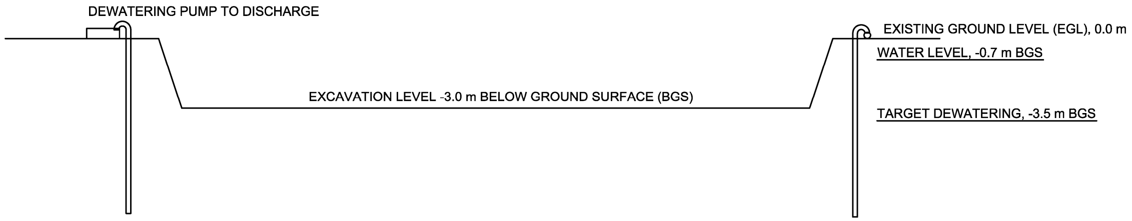
APPENDIX C1

DRAWINGS

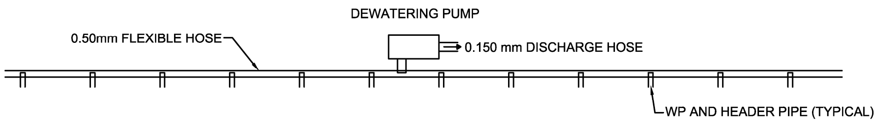

DRAWN BY
ASH
BMW
CHECKED BY
APPROVED BY
EOT
MJE
CAD FILE NUMBER
165054-994
EXCAVATION
-3.00m BGS
WELL POINT SYSTEM
(DETAIL 1)
EXISTING GROUND
LEVEL (EGL)
A
A
DEWATERING PUMP
TO DISCHARGE
MASTER BED ROOM
BATH ROOM
BATH ROOM
BED ROOM
CHANGING ROOM
BATH ROOM
DRESSING ROOM
DRESSING ROOM
BH3
SWIMMING POOL
FAMILY MALL
WASH ROOM
WASH ROOM
GYMNASIUM
BATH ROOM
BATH ROOM
BH1
BH2
FIGURE 1-1
DEWATERING
EXCAVATION
PLAN VIEW
PREPARED FOR
CLIENT
LOCATION
CONTRACTOR, Inc.
ENGINEERS / CONSULTANTS

ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

DRAWN BY	ASH	CHECKED BY	EOT		CAD FILE NUMBER	165054–995
	BMW	APPROVED BY	MJE			



A SECTION A
SECTION THROUGH EXCAVATION



1 DETAIL 1
WELL POINT (WP) SYSTEM, TYPICAL

FIGURE 1–2

DEWATERING
EXCAVATION AND
WELL POINT DETAILS
PREPARED FOR

CLIENT
LOCATION

CONTRACTOR, Inc.
ENGINEERS / CONSULTANTS

APPENDIX C2

CALCULATIONS

APPENDIX C2

CALCULATIONS

According to the ADM Guidelines for a shallow excavation in Hazard Zone C that is away from sensitive or large structures, only an analytical solution for dewatering design is necessary. Assumptions for using this solution are that the aquifer is homogeneous and isotropic and that the excavation is nearly square.

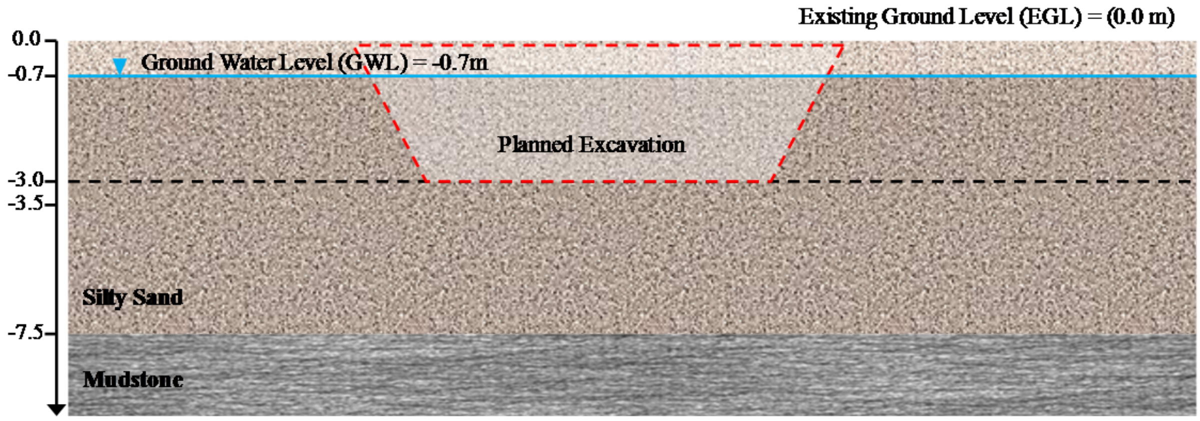
CALCULATION INPUT

Refer to ***Appendix C1*** for site geometry. Refer to ***Section 2.2 and Appendix C4*** for subsurface conditions. ***Table 1*** summarizes parameters used in this calculation, and ***Figure 1*** depicts a sketch of the initial condition of the project site.

TABLE 1
PARAMETERS USED IN PUMP CAPACITY AND SETTLEMENT HAND CALCUATIONS

PARAMETER	VALUE	JUSTIFICATION
Width of Excavation	21 m	From project drawings
Length of Excavation	26 m	Distance between sumps
Water Depth (BGL)	0.7m	From Geotechnical Investigation Report
Excavation Depth (BGL)	3 m	Max, from project drawings
Drawdown Depth (BGL)	3.5 m	Required 0.5 m below excavation by ADM Guidelines
Required drawdown	2.8 m	Required 0.5 m below excavation by ADM Guidelines
Permeability	5.00E-4 m/s	Mean from Table 10-1 of the ADM Guidelines for Fine Sand
Saturated Unit Weight, Sand	21.0 kN/m^3	From consolidation test ***Appendix C4***
Dry Unit Weight, Sand	16.8 kN/m^3	From consolidation test ***Appendix C4***
Initial Void Ratio	0.42	From consolidation test ***Appendix C4***
Compression Index	0.069	From consolidation test ***Appendix C4***

FIGURE 1
INITIAL CONDITIONS OF THE PROJECT SITE
(PUMP CAPACITY POINT OF VIEW)



ANALYTICAL SOLUTION

Pump Capacity

This calculation is based on the ADM Guidelines and the dewatering site is simulated as a single equivalent well. The flow rate required for the dewatering pump is calculated using:

1. The radius of a circle equivalent to the area of excavation.
2. Radius of influence considering depth of dewatering and subsurface permeability.

The equivalent radius, based on perimeter, r_w, is found using ***Equation 1***.

$$r_w = (a + b)/\pi \qquad \text{(Equation 1)}$$

where,
a = width of excavation = 21 m
b = length of excavation = 26 m

$$r_w = (21m + 26m)/\pi$$
$$r_w = 14.96m$$

r_w = equivalent radius in meters (m) = 15 m

The radius of influence from drawdown is found using ***Equation 2***.

$$R_0 = C * (H - h_w) * \sqrt{k} \qquad \text{(Equation 2)}$$

where,

C = empirical calibration factor = 3,000 for radial flow to pumped wells

$H - h_w$ = target drawdown of the excavation = 2.80 m

k = Mean permeability from the Typical Values of Permeability of Saturated Soils (Table 10-1) of the ADM Guidelines for Fine Sands = 5.00E-05 m/s

$$R_0 = 3{,}000 s^{0.5}/m^{0.5} * (2.8m) * \sqrt{5x10^{-5} m/s}$$
$$R_0 = 59.4m$$

R_0 = radius of influence = 60 m

Using the equation for well or point source dewatering assuming radial flow in an unconfined aquifer, the flow rate is calculated as in ***Equation 3***.

$$Q = \pi\, k\, (H^2 - h_w^2)/\ln(R_0/r_w) \qquad \text{(Equation 3)}$$

where,

H = hydraulic head of the original water table = 6.8 m

h_w = hydraulic head at bottom of wellpoint = 4 m

R_o = radius of influence as calculated in ***Equation 2*** = 60 m

r_w = equivalent radius of the well as calculated in ***Equation 1*** = 15 m

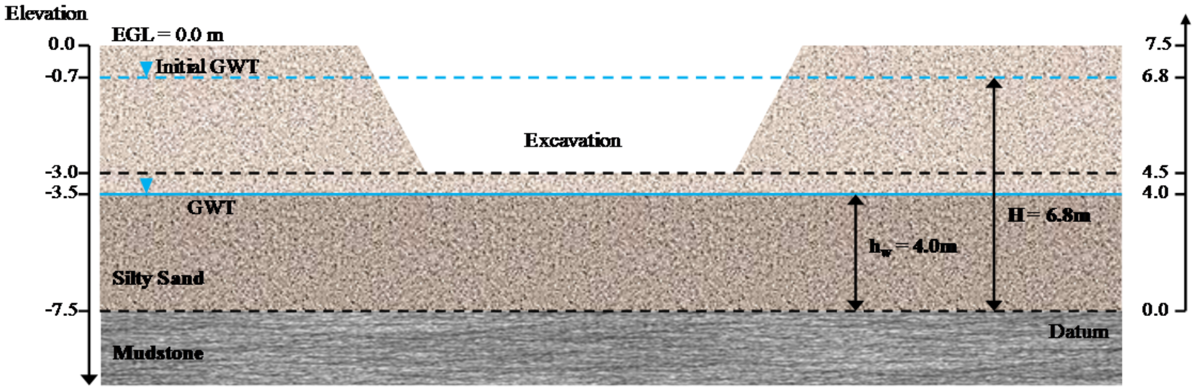
k = Mean permeability from the Typical Values of Permeability of Saturated Soils (Table 10-1) from the ADM Guidelines for Fine Sands = 5.00E-05 m/s

$$Q = \frac{\pi\ 5x10^{-5} m/s\ ((6.8m)^2 - (4m)^2)}{\ln\left(\frac{60m}{15m}\right)}$$
$$Q = 3.43x10^{-3} m^3/s$$

Q = flow rate of pump = 0.0034 m^3/s

For this example, we assumed that the datum of the aquifer is in the top of the mudstone.

FIGURE 2
CONDITIONS AFTER EXCAVATION AND DEWATERING
(PUMP CAPACITY POINT OF VIEW)



Settlement Calculation

Settlement from dewatering occurs from the consolidation of compressive sands due to an increase in effective stress, even if dewatering is carried out properly. Dewatering removes buoyancy from the soil, creating a differential of effective stresses during the process.

Estimated settlement is calculated using:

1. Saturated unit weight
2. Dry unit weight
3. Stresses from initial and targeted water level.

Saturated unit weight and dry unit weight are provided in ***Table 1***. These parameters are usually provided when consolidation tests have been performed.

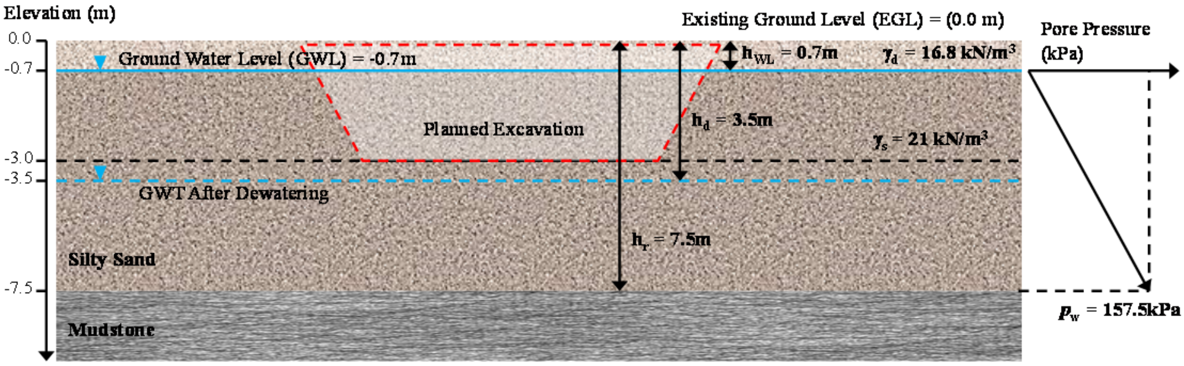
γ_s = saturated unit weight = 21 kN/m^3
γ_d = dry unit weight = 16.8 kN/m^3

Total and effective stresses at the middle of both the dewatered and saturated part of the silty sand layer are defined under initial conditions (***Figure 3***), i.e., before dewatering, using ***Equations 4 through 7***. Final effective stresses at the same locations after dewatering are calculated in ***Equation 8 and 11*** (***Figure 4***). Due to the increment of effective stresses, the soil dewatered and under the drawdown area is susceptible to settlement. This settlement is analyzed taking into account that consolidation represents the best behavior of this phenomenon.

Initial Stresses at Mid layer of the Dewatered Section

The mid layer of the dewatered section is located at a depth of 2.1 m. The total and effective stresses in this location under conditions prior to dewatering are calculated below. By calculating the stresses in the middle of the layer, it is considered that the mean values of these stresses would yield a more representative result for the settlement analysis in this section of the soil.

FIGURE 3
INITIAL SITE CONDITIONS
(SETTLEMENT CALCULATION POINT OF VIEW)



$$\sigma_{0\,@\,2.1m} = h_{WL} \times \gamma_d + (h_d - h_{WL})/2 \times \gamma_s \qquad \text{(Equation 4)}$$

$$\sigma'_{0\,@\,2.1m} = \sigma_{o\,@\,2.1m} - (h_d - h_{WL})/2 \times \gamma_w \qquad \text{(Equation 5)}$$

where,

γ_s = saturated unit weight = 21 kN/m^3

γ_d = dry unit weight = 16.8 kN/m^3

γ_w = unit weight of water = 9.8 kN/m^3

h_{WL} = depth of water level below ground surface = 0.7 m
h_d = depth of water level after dewatering = 3.5 (set equal to h_r if greater than h_r)
h_r = thickness of soil to calculate settlement = 7.5 m (thickness of surficial sand layer)

$$\sigma_{0\ @\ 2.1m} = 0.7m \times 16.8kN/m^3 + (3.5m - 0.7m)/2 \times 21kN/m^3$$
$$\sigma_{0\ @\ 2.1m} = 41.2kPa$$

$\sigma_{0\ @\ 2.1m}$ = total stress at 2.1 m depth under initial conditions = 41.2 kPa

$$\sigma'_{0\ @\ 2.1m} = 41.2kPa - (3.5m - 0.7m)/2 \times 9.8kN/m^3$$
$$\sigma'_0 = 27.4kPa$$

$\sigma'_{0\ @\ 2.1m}$ = effective stress at 2.1 m depth under initial conditions = 27.4 kPa

Initial Stresses at Mid-layer of the Saturated Section

The mid layer of the section below the drawdown considered in the settlement analysis is located at a depth of 5.5 m. The total and effective stresses in this location under conditions prior to dewatering are calculated below. By calculating the stresses in the middle of the layer, it is considered that the mean values of these stresses would yield a more representative result for the settlement analysis in this section of the soil.

$$\sigma_{0\ @\ 5.5m} = h_{WL} \times \gamma_d + [(h_d - h_{WL}) + (h_r - h_d)/2] \times \gamma_s \quad \text{(Equation 6)}$$

$$\sigma_{0\ @\ 5.5m} = 0.7 \times 16.8kN/m^3 + \left[3.5m - 0.7m + \frac{7.5m - 3.5m}{2}\right] \times \frac{21kN}{m^3}$$
$$\sigma_{0\ @\ 5.5m} = 112.6kPa$$

$\sigma_{0\ @\ 5.5m}$ = total stress at 5.5m depth under initial conditions = 112.6 kPa

$$\sigma'_{0\ @\ 5.5m} = \sigma_{o\ @\ 5.5m} - [(h_d - h_{WL}) + (h_r - h_d)/2] \times \gamma_w \quad \text{(Equation 7)}$$

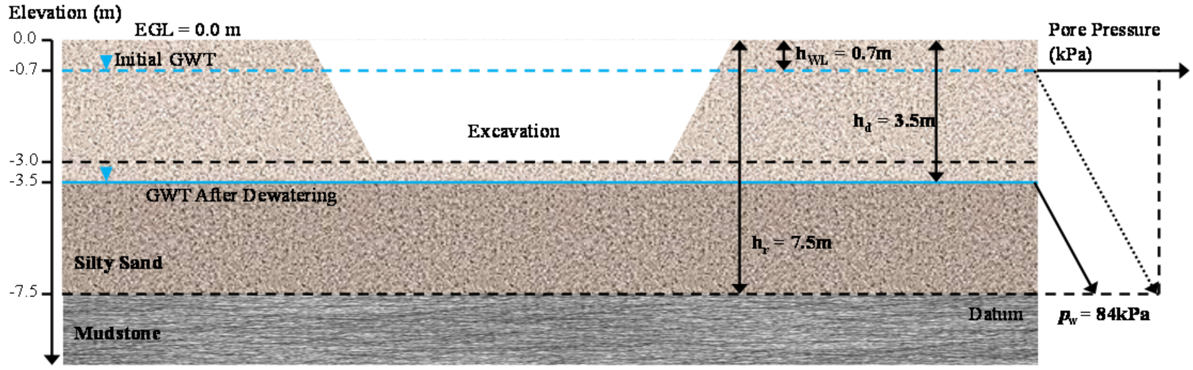
$$\sigma'_{0\ @\ 5.5m} = 83.2kPa - [(3.5m - 0.7m) + (7.5m - 3.5m)/2] \times 9.8kN/m^3$$
$$\sigma'_{0\ @\ 5.5m} = 65.5kPa$$

$\sigma'_{0\,@\,5.5m}$ = effective stress at 5.5m depth under initial conditions = 65.5 kPa

Final Stresses at Mid layer of the Dewatered Section

The mid layer of the dewatered section is located at a depth of 2.1 m. The total and effective stresses in this location after dewatering are calculated below. By calculating the stresses in the middle of the layer, it is considered that the mean values of these stresses would yield a more representative result for the settlement analysis in this section of the soil.

FIGURE 4
CONDITIONS AFTER EXCAVATION AND DEWATERING
(SETTLEMENT CALCULATION POINT OF VIEW)



$$\sigma_{f\,@\,2.1m} = [h_{WL} + (h_d - h_{WL})/2] \times \gamma_d \qquad \text{(Equation 8)}$$

$$\sigma'_{f\,@\,2.1m} = \sigma_{f\,@\,2.1m} \qquad \text{(Equation 9)}$$

$$\sigma'_{f\,@\,2.1m} = [0.7m + (3.5m - 0.7m)/2] \times 16.8kN/m^3$$

$$\sigma'_f = 35.3\ kPa$$

$\sigma'_{f\,@\,2.1m}$ = final effective stress at 2.1m under final conditions = 35.3 kPa

Final Stresses at Mid layer of the Saturated Section

The mid layer of the section below the drawdown considered in the settlement analysis is located at a depth of 5.5 m. The total and effective stresses in this location after dewatering are calculated below. By calculating the stresses in the middle of the layer, it is considered that the mean values of these stresses would yield a more representative result for the settlement analysis in this section of the soil.

$$\sigma_{f\ @\ 5.5m} = h_d \times \gamma_d + (h_r - h_d)/2\ \times \gamma_s \qquad \text{(Equation 10)}$$

$$\sigma_{f\ @\ 5.5m} = 3.5 \times 16.8kN/m^3 + (7.5m - 3.5m)/2 \times 21kN/m^3$$
$$\sigma_{f\ @\ 5.5m} = 100.8\ kPa$$

$\sigma_{f\ @\ 5.5m}$ = total stress at 5.5 m depth after dewatering= 100.8 kPa

$$\sigma'_{f\ @\ 5.5m} = \sigma_{f\ @\ 5.5m} - (h_r - h_d)/2\ \times \gamma_w \qquad \text{(Equation 11)}$$

$$\sigma'_{f\ @\ 5.5m} = 100.8kPa - (7.5m - 3.5m)/2 \times 9.8kN/m^3$$
$$\sigma'_{f\ @\ 5.5m} = 81.2\ kPa$$

$\sigma'_{f\ @\ 5.5m}$ = effective stress at 5.5 m depth after dewatering = 81.2 kPa

Settlement is calculated separately for the section above and below the drawdown depth. As stated previously, the settlement is analyzed taking into account that consolidation represents the best behavior of this phenomenon.

Settlement Calculation: Area above drawdown depth

Estimated settlement for the area above the drawdown depth is calculated using ***Equation 12*** from the compression index, void ratio, and the log difference of final and initial effective stresses.

$$\Delta S_1 = Cc \times (h_d - h_{WL}) \times \log(\sigma'_{f\ @\ 2.1m}/\sigma'_{0\ @\ 2.1m})/(1+e) \qquad \text{(Equation 12)}$$

where,

Cc = compression index = 0.069

e = void ratio = 0.42

$$\Delta S_1 = \frac{0.069 \times 2.8m \times \log(35.3KPa/27.4kPa)}{1 + 0.42}$$

$$\Delta S = 0.015m$$

ΔS_1 = estimated settlement of soils in the area above the drawdown depth= 15 mm

Settlement Calculation: Area below drawdown depth

Estimated settlement for the area below the drawdown depth is calculated using ***Equation 13*** from the compression index, void ratio, and the log difference of final and initial effective stresses.

$$\Delta S_2 = Cc \times (h_r - h_d) \times \log(\sigma'_{f\ @\ 5.5m} / \sigma'_{0\ @\ 5.5m})/(1 + e) \qquad \text{(Equation 12)}$$

where,
Cc = compression index = 0.069
e = void ratio = 0.42

$$\Delta S_2 = \frac{0.069 \times 4m \times \log(81.2KPa/65.5kPa)}{1 + 0.42}$$

$$\Delta S = 0.018m$$

ΔS_2 = estimated settlement of soils in the area above the drawdown depth= 18 mm

The final settlement would be the summation of the settlements above, as shown in ***Equation 13***.

$$\Delta S = \Delta S_1 + \Delta S_2 \qquad \text{(Equation 12)}$$

$$\Delta S = 15mm + 18mm$$

$$\Delta S = 33mm$$

The final settlement is of 33 mm.

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DEPARTMENT OF MUNICIPAL AFFAIRS

APPENDIX C3

RISK ANALYSIS

APPENDIX C3

RISK ANALYSIS

Risk analysis involves considering and rating potential hazards for the duration of the dewatering activity. Hazards are assigned a consequence and likelihood rating according to the following guidelines:

RISK ANALYSIS RATING

CONSEQUENCE RATING	SAFETY	HEALTH	ENVIRONMENT
1	First Aid	Immediate	Minor
2	Medical	Temporary	Short-term
3	Lost-time	Short-term	Long-term
4	Disability	Long-term	Serious
5	Fatality	Fatal	Catastrophic

LIKELIHOOD RATING	FREQUENCY
1	Highly Unlikely
2	Remote
3	Possible
4	Probable
5	Certain

The risk factor is the product of the consequence and likelihood rating. A risk factor 1 to 4 is low risk, 5 to 12 is medium risk, and 13 to 25 is high risk.

[*Address the consequence and likelihood of the following events and any other relevant situation with potential to be hazardous. Suggest control measure and address the consequence and likelihood with the proposed control measures in place.]*

The following hazards are considered for this Project:

1. Mobilization and Demobilization

   a. Loading/Unloading of Equipment on Site

      i. Objects falling from lifting equipment

ii. Sudden failure of wire rope/chain or hydraulic/mechanical system

iii. Materials falling from vehicle

iv. Injuries by vehicle door opening/closing

v. Injury to personnel and property damage by manual handling

vi. Run over/stacked/crushed by a vehicle

vii. Poor lighting

viii. Poor ground/road condition

2. Wellpoint Drilling

a. Movement of Drilling Rig from one location another location

i. Toppling over when travelling on sloped ground

ii. Risk of collision

b. Drilling Operations

i. Injury to the operator and operator assistant due to drill auger replacement

ii. Injury due to equipment rotating parts

3. Wellpoint Installation

a. Wetter Jetting

i. Personnel getting wet

ii. Eye injuries due to jetting water wash in to face

iii. Slip, trip, and fall due to wet surfaces

b. Wellpoint Installation

i. Dust pollution from aggregates

c. Header Pipe Installation

i. Injury to the personnel and property damage by manual handling

d. Dewatering Pump Installation

i. Personnel injury by lifting operation

e. Electrical equipment installation

i. Serious injury to personnel

ii. Fire hazard

4. Discharge Installation, Operation, and Maintenance

a. Discharge Tank Installation

i. Objects falling from lifting equipment while placing the discharge tank

ii. Seepage of water on site

iii. Overflow of water from tank

b. Discharge Pipe Installation

i. Risk of back injury while manual handling

ii. Temporary and permanent back injury by lifting heavy objects

c. Personnel Walking over Loose Sand

i. Slip, trip, and fall while walking over loose sand

5. Wellpoint Operation and Maintenance

a. Equipment Operation

i. Pump Failure

b. Equipment Maintenance

i. Injury from moving parts

c. Hand Tools

i. Personnel injury and property damage

ii. Cuts, burns, heat disorder and eye injury

6. Material storage

a. Injury to personnel

b. Manual handling injuries

c. Materials damage

d. Object falling from height

e. Slip, trip, and falls

f. Chance of fire and explosion hazard

7. Usage of natural resources (diesel/oil/water/gas)

a. Soil contamination on sand

b. Environmental pollution

c. Water pollution

d. Land pollution

e. Damage to property

f. Harm to the health of the environment, including the air, water, or land

8. Fuel Storage

a. Personnel injury or property damage by fire/explosion hazard

b. Spillage and environmental impact

c. Partial/total power loss

9. Working in Hot Weather

a. Potential for heat stress, dehydration, sunburn, and heat stroke, etc.

10. Out-of-hours Maintenance

a. Lone watching, poor lighting, difficult access

11. Neighboring Activities

a. Inadequate lighting

b. Injury to the personnel or property damage while working

c. Fire hazard

d. Falling/flying objects

e. Electric hazards

12. Watch man working at night

   a. Inadequate lighting

   b. Injury to the personnel or property damage while working at night

13. Unauthorized entry to dewatering work activity area

   a. Injuries to personnel

Using Activity 1a as an example, two potential hazards (i and ii) are objects falling from lifting equipment or sudden failure of wire rope/chain or hydraulic/mechanical system. The consequences of these hazards are potentially fatal (Consequence Rating = 5) and the likelihood is possible (Likelihood Rating = 3). The risk factor before control measures of this hazard is 15, which falls in the high risk category.

The following control measures for hazards in 1a (i and ii) are suggested:

1. Ensure that lifting equipment and gears are in good working condition and lifting operation is managed safely (3rd party test certification).

2. Personal protective equipment (PPE) rule must be enforced (high visibility clothing, hard hat, gloves, goggles, and safety footwear).

3. Provide suitable outrigger pads (1 m x 1 m).

4. Operator has valid license. Operator, slinger, and signaler are properly trained.

5. Valid 3rd party certificate for operator, riggers, and banks man.

6. Visible inspection required for overhead electric power lines and underground utilities.

7. Work area is barricaded by Main Contractor.

8. Warning signboards placed by Main Contractor.

9. Materials placed in clear area with proper access and egress.

10. Full-time supervision is necessary by competent person.

If these control measures are implemented the possible consequences are reduced to only first aid (Consequence Rating = 1) and the likelihood of the event occurring is remote (Likelihood Rating = 2). The risk factor of this hazard, after implementing control measures, is now 3, which falls in the low risk category.

APPENDIX C4

GEOTECHNICAL INVESTIGATION REPORT

ABU DHABI CITY MUNICIPALITY

APPENDIX C4

GEOTECHNICAL INVESTIGATION REPORT

Project Name :		**Project Ref No** :	**Borehole No.**
Location :		**Client** :	
Drilling Method :			
Equipment :	**Total Depth (m)** : 58.0		Coordinate X :
Flushing Medium :	**Ground Elevation** : 5.998		(Nahrwan 1967) Y :
Borehole Dia. (mm) : 96	**Core Dia. (mm)**: 60	**Drilling Date Start** :	**End** :
Groundwater Depth (m) : 0.70		**Backfill Date** :	
Casing Dia. (mm) : 114			

Sample/Core Run	Elevation, m (NADD)	Depth (m)	Description	S.P.T. N Value (0–50)	T.C.R. (%) (0–100)	S.C.R. (%) (0–100)	R.Q.D. (%) (0–100)	F.I.
	6	0	(FLL) Medium dense, cream/light greenish grey, gravelly SILT with numerous cemented silt pieces.(Very Recent Fill)	19				
	5	1	(FLL) Medium dense, brown, carbonate, very silty fine to medium SAND with occasional gravels.(Recent Fill)	21				
	4	2	(FLL) Loose, light grey/brown, carbonate, silty fine to medium SAND. (Recent Fill)	9				
	3	3		7				
	2	4	(FLL) Loose, light grey/brown, carbonate, very silty fine to medium SAND with numerous cemented sand pieces.(Recent Fill)	9				
	1	5	(SAS) Dense, light grey/brownish grey, carbonate, calyey SAND with numerous crystalline gypsum pieces.(Sabkha Sand)	39				
	0	6	(SAG) Very dense, cream/light greenish grey, clayey GRAVEL with gravel size gypsum pieces.(Sabkha Gravel)	50				
	-1	7		50				
0.30			(GYS) Weak to moderately strong, cream/light grey, thinly to medium bedded crystalline GYPSUM. Slightly and moderately weathered, very closely and closely, occasionally medium spaced sub-horizontal, rough fractures. (Sabkha Gypsum)		100	60	0	10
	-2	8						5
1.20			8.05 m - 8.35 m Very weak Mudstone.		100	88	69	10
	-3	9	(MDS) Very weak to weak, light grey/light greenish grey, very thinly bedded calcareous MUDSTONE with inclusions of crystalline gypsum					
1.90	-4	10			95	83	78	3

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GEOTECHNICAL INVESTIGATION REPORT (CONTINUED)

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Samp	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND

				Initial	Final
Diameter-D (mm)	71.30		Wet Wt.+Ring (g)	287.69	279.62
Area-A (mm²)	3992.72		Height-H (mm)	22.00	20.59
Height-Hs (mm)	15.50		Volume-V (cm^3)	87.84	82.21
Dry Wt.+Ring (g)	249.64		Wet Density (g/cm^3)	2.14	2.19
Ring Wt. (g)	99.40		Moisture Content (%)	25.33	19.95
Particle Density (Gs)	2.43		Void ratio (e)	0.42	0.33
Degree of Saturation S_o (%)	146.55		Dry Density (g/cm^3)	1.71	1.83

Increment From/To (kPa)	0-50	50-100	100-200	200-400	400-800	800-1400	1400-700	700-0
Pressure (kPa)	50	100	200	400	800	1400	700	1
Deformation of apparatus (mm)	0	0	0	0	0	0	0	0
Consolidated height, Ĥ (mm)	21.77	21.59	21.40	21.16	20.89	20.60	20.686	20.876
Voids ratio (e)	0.41	0.39	0.38	0.37	0.35	0.33	0.34	0.35
Height change-H (mm)	0.23	0.18	0.19	0.24	0.27	0.29		
Pressure change-p (kPa)	50	50	100	200	400	600		
Volume compress-m_v (m^2/MN)	0.21	0.17	0.09	0.06	0.03	0.02		
t_{100} (min)	4.00	7.84	8.41	15.21	16.00	30.25		
Average specimen height (mm)	21.89	21.68	21.49	21.28	21.02	20.74		
Coeff. of consol-C_v (m^2/year)	1.48	0.74	0.68	0.37	0.34	0.18		

Increment	Pressure (kPa)	Test Date
0-50	50	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.56	0.0	0.00
0.1	8.56	0.3	0.00
0.2	8.55	0.4	-0.01
0.5	8.48	0.7	-0.08
1	8.43	1.0	-0.13
2	8.40	1.4	-0.16
4	8.36	2.0	-0.20
8	8.34	2.8	-0.22
15	8.33	3.9	-0.23

Height before loading (mm)	22
Height after loading (mm)	21.77

Increment	Pressure (kPa)	Test Date
50-100	100	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.33	0.0	0.00
0.1	8.31	0.3	-0.02
0.2	8.28	0.4	-0.05
0.5	8.24	0.7	-0.09
1	8.21	1.0	-0.12
2	8.19	1.4	-0.14
4	8.18	2.0	-0.15
8	8.17	2.8	-0.16
15	8.16	3.9	-0.17
30	8.15	5.5	-0.18

Height before loading (mm)	21.77
Height after loading (mm)	21.59

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DEPARTMENT OF MUNICIPAL AFFAIRS

GEOTECHNICAL INVESTIGATION REPORT (CONTINUED)

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		
Borehole No.:		Sample No.:	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND

Increment	Pressure (kPa)	Test Date
100-200	200	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.15	0.0	0.00
0.1	8.13	0.3	-0.02
0.2	8.10	0.4	-0.05
0.5	8.04	0.7	-0.11
1	8.01	1.0	-0.14
2	8.00	1.4	-0.15
4	7.99	2.0	-0.16
9	7.98	3.0	-0.17
15	7.97	3.9	-0.18
30	7.96	5.5	-0.19

Height before loading (mm)	21.59
Height after loading (mm)	21.40

Increment	Pressure (kPa)	Test Date
200-400	400	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.96	0.0	0.00
0.1	7.92	0.3	-0.04
0.2	7.88	0.4	-0.08
0.5	7.82	0.7	-0.14
1	7.79	1.0	-0.17
2	7.78	1.4	-0.18
4	7.76	2.0	-0.20
8	7.75	2.8	-0.21
15	7.74	3.9	-0.22
30	7.73	5.5	-0.23
60	7.72	7.7	-0.24

Height before loading (mm)	21.40
Height after loading (mm)	21.16

Increment	Pressure (kPa)	Test Date
400-800	800	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.72	0.0	0.00
0.1	7.69	0.3	-0.03
0.2	7.65	0.4	-0.07
0.5	7.56	0.7	-0.16
1	7.53	1.0	-0.19
2	7.51	1.4	-0.21
4	7.50	2.0	-0.22
8	7.49	2.8	-0.23
20	7.47	4.5	-0.25
30	7.46	5.5	-0.26
60	7.45	7.7	-0.27

Height before loading (mm)	21.16
Height after loading (mm)	20.89

Increment	Pressure (kPa)	Test Date
800-1400	1400	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.45	0.0	0.00
0.1	7.43	0.3	-0.02
0.2	7.41	0.4	-0.04
0.5	7.32	0.7	-0.13
1	7.27	1.0	-0.18
2	7.25	1.4	-0.20
4	7.23	2.0	-0.22
8	7.21	2.8	-0.24
15	7.20	3.9	-0.25
30	7.19	5.5	-0.26
60	7.17	7.7	-0.28
120	7.16	11.0	-0.29

Height before loading (mm)	20.89
Height after loading (mm)	20.60

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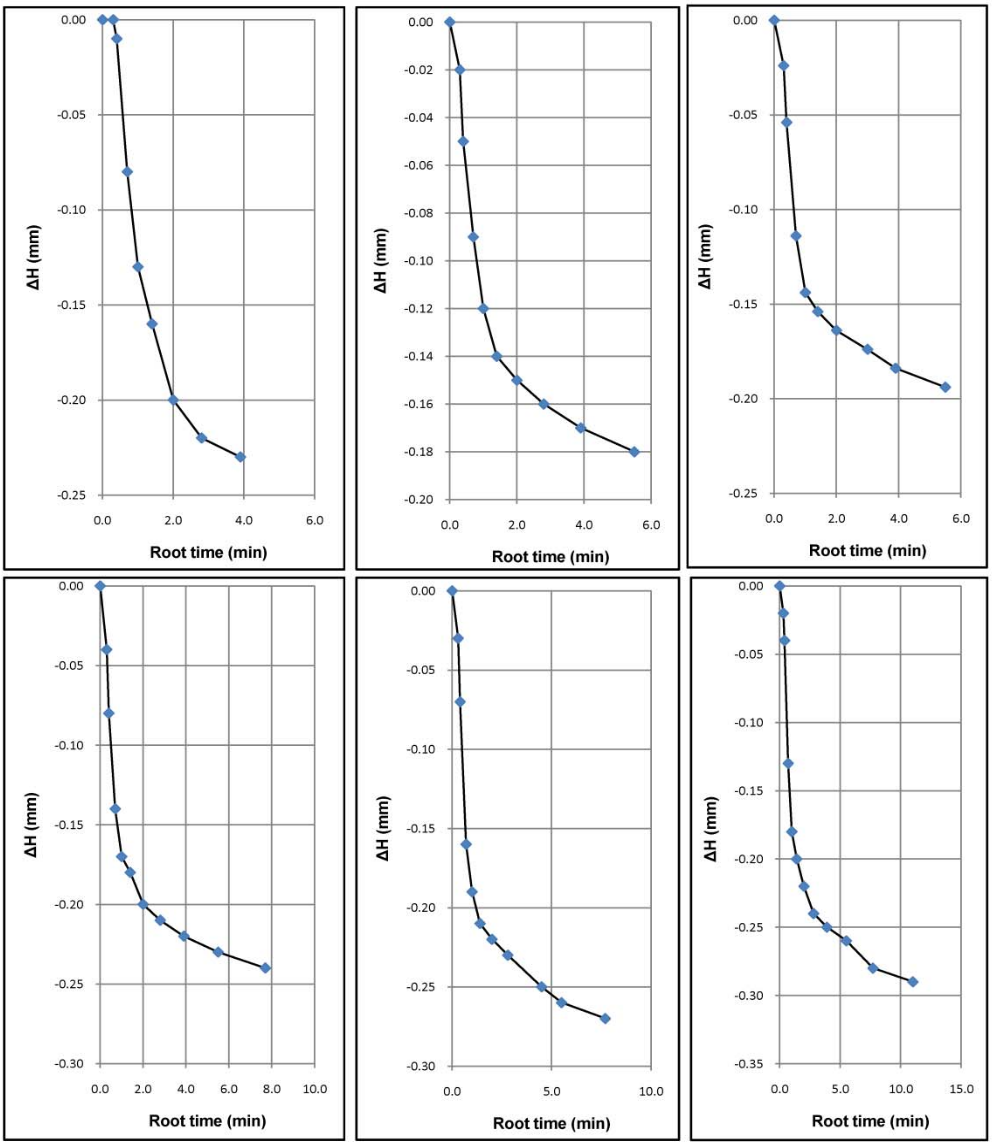
GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Hydraulic Consolidation Test

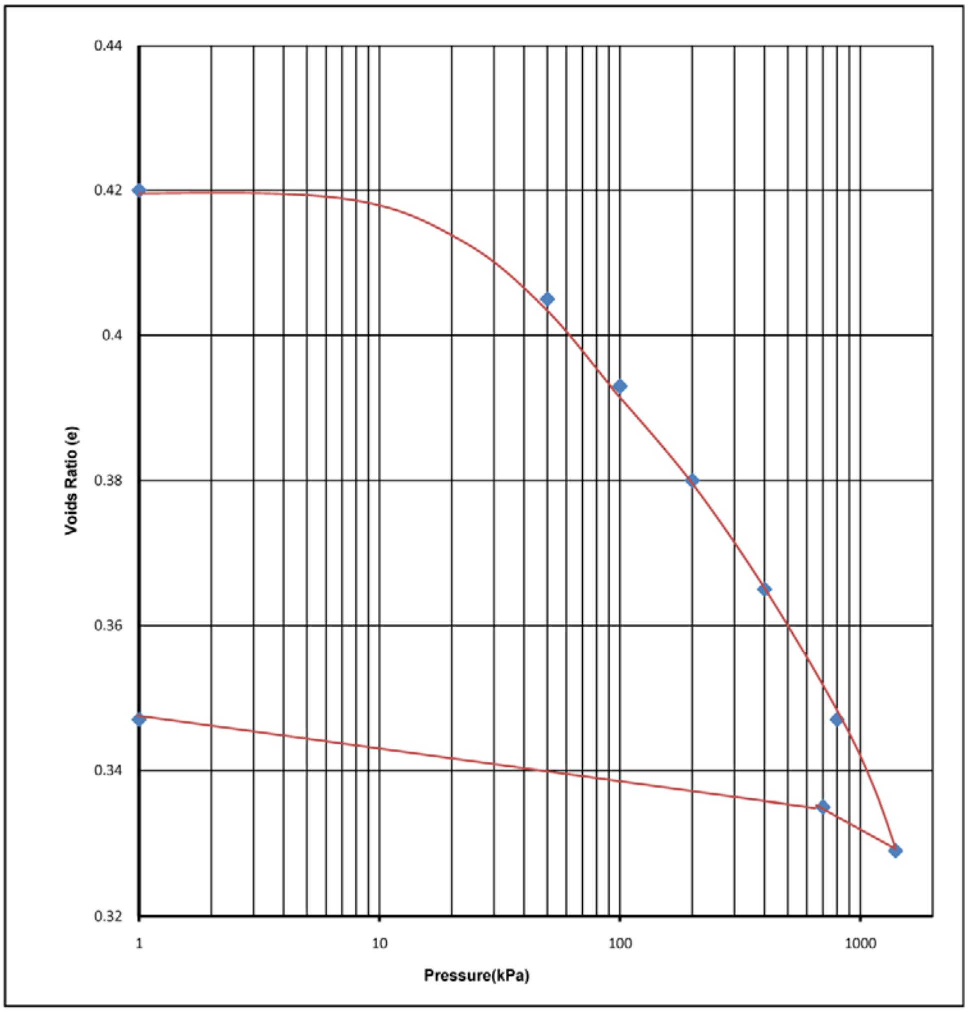
Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Sample No.:	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND



Hydraulic Consolidation Test

Project Name:	[illegible]	Lab. Room Temp.:	23°C
Proj. Ref. No:	[illegible]	Test Date:	[illegible]
Orientation:	Parallel To The Vertical.		
Borehole No.:	[illegible]	Sample No.:	[illegible]
Depth (m):	[illegible]	Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND



Preconsolidation Pressure (σ_c') = 85 (kPa)

Recompression Index (C_r) = 0.0022

Compression Index (C_c) = 0.069

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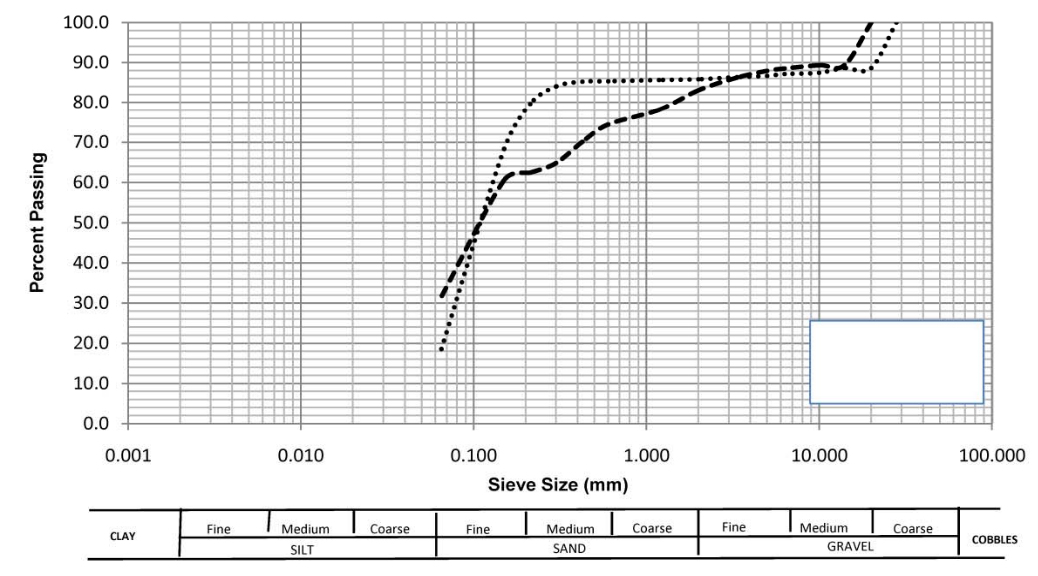
GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Grain Size Analysis (Sieve Test)

Project Name:		Proj. Ref. No.:	
Test Date:		Lab. Room Temp.:	23°C

Borehole No.:		Borehole No.:		Borehole No.:	-
Depth (m):		Depth (m):		Depth (m):	-
Sample Ref. No.:	2	Sample Ref. No.:	3	Sample Ref. No.:	-
Sample Type:	Small Disturbed	Sample Type:	Small Disturbed	Sample Type:	-
Sample Description:	Silty Sand	Sample Description:	Silty Sand	Sample Description:	-
Dry Sample Weight (g):	141.5	Dry Sample Weight (g):	156.8	Dry Sample Weight (g):	-

Sieve Size (mm)	Retained Weight (g)	% Passing	Sieve Size (mm)	Retained Weight (g)	% Passing	Sieve Size (mm)	Retained Weight (g)	% Passing
50.00	0.00	100.0	50.00	0.00	100.0	-	-	-
37.50	0.00	100.0	37.50	0.00	100.0	-	-	-
28.00	0.00	100.0	28.00	0.00	100.0	-	-	-
20.00	16.10	88.6	20.00	0.00	100.0	-	-	-
14.00	16.10	88.6	14.00	16.80	89.3	-	-	-
10.00	17.80	87.4	10.00	16.80	89.3	-	-	-
6.30	18.30	87.1	6.30	18.10	88.5	-	-	-
5.00	19.00	86.6	5.00	19.00	87.9	-	-	-
3.35	19.40	86.3	3.35	21.70	86.2	-	-	-
2.00	20.10	85.8	2.00	26.70	83.0	-	-	-
1.18	20.40	85.6	1.18	34.30	78.1	-	-	-
0.60	20.80	85.3	0.60	40.10	74.4	-	-	-
0.43	21.00	85.2	0.43	46.90	70.1	-	-	-
0.30	22.70	84.0	0.30	55.20	64.8	-	-	-
0.21	28.90	79.6	0.21	58.90	62.4	-	-	-
0.15	44.90	68.3	0.15	62.00	60.5	-	-	-
0.06	117.80	16.7	0.06	108.90	30.5	-	-	-



SJ-F-IMS-RRF-012, R-01, D-16/02/10

APPENDIX C5

PERMIT APPLICATION FORMS

ABU DHABI CITY MUNICIPALITY

DEPARTMENT OF MUNICIPAL AFFAIRS

APPENDIX D

EXAMPLE APPLICATION SUBMITTAL FOR DEWATERING OF INTERMEDIATE DEPTH AND COMPLEXITY

PROJECT DEWATERING REPORT

MEDIUM-DEPTH DEWATERING PROGRAM FOR CONSTRUCTION OF (*STRUCTURE NAME)*

ADDRESS OF PROJECT
ZONE NO./NAME
SECTOR NO.
PLOT NO.

PROJECT NO. XX
REVISION 1
XX JUNE 2014

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ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

PROJECT DEWATERING REPORT

MEDIUM-DEPTH DEWATERING FOR CONSTRUCTION OF (*STRUCTURE NAME*)

1.0 INTRODUCTION

1.1 PROJECT UNDERSTANDING

The Project consists of dewatering for the proposed sewage connection located at *(location of structure)*. The scope involves the dewatering of the site utilizing a sump and trench dewatering system to enable construction to be carried out in dry, safe conditions.

The dewatering system installation shall be dependent on the site access and working space available. The dewatering system needs to be installed and operational before the excavation begins. After the Method Statement (***Section 4.0***) has been approved by the Abu Dhabi City Municipality (ADM), the Main Contractor should be given instruction to begin mobilizing.

1.2 PROJECT ENTITIES: OWNER, CONTRACTOR, AND DEWATERING SUBCONTRACTOR

It is the intention of the Dewatering Subcontractor to coordinate closely with the Main Contractor in terms of mobilization, installation, commissioning, operation, and maintenance of the dewatering system. Relevant project entities are as follows:

Owner:	(Name and Address of Owner)
Main Contractor:	(Main Contractor name)
Dewatering Subcontractor:	(Dewatering Subcontractor name)

1.3 HEALTH AND SAFETY PROGRAM

Personnel Health and Safety will be protected under the Main Contractor's and Dewatering Subcontractor's Health and Safety Plans. Potential risks to personnel safety include: excavation collapse, falls into the excavation, risks due to working in confined spaces (air quality, entrapment), and risk of electric shock due to wiring and power supply to the sump pumps. These risks and control measures are discussed in the Risk Analysis provided in ***Appendix D3***.

1.4 Environmental Protection Program

Owner's environmental guidelines and standards have been considered throughout the design of the proposed system. Environmental protections will be provided in accordance with local regulations and Dewatering Subcontractor, Inc.'s standard environmental protection plans.

To limit emissions on the site, electric motors shall be used as prime movers, where possible allowing for the concentration of diesel prime movers at dedicated control stations. Owner shall be responsible for all permits associated with dewatering.

Diesels and oils shall be restricted within drip trays to prevent contamination of the sand. Bulk storage fuel tanks shall be bundled to contain waste or spillage.

All dewatering system effluent shall pass through a settlement tank before discharge into the approved collection area, or approved stormwater discharge manhole.

An outline of environment hazards is included in the Risk Analysis provided in ***Appendix D3***.

1.5 Quality Assurance/Quality Control Management Program

Quality Assurance/Quality Control (QA/QC) Management will be provided under Dewatering Subcontractor's internal processes and will be the responsibility of the Site Manager.

2.0 SITE SUBSURFACE CONDITIONS

2.1 DESCRIPTION OF SITE GEOLOGIC SETTING AND HYDROGEOLOGY[1]

Generally, Abu Dhabi Emirate is divided into four physiographic regions, each with unique topographic, geologic, geomorphic, and hydrologic characteristics. Namely, these regions include the Al Hajar Mountains, the wadis and alluvial fans along the flanks of the Al Hajar, the vast desert region of sand dunes and inland sabkhas (the largest area within the Abu Dhabi Emirate), and the coastal region that includes sand dunes, coastal sabkhas, lagoons, and tidal zones. Overall, this coastal physiographic region is characterized by the lowest elevations (typically less than 20 meters mean sea level [m msl]) and flattest surfaces in the Emirate, and contains a higher proportion of sabkha (coastal sabkha). The ADM lies primarily within the coastal region but also the desert region in its southern and eastern extents.

Erosion and fluvial transport of sand, gravel, and boulders down the sides of the Al Hajar and in mountain valleys (wadis) have resulted in the formation of extensive outwash sand, gravel deposits, and alluvial fans that empty out onto the desert floor (Kumar et al., 2008). Within the eastern ADM, these alluvial and wadi land surfaces are covered with a vast expanse of desert filled with sand dunes and inland sabkhas (Styles et al., 2006). Along the Arabian Gulf coastline, which includes most of the ADM, the land surface contains sand dunes, beach sand deposits, lagoonal silts and clays, and coastal sabkha deposits (Farrant et al., 2012a).

In the ADM, Precambrian (Proterozoic) basement rocks lay approximately 9 km below ground level (BGL) (Farrant et al., 2012a). Overlying the crystalline basement in the United Arab Emirates (UAE) is a thick sequence of shallow, nearly flat-lying marine sedimentary rocks that include up to 2,500 m of Cambrian to Carboniferous aged primarily clastic rock, over which was deposited approximately 4,300 m of mainly limestone, dolomite, mudstone, and anhydrite of mid-Permian to Cretaceous age (Styles et al., 2006). Following a period of erosion and non-deposition that created a regional unconformity at the end of the Mesozoic Era, significant amounts of shale, mudstone, argillaceous limestone, and anhydrite layers

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[1] This Section should be project specific. As no location data is presented for the example, only a general description of geologic setting is provided here.

were deposited from the Paleogene to mid-Miocene (Farrant et al., 2012a). The Gachsaran Formation is the uppermost of these younger rock strata in the ADM.

The overlying formations represent the shallow groundwater flow system within ADM, and are the most important relative to fresh groundwater supplies in the Emirate and to the geotechnical and engineering problems associated with construction and dewatering in the ADM. Overlying the Gachsaran in the northern and eastern parts of the ADM are the distal ends of wadi and alluvial fan deposits of the Barzaman Formation and the Hili Formation (Farrant et al., 2012b). Barzaman rocks grade laterally to the south and to the west into the Baynunah formation whereas the Hili deposits are younger and cut down through the older Barzaman or Baynunah deposits (Farrant et al., 2012b). These units make up the upper bedrock surface within the ADM.

The late Pleistocene (Quaternary) age, weakly- to well-cemented, aeolian sandstone deposits of the Ghayathi Formation overlies the above-mentioned formations (Farrant et al., 2012b). the Ghayathi is in turn overlain by younger, late Pleistocene to Holocene age unconsolidated dune deposits of the Rub al Khali Formation that includes thick deposits of fine- to medium-grained sands that are carbonate-rich near the coast and quartz-rich inland (Farrant et al., 2012b). Within many inland areas, the younger Rub al Khali sands have been deflated down to the water table, where these flat, low-lying areas frequently becomes cemented with halite and gypsum, forming sabkhas. Along the coast, and extending a few kilometers inland in the ADM, the coastal zone sand dunes, beach sands, sabkha, lagoonal muds, intertidal algal mats, and other near-shore marine deposits of late Pleistocene to Holocene age are collectively referred to as the Abu Dhabi Formation (Farrant et al., 2012b).

It is important to note, however, that much of the ADM includes land areas that have been reclaimed by sabkha infilling, or that have been formed by the artificial expansion of existing islands using fill material. Fill material utilized in the development of this “made ground” is typically dredged sediment from near-shore shipping channels, near-shore sand dunes, or fill material transported from greater distances.

The top of bedrock elevations, within the ADM, range from roughly -20 to 100 m msl. Generally, the lowest top of bedrock surface elevations (-20 to 10 m msl) are located in the western ADM, but rise inland, effectively mimicking surface elevation changes. In the core geotechnical hazard area, bedrock elevations range from approximately -15 to 55 m msl, with top of rock elevations being highest (10 to almost 55 m msl) to the east.

Total thickness of the unconsolidated sediments and fill material (made ground) within the ADM region (i.e., Quaternary aquifer) ranges from 0 to about 24 m, although in most areas, the overburden thickness ranges only from 0 to 10 m. In the core geotechnical hazard zone, the total thickness of unconsolidated sediments is typically 15 m or less.

Interpreted potentiometric surfaces from waterstrike data indicates that groundwater elevations in the western and north-central ADM are relatively flat, ranging from approximately -15 up to about 10 m msl. Groundwater elevations generally increase in an easterly direction, mimicking ground surface elevations with the relatively highest groundwater levels (roughly 70 to 102 m msl) observed in the southeastern ADM. In the westernmost core ADM area, groundwater elevations are relatively flat, ranging from approximately -15 to 5 m msl while in the eastern core area, groundwater elevations appear much higher, between about 5 and 45 m msl.

2.2 DESCRIPTION OF GEOTECHNICAL CONDITIONS

Geotechnical investigations and testing of the subsurface materials and ground water were performed in accordance to the recommendations given in the Field Investigation Scope (Table 11-3) of the ADM Guidelines and the associated key shown below. The investigation includes 6 borings at 50 m on center which meets the five boring requirement of the ADM Guidelines. The borings were drilled using rotary drilling techniques. Three borings extended to 12 m (1.5x depth of excavation) and three to 16 m (2x depth of excavation). A slug test was performed in the sand layer, to determine permeability.

FIELD INVESTIGATION SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis

FIELD INVESTIGATION SCOPE
(CONTINUED)

KEY		
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspections of boring required
		Visual inspections of boring not required

EXCERPT FROM TABLE 11-3 OF THE ADM GUIDELINES								
HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	BOREHOLE DISTRIBUTION (< 10000 m^2)	VISUAL INSPECTIONS	LAB TESTING[3]	FIELD TESTING	THIRD PARTY REVIEW
B	1,2	i	Shallow	5 [1,2]	X	1 [4]	Slug Test	
		i, ii, iii	Medium	5 [1,2]		1 [4]	Packer Test[5]	X
			Deep	5 [1,2]		1 [4]	Pumping Test	X
	3	i	Shallow	5 [1,2]	X	1 [4]		
		ii, iii	**Medium**	**5 [1,2]**	**X**	**1 [4]**	**Slug Test**	
		i, ii, iii	Deep	5 [1,2]		1 [4]	Packer Test[5]	X

Notes:

1 Two thirds of the boreholes should be up to 1.5 x depth of excavation and the remaining boreholes up to 2 x depth of excavation.

2 One borehole each at the corners and one at approximate center location or at a spacing not exceeding 50 m c/c. For soil and ground water testing refer to ***Section 11***.

3 Sieve analysis and Atterberg Limits (Soil Classification, e.g., USCS).

4 One test per geologic layer (based on geologist's description) but no less than one test per 3 m of depth.

5 Packer tests are performed in rock formations only. If not applicable, a slug test is recommended.

Per laboratory testing, soil encountered was silty sand (SM) to approximately 4.5 m below ground surface, underlain by slightly weathered calcarenite up to a depth of 10.0 m below ground surface. No soluble minerals were identified in the soils report.

The groundwater table is at a depth of 1.5 m below the existing ground level.

Details on the Geotechnical Investigation are presented in ***Appendix D4***[2].

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2 For the sake of brevity, only select pages are provided in this example. The full Geotechnical Investigation Report is required in an actual application, to be provided as an Appendix to the application.

3.0 DEWATERING SYSTEM DESIGN

3.1 SUMMARY OF DESIGN CALCULATIONS

The site is in Hazard Zone B away from sensitive buildings. For a medium-depth excavation in this location, only hand calculations are required for both pumping capacity and settlement analysis, per the Dewatering Design Scope (Table 10-3) of the ADM Guidelines (see key and excerpt from Table 10-3 below). These calculations are provided in ***Appendix D2***.

DEWATERING DESIGN SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspections of boring required
		Visual inspections of boring not required

DEWATERING DESIGN SCOPE
(CONTINUED)

EXCERPT FROM TABLE 10-3 OF THE ADM GUIDELINES						
HAZARD ZONE	**PROXIMITY OF STRUCTURES**	**EXCAVATION/ DEWATER**	**EXCAVATION DEPTH**	**PUMPING CAPACITY ANALYSIS**	**SETTLEMENT ANALYSIS**	**THIRD PARTY REVIEW**
B	1,2	I	Shallow	a	I	
		i, ii, iii	Medium	b	I	X
			Deep	c	II	X
	3	I	Shallow	a	I	
		ii, iii	Medium	a	I	
		i, ii, iii	Deep	b	II	

3.2 EVALUATION OF POTENTIAL SITE IMPACTS DUE TO DEWATERING

The Site is expected to have medium risk of detrimental effects from a properly designed and operated dewatering system. The site is away from sensitive structures, but because it is in an urban area, settlement due to soil collapse has the potential to cause minor impacts to the project area if they occur. The following sections discuss several typical site impacts caused by a trench and sump dewatering system.

3.2.1 Settlement and Soil Collapse

Settlement due to soil collapse is possible when dewatering operations cause the migration of fine materials from the surrounding soil through the dewatering system, or where the flow of water through the surrounding soil encounters soluble minerals, such as sabkha or gypsum. The primary cause of settlement, however, is the increase in effective stress due to dewatering. Differential settlement is of special concern.

Risk due to migration of fine materials will be mitigated by the presence of filter zones and by the presence of the Site Manager, who will inspect flows for signs of turbidity and take appropriate measures if excessive turbidity occurs. The gravel backfill around the deep sumps, the wellpoints, and the manhole locations will act as a filter and will prevent collapse of the sump during construction.

The consolidation settlement calculation is provided analytically in ***Appendix D2***.

3.2.2 Excavation Slope Stability

Dewatering to levels below the planned excavation elevation is required to maintain slope stability in the sandy materials at the site. Groundwater levels in the excavation area will be observed and monitored throughout the dewatering process. Slope stability is to be verified by the excavation design engineer using results from this dewatering design before construction. Open excavation may be allowed if it remains within the property limits, i.e., the excavation shall not encroach into any neighboring property.

Material stockpiles, vehicular traffic, and other loads near the edge of the excavation can cause slope instability and excavation collapse. To mitigate this risk, vehicles or material will not be allowed within 1 m of the top of the excavation.

Because the site is in a developed area where most ground surfaces are impermeable, slope stability could be negatively impacted by flows into the excavation from other sources at the surface. To mitigate this risk, a sand bund will be established around the excavations.

Instrumentation for measuring slope movements is provided near the sensitive structure, as required by the ADM Guidelines for this type of project. Survey monuments for measuring deformation are specified every 50 m, as required by the Dewatering Monitoring Scope (Table 12-2) of the ADM Guidelines (see key and excerpt from ADM Guidelines Table 12-2 in ***Section 3.2.3***). Instruments for monitoring slope stability are placed on steep sections of slope as required in the Dewatering Monitoring Scope (Table 12-2) of the ADM Guidelines.

3.2.3 Migration of Fine Materials

As discussed in ***Section 3.2.1***, migration of fine materials is a primary cause for excessive settlement due to soil collapse. In addition, discharge of highly turbid water into the adjacent storm sewer system is prohibited by the site permit. Discharge of highly turbid water will be prevented by the filter zone around the sump as described in ***Section 4.0*** and by daily observation of dewatering discharges. Flow will be measured and monitored around each sump along the excavation trench. Water level is monitored by piezometers per the Dewatering Monitoring Scope (Table 12-2) of the ADM Guidelines (see excerpt from Table 12-2 below and table key in ***Section 2.2***). In this application, the radius of influence is calculated to be less than 200 m so piezometers (and survey monuments) are only placed within 200 m of the excavation on 50 m centers.

DEWATERING MONITORING SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspections of boring required
		Visual inspections of boring not required

DEWATERING MONITORING SCOPE
(CONTINUED)

EXCERPT FROM TABLE 12-2 OF THE ADM GUIDELINES								
HAZARD ZONE	**PROXIMITY OF STRUCTURES**	**EXCAVATION/ DEWATERING TYPE**	**EXCAVATION DEPTH**	**WATER LEVEL / TURBIDITY / TDS**	**SURVEY/ DEFORMATION**	**SETTLEMENT**	**SLOPE MOVEMENT**	**FLOW MEASUREMENT**
B	1,2	i	Shallow					
		i, ii, iii	Medium	4 [1]	1 [2]	1 [2]	1 [2]	X
			Deep	4 [1]	1 [2]	1 [2]	1 [2]	X
	3	i	Shallow					
		ii, iii	**Medium**	**4 [3]**	**1 [4]**	**-**	**1 [2]**	**X**
		i, ii, iii	Deep	4 [3]	1 [4]		1 [2]	X

Notes:

[1] One piezometer each at the corners or at a spacing not exceeding 50 m c/c.

[2] One instrument on or near to each sensitive structures/steep section of the slope.

[3] One piezometer each at the corners or at a spacing not exceeding 50 m c/c within 200 m of periphery of the excavation and at 100 m c/c between 200 m to cone of depression.

[4] Install Survey Monument at a spacing not exceeding 50 m c/c within 200 m of periphery of the excavation and at 100 m c/c between 200 m to cone of depression.

The filter zone will be designed according to Section 4.3 of the Dewatering Guidelines and properties of the silty sand material surrounding the filter zone. Uniformity Coefficient (C_u) and D_{50} are determined from Sieve Analysis results such as those presented for the silty sand in ***Appendix D4***. In this case, D_{50} of the silty sand is around 0.24 mm and C_u is conservatively around 7. Since C_u is generally greater than 7, the D_{50} of the filter pack should be 7 to 8 times D_{50} of the silty sand. The uniformity coefficient, C_u, of the filter pack material must be smaller (more uniform) than C_u of the silty sand.

بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

4.0 CONSTRUCTION METHOD

4.1 PURPOSE, SCOPE, AND DESIGN BASIS

This Method Statement describes the activities and procedures for the dewatering of the proposed structure at (location), Abu Dhabi. The scope involves the dewatering of the proposed site to enable construction to be carried out in dry, safe conditions by a single-stage wellpoint dewatering system and collection trenches/pits.

The works for the proposed development involves the excavation as per the following information:

- Area for Dewatering: 12 m x 100 m approximately
- Ground Level: +0.00 m
- Existing Water Table level: approximately 1.5 m Below Ground Level (BGL)
- Max Excavation level: approximately 8.00 m BGL
- Shoring: None

4.2 SUMMARY OF PROPOSED DEWATERING SYSTEM

Prior to excavation, it is proposed to install a single-stage wellpoint system to control the groundwater within the excavation area and to provide initial lowering of the groundwater table to allow for installation of sumps at the deeper locations of the excavation. Wellpoint dewatering involves the installation of riser pipes with a filter section on the lower portion, connected into a common header pipe from which the water can be pumped by a vacuum assisted pump to a convenient discharge point. Polyvinyl chloride (PVC) riser pipes shall be inserted in predrilled holes, which are widened and cleaned through the use of high pressure water before adding filter sand around the PVC riser pipes.

When the wellpoint system has lowered the groundwater sufficiently, excavation will continue below the water level. A collection trench/pit will be placed to one side of the excavation and sumps will be installed in areas requiring locally deeper excavations. Sumps will be backfilled with clean gravel around a perforated pipe that will house the electric sump pump.

4.2.1 Physical Arrangement of Dewatering Equipment

Wellpoints shall be installed at 1.00 m intervals on a bench up to 2.5 m below the existing ground surface around the perimeter of the excavation. Sumps will be installed at the locations of deep excavations next to manholes. The dewatering equipment will be arranged as shown in ***Appendix D1***.

4.2.2 Installation Method and Required Equipment

Dewatering with wellpoints prior to excavation:

- Wellpoints will be drilled by augers and/or high-pressure water jet and fitted with riser pipes at sufficient intervals (1.0 m) to maintain the surrounding water table below the level of excavation.
- After wellpoints are driven into the ground, they are surrounded with a sand filter to facilitate suction.
- Flexible elbows are used to connect the riser pipes to header pipes, which in turn are connected to suction pumps.
- The water is then pumped through flexible hoses, PVC pipes, or discharge lines to the discharge manholes approved by concerned authorities.
- Once the water table has sufficiently dropped, the excavation may proceed in dry soil.
- To check the water level, a piezometer will be provided in each section (location must be approved first by the Engineer).
- Settlement gauges will be provided at several locations to monitor settlement of new and existing facilities (location must be approved first by the Engineer).

Dewatering with sumps after excavation:

- A collection trench and/or pit is placed to one side of the excavation.
- Sumps will be constructed by excavating 1.0 m diameter pits in the bottom of the excavation to a depth suitable to lower the groundwater table below the base of the excavation.
- Sumps will be backfilled with clean gravel around a perforated pipe that will house the electric sump pump.
- The water table will be maintained at least 300 millimeters (mm) below the formation level of the excavation during construction activities.

As a means of ensuring the integrity of the wellpoint installation, boreholes shall be predrilled to the required depth at1.00 m intervals. Predrilling shall be executed by the use of a self-propelled drilling rig and continuous flight augers. After completion of predrilling, wellpoints and a jetting lance will be inserted and high-pressure water shall be injected into the ground through the jetting lance, widening and cleaning the borehole. This development of wells is important to maximize the effectiveness of the system and to produce clean discharge.

4.2.3 Operations, Monitoring, and Maintenance during Construction

As required for a medium-depth excavation in Hazard Zone B near sensitive or large structures, instrumentation for measuring the groundwater level, deformation and slope movements are provided. The instruments shall be regularly monitored to ensure safety of excavation and the dewatering system. Instrumentation for monitoring includes:

- One piezometer and one survey monument at each corner or at a spacing not exceeding 50 m on center within 200 m of excavation periphery.
- One piezometer and one survey monument up to 100 m on center between 200 m of the excavation periphery and the radius of influence.
- One instrument on or near each sensitive building or steep section of slope.

In addition to instrument monitoring, visual inspection will be performed by the Site Manager during the operations and maintenance of the wellpoint dewatering system and will consist of the following daily checks:

- Dewatering pump operation and condition, including performance of any required maintenance or repairs.
- Daily monitoring of dewatering system effluent to identify excessive turbidity in flows. If excessive turbidity is noted, the Site Manager will adjust the dewatering system to limit the migration of fine materials by throttling or shutting off specific wellpoints.
- Visual inspection of the surrounding area and excavation slopes for signs of excessive settlement, slope instability, or excessive seepage into the excavation.

No overhead obstructions are expected. Underground structures have been identified and excavation will proceed with caution in the vicinity of expected underground structures.

5.0 REFERENCES

Farrant, A.R., R.A. Ellison, J.W. Merritt, J.E. Merritt, A.J. Newell, J.R. Lee, S.J. Price, R.J. Thomas, and A. Leslie, 2012a, “Geology of the Abu Dhabi 1:100,000 Map Sheet, 100-16, United Arab Emirates,” Ministry of Energy, United Arab Emirates, 69 p.

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APPENDICES

APPENDIX D1

DRAWINGS

DRAWN BY	ASH	CHECKED BY	EOT		CAD FILE NUMBER	168024-334
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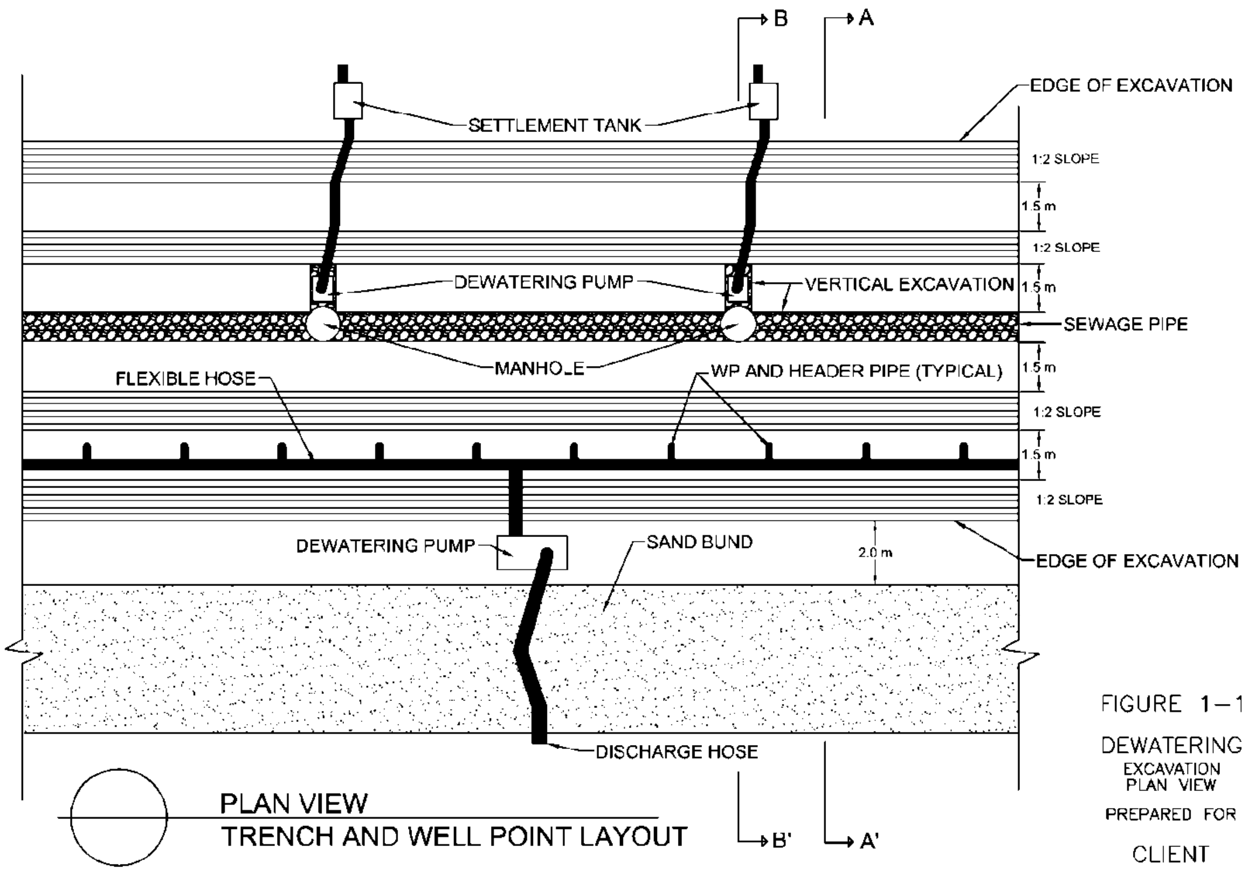

FIGURE 1–1

DEWATERING
EXCAVATION
PLAN VIEW

PREPARED FOR

CLIENT
LOCATION

CONTRACTOR, Inc.
ENGINEERS / CONSULTANTS

دائرة الشؤون البلدية
بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

					CAD FILE NUMBER 168024–335
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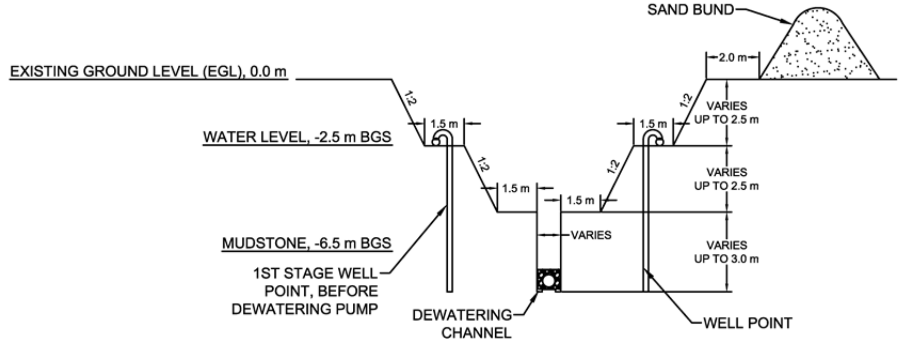
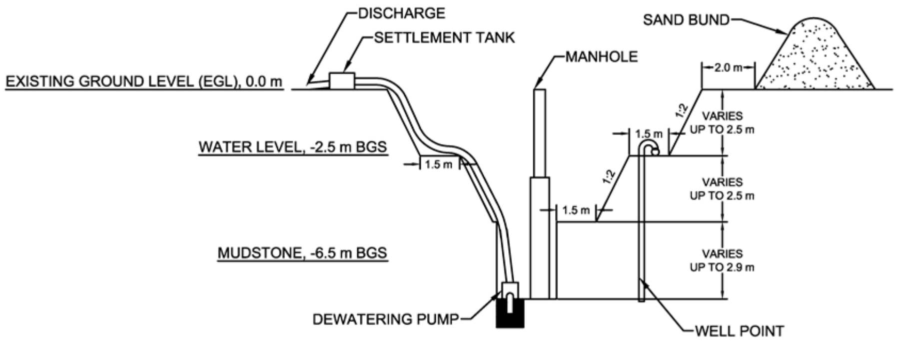
	BMW	APPROVED BY	MJE		



A SECTION A
SECTION THROUGH TRENCH



B SECTION B
SECTION THROUGH MANHOLE

FIGURE 1–2

DEWATERING
EXCAVATION AND
WELL POINT DETAILS
PREPARED FOR

CLIENT
LOCATION

CONTRACTOR, Inc.
ENGINEERS / CONSULTANTS

APPENDIX D2

CALCULATIONS

APPENDIX D2

CALCULATIONS

According to the ADM Guidelines for a medium-depth excavation in Hazard Zone B that is away from sensitive or large structures, hand calculations are acceptable for pumping capacity and estimated settlement.

ASSUMPTIONS

1. The well points are installed into the mudstone assuming the mudstone is permeable.
2. Total head in the aquifer is estimated assuming the depth of the impermeable layer is unknown.
3. Analytical solutions assume homogenous, isotropic material whereas the subsurface in Abu Dhabi is layered. Numerical models do consider this layering.
4. Parameters for permeability, void ratio, specific gravity, and compression index in the analytical solution are assumed for the surficial soil layer and extended to the mudstone. This is considered conservative because the mudstone will be less permeable (lower flow rate) and less compressible (lower settlement) than the surficial soil.
5. One-dimensional consolidation settlement is assumed as an approximation of settlement from dewatering.

CALCULATION INPUT

Refer to ***Appendix D1*** for site geometry. Refer to ***Section 2.2*** and ***Appendix D4*** for subsurface conditions. ***Table 1*** summarizes parameters used in this calculation.

TABLE 1
PARAMETERS USED IN PUMP CAPACITY AND SETTLEMENT HAND CALCUATIONS

PARAMETER	VALUE	JUSTIFICATION
Width of Excavation	12 m	From project drawings
Length of Excavation	100 m	Distance between sumps
Water Depth (BGL)	2.5 m	From Soils Report
Excavation Depth (BGL)	8 m	Max, from project drawings
Required drawdown	6 m	Required at least 0.5 m below excavation by ADM Guidelines
Drawdown Depth (BGL)	8.5 m	Required at least 0.5 m below excavation by ADM Guidelines
Permeability	1E-5 m/s	From Slug Test (Variable Head Permeability Test), ***Appendix D4***
Void Ratio	0.42	From consolidation test in ***Appendix D4***
Saturated Unit Weight, Sand	21.0 kN/m^3	From consolidation test in ***Appendix D4***
Dry Unit Weight, Sand	16.8 kN/m^3	From consolidation test in ***Appendix D4***
Compression Index, Sand	0.069	From consolidation test ***Appendix D4***

PUMPING CAPACITY ANALYTICAL SOLUTION

The calculation for pumping capacity is based on the ADM Guidelines and the dewatering site is simulated as a trench. The flow rate required for the dewatering system is calculated assuming flow from a line source to a parallel trench at a distance L away.

The distance of a line source and radius of influence from drawdown are estimated using ***Equations 1 and 2***.

$$R_0 = C * (H - h) * \sqrt{k} \quad \text{(Equation 1)}$$

$$L = R_0/2 \quad \text{(Equation 2)}$$

where,

C = empirical calibration factor = 1500 for line flow to trenches

$H - h$ = target drawdown of the excavation = 7 m

k = permeability coefficient = 1E-05 m/s

R_0 = radius of influence = 33.2 m

L = distance from a line source to a parallel dewatering trench = 16.6 m

Using the dewatering equation for a drainage trench in a water table aquifer, the flow rate is calculated as in ***Equation 3***.

$$\frac{Q}{x} = \frac{k\,(H^2 - h_w^2)}{2L} \qquad \text{(Equation 3)}$$

where,

H = hydraulic head of the original water table = 12.5 m (assumed 15 m head at surface)

h_w = hydraulic head at bottom of wellpoint = 6.5 m

L, k = defined previously

Q/x = flow rate into unit length drainage trench from one side = 0.00004 m^3/s

x = unit length of drainage trench = 100 m

2Q = flow into full drainage trench from both sides = 0.008 m^3/s

SETTLEMENT ANALYTICAL SOLUTION

Settlement from dewatering occurs from the consolidation of compressive sands due to an increase in effective stress, even if dewatering is carried out properly. Dewatering removes buoyancy from the soil, increasing the effective stress as total stress remains constant. Estimated settlement is calculated by finding effective stresses at the static and drawn down water levels.

Total and effective stress is defined under initial conditions, before dewatering, as in ***Equations 4 and 5***. Final effective stress, after dewatering, is calculated in ***Equation 6***.

$$\sigma_0 = h_{WL} * \gamma_d + (h_r - h_{WL}) * \gamma_s \qquad \text{(Equation 4)}$$

$$\sigma'_0 = \sigma_o - (h_r - h_{WL}) * \gamma_w \qquad \text{(Equation 5)}$$

$$\sigma'_f = h_d * \gamma_d + (h_r - h_d) * (\gamma_s - \gamma_w) \qquad \text{(Equation 6)}$$

where,

γ_d = dry unit weight = 16.8 kN/m^3 from ***Appendix D4***

γ_s = saturated unit weight = 21.0 kN/m^3 from ***Appendix D4***

γ_w = unit weight of water = 9.8 kN/m^3

h_{WL} = depth of original water level below ground surface = 2.5 m

h_d = depth of water level after dewatering = 6.5 m (equals h_r if greater than h_r)

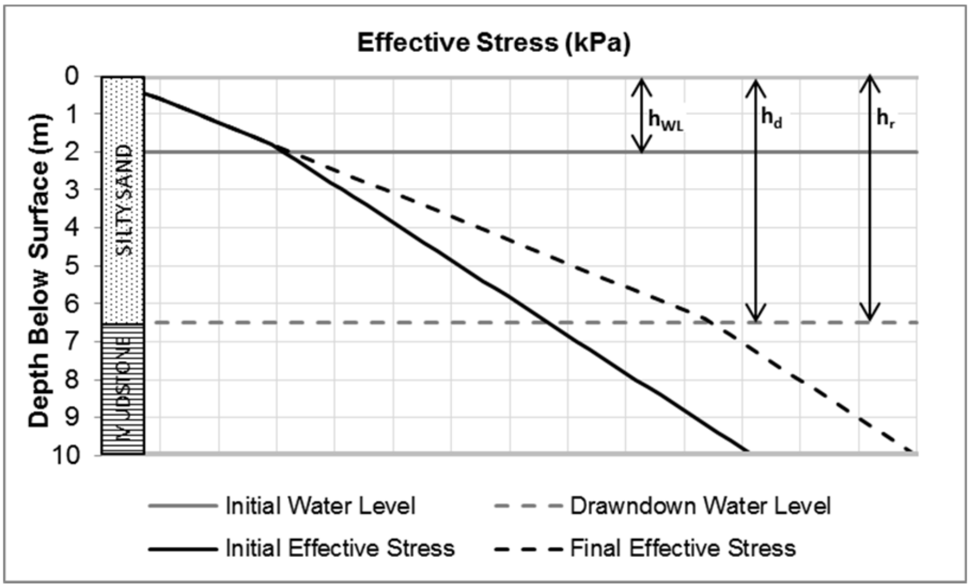
h_r = thickness of soil layer to calculate settlement = 6.5 m (thickness of surficial sand layer)

σ_0 = total stress under initial conditions

σ'_0 = effective stress under initial conditions = 86.7 kN/m^2

σ'_f = final effectives stress = 109.0 kN/m^2

FIGURE 1
GRAPHICAL SCHEMATIC OF INITIAL AND FINAL EFFECTIVE STRESS



Estimated settlement is calculated in ***Equation 7*** from the compression index, void ratio, and the log difference of final and initial effective stresses. The silty sand layer is broken down into two layers, above the initial water table and below. Above the initial water table, there is no change in effective stress from initial to final stress, and therefore no settlement. The surcharge from the layer above the water table is included in both initial and final stress of the layer below, and is, therefore, canceled out. Settlement in the layer below the initial water table is as follows.

$$\Delta S = \frac{Cc * h_r * \log(\sigma'_{fm}/\sigma'_{0m})}{(1+e)} \qquad \text{(Equation 7)}$$

where,

σ'_{0m} = initial effective stress, mid layer, plus surcharge from layer above = 64.3 kN/m^2

σ'_{fm} = final effectives stress, mid layer, plus surcharge from layer above = 75.5 kN/m^2

Cc = compression index = 0.069

h_r = thickness of soil layer to calculate settlement = 6.5 m

e = void ratio = 0.42

ΔS = estimated settlement of soils affected by dewatering = 13.5 mm

RESULTS AND DISCUSSION

Calculated flow rate from both sides of the trench will be approximately 0.008 m^3/s for every 100 m section. Maximum settlement from dewatering effects is estimated at 13.5 mm. For more accurate settlement estimation, a numerical model should be developed, however, since no sensitive buildings are in the area, the 1-D consolidation equation is considered sufficient. Migration of fine materials and collapse of karst may also contribute to settlement. Mudstone may also contribute to settlement from dewatering, but not as much as the silty sand. Settlement in mudstone is not included in this calculation but may be included if the Main Contractor deems necessary. Slope stability will also need to be verified with the implications of this dewatering design.

REFERENCE

1. Bowles, J. E., *Foundation Analysis and Design,* The McGraw-Hill Companies, Inc., 1996, Sections 2.10 and 5.12.

APPENDIX D3

RISK ANALYSIS

APPENDIX D3

RISK ANALYSIS

Risk analysis involves considering and rating potential hazards for the duration of the dewatering activity. Hazards are assigned a consequence and likelihood rating according to the following guidelines:

RISK ANALYSIS RATING

CONSEQUENCE RATING	SAFETY	HEALTH	ENVIRONMENT
1	First Aid	Immediate	Minor
2	Medical	Temporary	Short-term
3	Lost time	Short-term	Long-term
4	Disability	Long-term	Serious
5	Fatality	Fatal	Catastrophic

LIKELIHOOD RATING	FREQUENCY
1	Highly Unlikely
2	Remote
3	Possible
4	Probable
5	Certain

The risk factor is the product of the consequence and likelihood rating. A risk factor 1 to 4 is low risk, 5 to 12 is medium risk, and 13 to 25 is high risk.

(Address the consequence and likelihood of the following events and any other relevant situation with potential to be hazardous. Suggest control measure and address the consequence and likelihood with the proposed control measures in place.)

The following hazards are considered for this project:

1. Working Inside Excavation Area

   a. Excavation Cave-in or Soil Collapse

b. Atmospheric Gases Failure

c. Injuries caused by pump moving parts

d. Improper means of access/egress

2. Working in Open Area

   a. Working in high temperatures

3. Drilling and Installation of Pipe

   a. Damage to underground services

   b. Noise and entanglement

   c. Contact with overhead power lines and other overhead structures

4. Working adjacent to traffic

   a. Moving vehicle/equipment

5. Dewatering

   a. Increased noise level

   b. Greenhouse gas emissions

   c. Oil/fuel leakages polluting soil and water

6. Storage and use of construction vehicles and machinery

   a. Increased noise level

   b. Greenhouse gas emissions

   c. Engine fluids leakages

7. Drilling

   a. Noise and smoke emissions affecting the environment and nearby workers

Using Activity 7a as an example, potential hazards are environmental pollution and physical effect to workers nearby. The consequences of these hazards are long-term health effects or serious environmental effects (Consequence Rating = 4) and the likelihood is possible (Likelihood Rating = 3). The risk factor before control measures of this hazard is 12, which falls in the high-risk category.

The following control measures for hazards in 7a are suggested:

1. Plant and equipment will be used on an intermittent basis, and will be shut, or throttled down when not in use.

2. Daily inspections and repairs when appropriate should be made to ensure that equipment is not emitting unusual noise.

3. Noisy machinery will be provided with silencer.

4. Damaged machinery will be repaired immediately in the work shop.

5. Damaged machinery will not be allowed to work on site.

If these control measures are implemented, the possible consequences are reduced to temporary health effects and short-term environmental effects (Consequence Rating = 2) and the likelihood of the event occurring is highly remote (Likelihood Rating = 2). The risk factor of this hazard after implementing control measures is now 4, which falls in the low risk category.

APPENDIX D4

GEOTECHNICAL INVESTIGATION REPORT

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APPENDIX D4

GEOTECHNICAL INVESTIGATION REPORT

Project Name : Deep Tunnel Sewer T-02.	**Project Ref No** : 001-10	**Borehole No.** AB 51B	SPEKTRA JEOTEK Geotechnical Engineers
Location : Abu Dhabi	**Client** : Impregilo		
Drilling Method : Triple Tube Rotary Wireline.			
Equipment : Delta-120	**Total Depth (m)** : 59.70	Coordinate	**X :** 253930.107
Flushing Medium : Water+Polymer	**Ground Elevation** : 5.852	(Nahrwan 1967)	**Y :** 2688696.027
Borehole Dia. (mm) : 96	**Core Dia. (mm):** 60	**Drilling Date Start** : 06.03.10	**End** : 07.03.10
Groundwater Depth (m) : 2.28		**Backfill Date** :	
Casing Dia. (mm) : 114			

Sample/Core Run	Elevation, m (NADD)	Depth (m)	Description	S.P.T. N Value (0–50)	T.C.R. (%)	S.C.R. (%)	R.Q.D. (%)	F.I.
	5.852 – 3.852	0 – 2	(FLL) Medium dense,light grey/brownish grey, carbonate, silty fine to medium SAND with numerous cemented sand pieces.(Recent Fill)	17, 20				
	3.852 – 1.852	2 – 4	(FLL) Dense, light grey/brownish grey, carbonate, very silty fine to medium SAND with numerous cemented sand pieces.(Recent Fill)	40, 41				
		4 – 5	(SSI) Medium dense, brownish grey, carbonate, gypsiferous sandy SILT with occasional crystalline gypsum pieces. (Sabkha Silt)	17				
		5 – 6	(SAS) Very dense, brownish grey, carbonate, very silty fine SAND with numerous crystalline gpsum pieces. (Sabkha Sand)	50				
		6 – 6.4	(SAG).Very dense, cream, silty GRAVEL with numerous weathered crystalline gypsum pieces.(Sabkha Gravel)	50				
1.60		6.4 – 7.2	(GYS).Weak to moderately weak, thinly to medium bedded crystalline GYPSUM. Slightly weathered, closely spaced sub horizontal, rough fractures.(Sabkha Gypsum)		94	69	60	4
		7.2 – 9	(MDS).Very weak to weak, light greenish grey, very thinly bedded calcareous MUDSTONE with inclusions of gupsum(~15%). Slightly and moderately weathered, very closely and medium spaced sub horizontal, rough fractures.(Marine Mudstone)					N.I
1.10					91	60	58	3 / N.I
		9 – 9.4	(GYS).Weak to moderately weak, crystalline GYPSUM. (Sabkha Gypsum)					
		9.4 – 10	(MDS).Very weak to weak, light greenish grey/dark grey, very thinly bedded calcareous MUDSTONE. Slightly weathered, closely and medium spaced sub horizontal, rough fractures.(Marine Mudstone)					

Remarks	Date	Time	Drilled Depth (m)	Water Level (m)	
1. Pressuremeter Test conducted.	06.03.10	14	22	2.10	Logged by :- U.K
2.V.S.P Standpipe Installed.	07.03.10	18	37.70	2.20	Approved by :- R.V
	08.03.10	12.30		2.30	Standards and codes used BS 5930:1999
	09.03.10			2.28	Sheet 1 of 6

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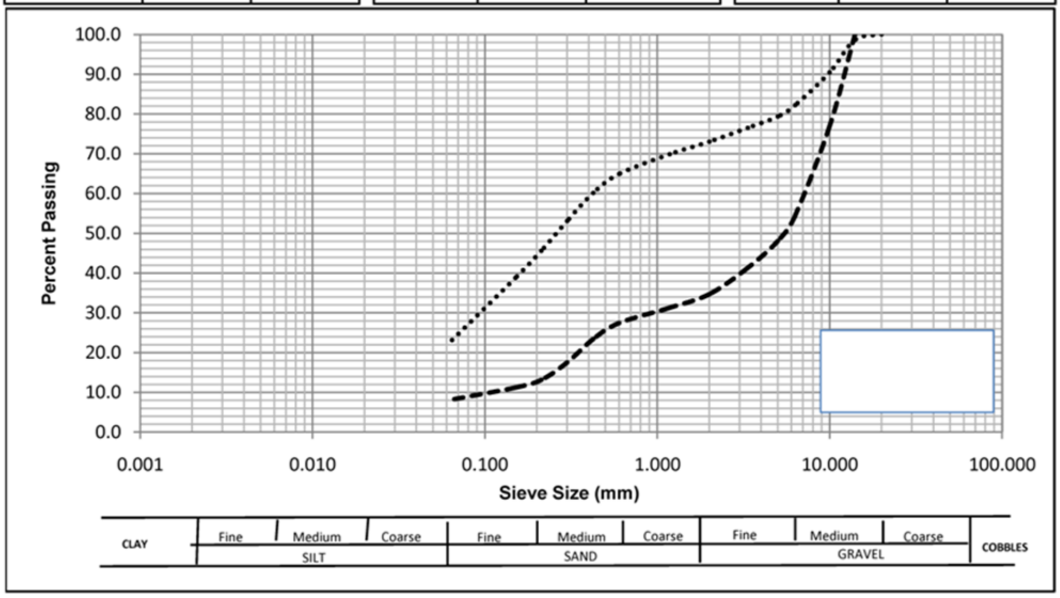
GEEOTECHNICAL INVESTIGATION REPORT (CONTINUED)

Grain Size Analysis (Sieve Test)

Project Name:		Proj. Ref. No.:	
Test Date:		Lab. Room Temp.:	23°C

Borehole No.:		Borehole No.:		Borehole No.:	-
Depth (m):		Depth (m):		Depth (m):	-
Sample Ref. No.:	2	Sample Ref. No.:	3	Sample Ref. No.:	-
Sample Type:	Small Disturbed	Sample Type:	Small Disturbed	Sample Type:	-
Sample Description:	Very Silty Sand	Sample Description:	Silty Gravel	Sample Description:	-
Dry Sample Weight (g):		Dry Sample Weight (g):		Dry Sample Weight (g):	-

Sieve Size (mm)	Retained Weight (g)	% Passing	Sieve Size (mm)	Retained Weight (g)	% Passing	Sieve Size (mm)	Retained Weight (g)	% Passing
50.00	0.00	100.0	50.00	0.00	100.0	-	-	-
37.50	0.00	100.0	37.50	0.00	100.0	-	-	-
28.00	0.00	100.0	28.00	0.00	100.0	-	-	-
20.00	0.00	100.0	20.00	0.00	100.0	-	-	-
14.00	2.70	98.6	14.00	0.00	100.0	-	-	-
10.00	18.30	90.4	10.00	17.40	76.9	-	-	-
6.30	33.90	82.2	6.30	34.30	54.4	-	-	-
5.00	39.40	79.3	5.00	39.10	48.1	-	-	-
3.35	44.70	76.5	3.35	44.20	41.3	-	-	-
2.00	51.40	73.0	2.00	49.20	34.7	-	-	-
1.18	57.30	69.9	1.18	51.70	31.3	-	-	-
0.60	66.70	64.9	0.60	54.70	27.4	-	-	-
0.43	75.80	60.1	0.43	57.70	23.4	-	-	-
0.30	89.10	53.1	0.30	62.10	17.5	-	-	-
0.21	103.40	45.6	0.21	65.40	13.1	-	-	-
0.15	116.30	38.8	0.15	66.90	11.2	-	-	-
0.06	146.90	22.7	0.06	69.20	8.1	-	-	-



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GEEOTECHNICAL INVESTIGATION REPORT (CONTINUED)

Hydraulic Consolidation Test

Project Name:	[illegible]	Lab. Room Temp.:	23°C
Proj. Ref. No:	[illegible]	Test Date:	[illegible]
Orientation:	Parallel To The Vertical.		

Borehole No.:	[illegible]	Sample No.:	[illegible]
Depth (m):	[illegible]	Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND

Diameter-D (mm)	71.30
Area-A (mm²)	3992.72
Height-Hs (mm)	15.50
Dry Wt.+Ring (g)	249.64
Ring Wt. (g)	99.40
Particle Density (Gs)	2.43
Degree of Saturation **S**o (%)	146.55

	Initial	Final
Wet Wt.+Ring (g)	287.69	279.62
Height-H (mm)	22.00	20.59
Volume-V (cm^3)	87.84	82.21
Wet Density (g/cm^3)	2.14	2.19
Moisture Content (%)	25.33	19.95
Void ratio (e)	0.42	0.33
Dry Density (g/cm^3)	1.71	1.83

Increment From/To (kPa)	0-50	50-100	100-200	200-400	400-800	800-1400	1400-700	700-0
Pressure (kPa)	50	100	200	400	800	1400	700	1
Deformation of apparatus (mm)	0	0	0	0	0	0	0	0
Consolidated height, Ĥ (mm)	21.77	21.59	21.40	21.16	20.89	20.60	20.686	20.876
Voids ratio (e)	0.41	0.39	0.38	0.37	0.35	0.33	0.34	0.35
Height change-H (mm)	0.23	0.18	0.19	0.24	0.27	0.29		
Pressure change-p (kPa)	50	50	100	200	400	600		
Volume compress-m_v (m^2/MN)	0.21	0.17	0.09	0.06	0.03	0.02		
t_{100} (min)	4.00	7.84	8.41	15.21	16.00	30.25		
Average specimen height (mm)	21.89	21.68	21.49	21.28	21.02	20.74		
Coeff. of consol-C_v (m^2/year)	1.48	0.74	0.68	0.37	0.34	0.18		

Increment	Pressure (kPa)	Test Date
0-50	50	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.56	0.0	0.00
0.1	8.56	0.3	0.00
0.2	8.55	0.4	-0.01
0.5	8.48	0.7	-0.08
1	8.43	1.0	-0.13
2	8.40	1.4	-0.16
4	8.36	2.0	-0.20
8	8.34	2.8	-0.22
15	8.33	3.9	-0.23

Height before loading (mm)	22
Height after loading (mm)	21.77

Increment	Pressure (kPa)	Test Date
50-100	100	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.33	0.0	0.00
0.1	8.31	0.3	-0.02
0.2	8.28	0.4	-0.05
0.5	8.24	0.7	-0.09
1	8.21	1.0	-0.12
2	8.19	1.4	-0.14
4	8.18	2.0	-0.15
8	8.17	2.8	-0.16
15	8.16	3.9	-0.17
30	8.15	5.5	-0.18

Height before loading (mm)	21.77
Height after loading (mm)	21.59

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GEEOTECHNICAL INVESTIGATION REPORT (CONTINUED)

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Sample No.:	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND

Increment	Pressure (kPa)	Test Date
100-200	200	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.15	0.0	0.00
0.1	8.13	0.3	-0.02
0.2	8.10	0.4	-0.05
0.5	8.04	0.7	-0.11
1	8.01	1.0	-0.14
2	8.00	1.4	-0.15
4	7.99	2.0	-0.16
9	7.98	3.0	-0.17
15	7.97	3.9	-0.18
30	7.96	5.5	-0.19

Height before loading (mm)	21.59
Height after loading (mm)	21.40

Increment	Pressure (kPa)	Test Date
200-400	400	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.96	0.0	0.00
0.1	7.92	0.3	-0.04
0.2	7.88	0.4	-0.08
0.5	7.82	0.7	-0.14
1	7.79	1.0	-0.17
2	7.78	1.4	-0.18
4	7.76	2.0	-0.20
8	7.75	2.8	-0.21
15	7.74	3.9	-0.22
30	7.73	5.5	-0.23
60	7.72	7.7	-0.24

Height before loading (mm)	21.40
Height after loading (mm)	21.16

Increment	Pressure (kPa)	Test Date
400-800	800	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.72	0.0	0.00
0.1	7.69	0.3	-0.03
0.2	7.65	0.4	-0.07
0.5	7.56	0.7	-0.16
1	7.53	1.0	-0.19
2	7.51	1.4	-0.21
4	7.50	2.0	-0.22
8	7.49	2.8	-0.23
20	7.47	4.5	-0.25
30	7.46	5.5	-0.26
60	7.45	7.7	-0.27

Height before loading (mm)	21.16
Height after loading (mm)	20.89

Increment	Pressure (kPa)	Test Date
800-1400	1400	20/08/2011

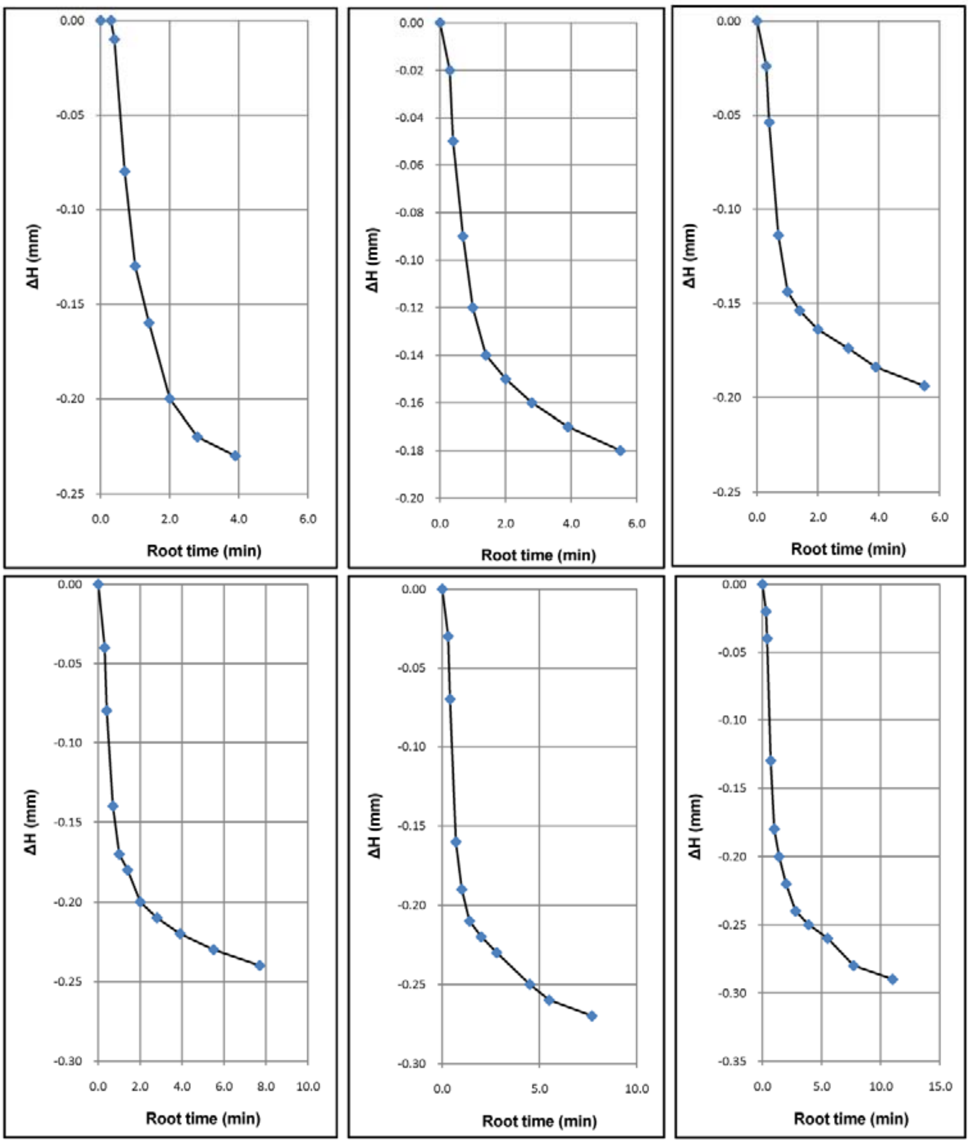
Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.45	0.0	0.00
0.1	7.43	0.3	-0.02
0.2	7.41	0.4	-0.04
0.5	7.32	0.7	-0.13
1	7.27	1.0	-0.18
2	7.25	1.4	-0.20
4	7.23	2.0	-0.22
8	7.21	2.8	-0.24
15	7.20	3.9	-0.25
30	7.19	5.5	-0.26
60	7.17	7.7	-0.28
120	7.16	11.0	-0.29

Height before loading (mm)	20.89
Height after loading (mm)	20.60

Hydraulic Consolidation Test

Project Name:	[illegible]	Lab. Room Temp.:	23°C
Proj. Ref. No:	[illegible]	Test Date:	[illegible]
Orientation:	Parallel To The Vertical.		

Borehole No.:	[illegible]	Sample No.:	[illegible]
Depth (m):	[illegible]	Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND

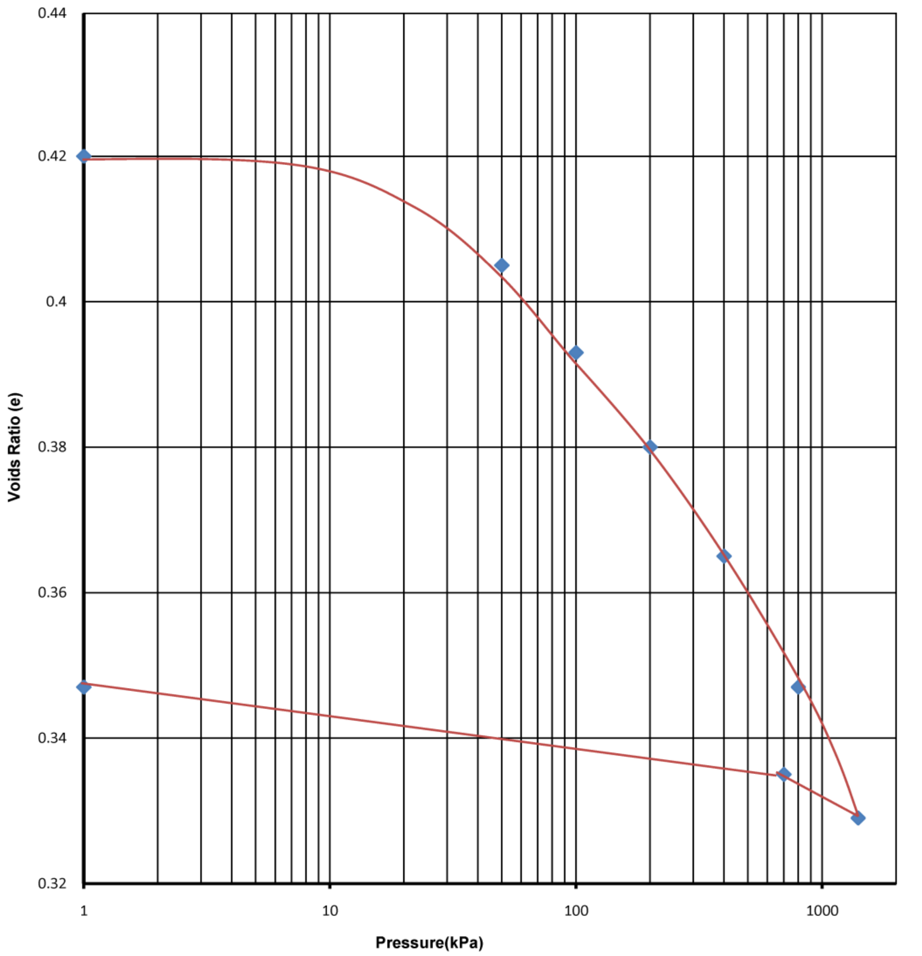


GEEOTECHNICAL INVESTIGATION REPORT (CONTINUED)

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Sample No.:	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND



Preconsolidation Pressure (σ_c') = 85 (kPa)

Recompression Index (C_r) = 0.0022

Compression Index (C_c) = 0.069

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GEEOTECHNICAL INVESTIGATION REPORT (CONTINUED)

Variable Head Permeability Test

Project Name:	Proj. Ref. No:	Location:	
Borehole ID:	Material Description: Soil	Depth of test section (m)	11.9
Test No: 1	Test Date:	Time of Start:	4.00 PM
Height of Casing above EGL (m) 0.50	F(d)*: 0.472	GW Depth (m):	3.18

Borehole /Casing Dia (m)	0.114	Section Area (m²)	0.01021	Depth of Casing below EGL (m)	11.4	Height of Uncased Section (m)	0.50

Head Reading (m)	Time of Reading (min)	Ht (m)	(Ht/H0)
0.00	0.00	3.68	1.000
0.07	0.25	3.61	0.981
0.13	0.50	3.55	0.965
0.18	0.75	3.50	0.951
0.21	1.00	3.47	0.943
0.26	1.50	3.42	0.929
0.34	2.00	3.34	0.908
0.40	3.00	3.28	0.891
0.48	4.00	3.20	0.870
0.57	5.00	3.11	0.845
0.67	6.00	3.01	0.818
0.74	7.00	2.94	0.799
0.81	8.00	2.87	0.780
0.86	9.00	2.82	0.766
0.94	10.00	2.74	0.745
1.10	15.00	2.58	0.701
1.18	20.00	2.50	0.679
1.26	25.00	2.42	0.658
1.34	30.00	2.34	0.636
1.51	40.00	2.17	0.590
1.67	50.00	2.01	0.546
1.83	60.00	1.85	0.503
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-
-	-	-	-

k (cm/s)= 9.34198E-04

k(m/s)= (A /F(t2-t1)) ln(H1/H2)

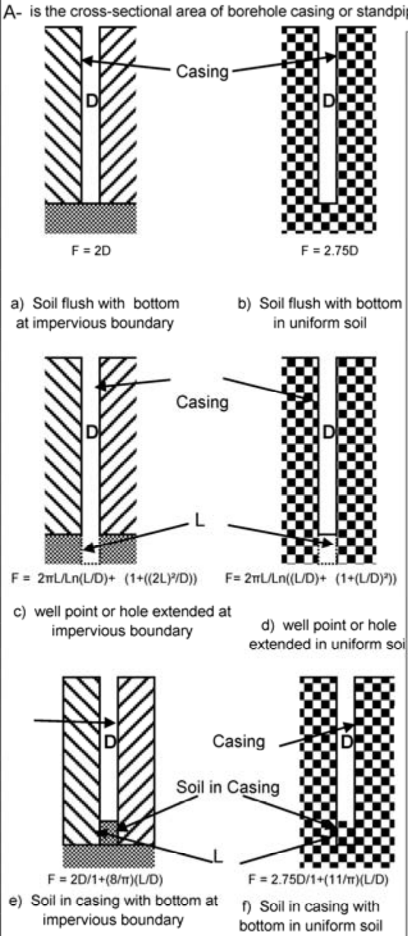
k- is the Permeability of Soil

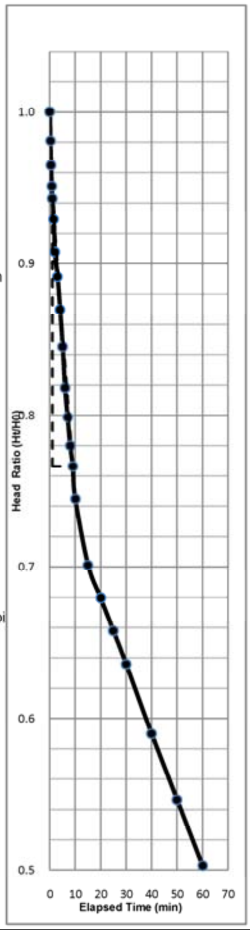
F- is the intake factor (see below)

H- is the variable head measured at time t1 after commencement of test

H- is the variable head measured at time t2 after commencement of test

A- is the cross-sectional area of borehole casing or standpipe as appropriate





* Method 'd' has been used on site.

SG-F-IMS-RRF-042, R-00, D-01/05113

APPENDIX E

EXAMPLE APPLICATION SUBMITTAL FOR DEWATERING OF DEEP, COMPLEX EXCAVATIONS

PROJECT DEWATERING REPORT

DEEP DEWATERING PROGRAM FOR CONSTRUCTION OF (*STRUCTURE NAME*)

ADDRESS OF PROJECT
ZONE NO./NAME
SECTOR NO.
PLOT NO.

PROJECT NO. XX
REVISION 1
XX JUNE 2014

CONTRACTOR NAME
CONTRACTOR ADDRESS
CONTRACTOR PHONE CONTACTS
WWW.CONTRACTOR.COM

TABLE OF CONTENTS

PROJECT DEWATERING REPORT

DEEP DEWATERING FOR CONSTRUCTION OF (*STRUCTURE NAME*)

1.0 INTRODUCTION

1.1 PROJECT UNDERSTANDING

The Project consists of dewatering for the proposed infrastructure located at *(location of structure)*. The scope involves the dewatering of the site utilizing a progressive sump and trench dewatering system to enable construction to be carried out in dry, safe conditions.

The dewatering system installation will be dependent on the site geology, hydrogeological conditions, access, and working space available. The dewatering system needs to be installed and operational before the excavation begins. After the Method Statement (***Section 4.0***) has been approved by the Abu Dhabi City Municipality (ADM), the Main Contractor should be given approval to begin mobilizing.

1.2 PROJECT ENTITIES: OWNER, CONTRACTOR, AND DEWATERING SUBCONTRACTOR

It is the intention of the Dewatering Subcontractor to coordinate closely with the Main Contractor in terms of mobilization, installation, commissioning, operation, and maintenance of the dewatering system. Relevant project entities are as follows:

Owner:	(Name and Address of Owner)
Main Contractor:	(Main Contractor name)
Dewatering Subcontractor:	(Dewatering Subcontractor name)

1.3 HEALTH AND SAFETY PROGRAM

Personnel Health and Safety will be protected under the Main Contractor's and Dewatering Subcontractor's Health and Safety Plans. Potential risks to personnel safety include excavation collapse, falls into the excavation, risks due to working in confined spaces (air quality, entrapment), and risk of electric shock due to wiring and power supply to the sump pumps. These risks and control measures are discussed in the Risk Analysis provided in ***Appendix E3***.

1.4 ENVIRONMENTAL PROTECTION PROGRAM

Owner's environmental guidelines and standards have been considered throughout the design of the proposed system. Environmental protection will be provided in accordance with local regulations and Dewatering Subcontractor's standard environmental protection plans. Owner shall be responsible for all permits associated with dewatering.

To limit emissions at the site, electric motors shall be used as prime movers where possible, allowing for the concentration of diesel prime movers at dedicated control stations.

Diesels and oils shall be restricted within drip trays to prevent contamination of the sand. Bulk storage fuel tanks shall be bundled to contain waste or spillage.

All dewatering system effluent shall pass through a settlement tank before discharge into the approved collection area, or approved stormwater discharge manhole.

An outline of environmental hazards is included in the Risk Analysis provided in ***Appendix E3***.

1.5 QA/QC MANAGEMENT PROGRAM

Quality Assurance/Quality Control (QA/QC) Management will be provided under Dewatering Subcontractor's internal processes and will be the responsibility of the Dewatering Subcontractor Site Manager.

2.0 SITE SUBSURFACE CONDITIONS

2.1 DESCRIPTION OF SITE GEOLOGIC SETTING AND HYDROGEOLOGY[1]

Generally, Abu Dhabi Emirate is divided into four physiographic regions, each with unique topographic, geologic, geomorphic, and hydrologic characteristics. Namely, these regions include the Al Hajar Mountains, the wadis and alluvial fans along the flanks of the Al Hajar, the vast desert region of sand dunes and inland sabkhas (the largest area within the Abu Dhabi Emirate), and the coastal region that includes sand dunes, coastal sabkhas, lagoons, and tidal zones. Overall, this coastal physiographic region is characterized by the lowest elevations (typically less than 20 meters mean sea level [m msl]) and flattest surfaces in the Emirate, and contains a higher proportion of sabkha (coastal sabkha). The ADM lies primarily within the coastal region but also the desert region in its southern and eastern extents.

Erosion and fluvial transport of sand, gravel, and boulders down the sides of the Al Hajar and in mountain valleys (wadis) have resulted in the formation of extensive outwash sand, gravel deposits, and alluvial fans that empty out onto the desert floor (Kumar et al., 2008). Within the eastern ADM, these alluvial and wadi land surfaces are covered with a vast expanse of desert filled with sand dunes and inland sabkhas (Styles et al., 2006). Along the Arabian Gulf coastline, which includes most of the ADM, the land surface contains sand dunes, beach sand deposits, lagoonal silts and clays, and coastal sabkha deposits (Farrant et al., 2012a).

In the ADM, Precambrian (Proterozoic) basement rocks lay approximately 9 kilometer (km) below ground level (BGL) (Farrant et al., 2012a). Overlying the crystalline basement in the United Arab Emirates (UAE) is a thick sequence of shallow, nearly flat-lying marine sedimentary rocks that include up to 2,500 meters (m) of Cambrian to Carboniferous aged primarily clastic rock, over which was deposited approximately 4,300 m of mainly limestone, dolomite, mudstone, and anhydrite of mid-Permian to Cretaceous age (Styles et al., 2006). Following a period of erosion and non-deposition that created a regional unconformity at the end of the Mesozoic Era, significant amounts of shale, mudstone, argillaceous limestone, and

[1] This Section should be project specific. As no location data is presented for the example, only a general description of geologic setting is provided here.

anhydrite layers were deposited from the Paleogene to mid-Miocene (Farrant et al., 2012a). The Gachsaran Formation is the uppermost of these younger rock strata in the ADM.

The overlying formations represent the shallow groundwater flow system within ADM, and are the most important relative to fresh groundwater supplies in the Emirate and to the geotechnical and engineering problems associated with construction and dewatering in the ADM. Overlying the Gachsaran in the northern and eastern parts of the ADM are the distal ends of wadi and alluvial fan deposits of the Barzaman Formation and the Hili Formation (Farrant et al., 2012b). Barzaman rocks grade laterally to the south and to the west into the Baynunah Formation. The Hili deposits are younger and cut down through the older Barzaman or Baynunah deposits (Farrant et al., 2012b). Together these units make up the uppermost bedrock surface within the ADM.

The late Pleistocene (Quaternary) age, weakly- to well-cemented, aeolian sandstone deposits of the Ghayathi Formation overlies the above-mentioned formations (Farrant et al., 2012b). The Ghayathi is in turn overlain by younger, late Pleistocene to Holocene age unconsolidated dune deposits of the Rub al Khali Formation that includes thick deposits of fine- to medium-grained sands that are carbonate-rich near the coast and quartz-rich inland (Farrant et al., 2012b). Within many inland areas, the younger Rub al Khali sands have been deflated down to the water table, where these flat, low-lying areas frequently becomes cemented with halite and gypsum, forming sabkhas. Along the coast, and extending a few kilometers inland in the ADM, the coastal zone sand dunes, beach sands, sabkha, lagoonal muds, intertidal algal mats, and other near-shore marine deposits of late Pleistocene to Holocene age are collectively referred to as the Abu Dhabi Formation (Farrant et al., 2012b).

It is important to note, however, that much of the ADM includes land areas that have been reclaimed by sabkha infilling, or that have been formed by the artificial expansion of existing islands using fill material. Fill material utilized in the development of this “made ground” is typically dredged sediment from near-shore shipping channels, near-shore sand dunes, or fill material transported from greater distances.

The top of bedrock elevations, within the ADM, range from roughly -20 to 100 m msl. Generally, the lowest top of bedrock surface elevations (-20 to 10 m msl) are located in the western ADM, but rise inland, effectively mimicking surface elevation changes. In the core geotechnical hazard area, bedrock elevations range from approximately -15 to 55 m msl, with top of rock elevations being highest (10 to almost 55 m msl) to the east.

Total thickness of the unconsolidated sediments and fill material (made ground) within the ADM region (i.e., Quaternary aquifer) ranges from 0 to about 24 m, although in most areas, the overburden thickness ranges only from 0 to 10 m. In the core geotechnical hazard zone, the total thickness of unconsolidated sediments is typically 15 m or less.

Interpreted potentiometric surfaces from waterstrike data indicates that groundwater elevations in the western and north-central ADM are relatively flat, ranging from approximately -15 up to about 10 m msl. Groundwater elevations generally increase in an easterly direction, mimicking ground surface elevations with the relatively highest groundwater levels (roughly 70 to 102 m msl) observed in the southeastern ADM. In the westernmost core ADM area, groundwater elevations are relatively flat, ranging from approximately -15 to 5 m msl while in the eastern core area, groundwater elevations appear much higher, between about 5 and 45 m msl.

2.2 DESCRIPTION OF GEOTECHNICAL CONDITIONS

Geotechnical investigations and testing of the subsurface materials and ground water were performed in accordance to the recommendations given in the Field Investigation Scope (Table 11-3) of the ADM Guidelines and the associated key (shown below). The investigation includes one boring every 50 m along the length of the trench. All 20 borings extend to a depth of at least 30 m and samples are taken in every geologic layer. This meets the requirements per the ADM guidelines.

FIELD INVESTIGATION SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis

FIELD INVESTIGATION SCOPE (CONTINUED)

KEY		
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspection of boring is required
		Visual inspection of boring is not required

EXCERPT FROM TABLE 11-3 OF THE ADM GUIDELINES

Hazard Zone	**Proximity of Structures**	**Excavation/ Dewatering Type**	**Excavation Depth**	**Borehole Distribution (< 10000 m^2)**	**Visual Inspections**	**Lab Testing[3]**	**Field Testing**	**Third Party Review**
C	**1,2**	i	Shallow	5 [1,2]	X	1 [4]		
		i, ii, iii	Medium	5 [1,2]		1 [4]	Slug Test	
			Deep	**5 [1,2]**	**-**	**1 [4]**	**Packer Test[5]**	**X**
	3	i	Shallow	5 [1,2]	X	1 [4]		
		ii, iii	Medium	5 [1,2]	X	1 [4]	Slug Test	
		i, ii, iii	Deep	5 [1,2]		1 [4]	Packer Test[5]	X

Notes:

1 Two thirds of the boreholes should be up to 1.5 x depth of excavation and the remaining boreholes up to 2 x depth of excavation.

2 One borehole each at the corners and one at approximate center location or at a spacing not exceeding 50 m c/c. For soil and ground water testing refer to ***Section 11***.

3 Sieve analysis and Atterberg Limits (Soil Classification, e.g., USCS).

4 One test per geologic layer (based on geologist's description) but no less than one test per 3 m of depth.

5 Packer tests are performed in rock formations only. If not applicable, a slug test is recommended.

The site subsurface is characterized by up to approximately 6 m of medium dense to very dense, fine to medium-grained silty sands (SM). Laboratory tests confirm the United Soil Classification System (USCS) classification of SM. The silty sand is underlain by siltstone, sandstone, and gypsum. Gypsum bands range from 0.3 m-6 m thick, averaging around 2-3 m. The main gypsum layer ranges from 11 to 14 m depth (***Appendix E4***).

The groundwater table is generally 2 m below the existing ground level. Hydraulic permeability of rock mudstone is determined through packer tests. Hydraulic permeability of soil is determined according to USCS classification and sieve analysis.

Details on the Geotechnical Investigation are provided in ***Appendix E4***[2].

[2] For the sake of brevity, only select pages are provided in this example. The full Geotechnical Investigation Report is required in an actual application, to be provided as an Appendix to the application.

3.0 DEWATERING SYSTEM DESIGN

3.1 SUMMARY OF DESIGN CALCULATIONS

The Site is in Hazard Zone C near a sensitive building. As construction requires excavation to a depth of 15 m, the excavation is considered deep. According to the Dewatering Design Scope (Table 10-3) in the ADM Guidelines, a numerical model is required for pumping capacity analysis and a hand calculation is required for settlement analysis (Refer to the key in ***Section 2.2*** and excerpt from Table 10-3 below). These calculations are provided in ***Appendix E2***.

DEWATERING DESIGN SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspection of boring is required
		Visual inspection of boring is not required

DEWATERING DESIGN SCOPE (CONTINUED)

EXCERPT FROM TABLE 10-3 OF THE ADM GUIDELINES						
HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	PUMPING CAPACITY ANALYSIS	SETTLEMENT ANALYSIS	THIRD PARTY REVIEW
C	**1,2**	i	Shallow	a	I	
		i, ii, iii	Medium	a	I	
			Deep	**c**	**I**	**X**
	3	i	Shallow	a	I	
		ii, iii	Medium	a	I	
		i, ii, iii	Deep	b	I	X

3.2 EVALUATION OF POTENTIAL SITE IMPACTS DUE TO DEWATERING

The Site is expected to have relatively low risk of detrimental effects from a properly designed and operated dewatering system. Because the site is in an urban area, settlement due to soil collapse has the potential to cause large impacts in the project area if it occurs. The following sections discuss several typical site impacts caused by a trench and sump dewatering system.

3.2.1 Settlement and Soil Collapse

Settlement due to soil collapse is possible when dewatering operations cause the migration of fine materials from the surrounding soil through the dewatering system, or where the flow of water through the surrounding soil encounters soluble minerals such as sabkha or gypsum. The primary cause of settlement, however, is the increase in effective stress due to dewatering. Differential settlement is of special concern since the increase in effective stress is non-uniform laterally.

Risk due to migration of fine materials will be mitigated by the presence of filter zones and by the supervision of the Site Manager, who will inspect flows for signs of turbidity and take appropriate measures such as reducing the flow rates or abandoning the well if excessive turbidity occurs. The gravel backfill around the deep sumps will act as a filter and will prevent collapse of the sump during construction.

Gypsum and salt interbeds were encountered in the geotechnical exploration at depths as shallow as 10 m below ground surface.

Instrumentation for measuring settlement, deformation, and slope movement is provided near the sensitive structure, as required by the ADM Guidelines for this type of project. Piezometers for measuring water level will be installed every 50 m, as required. Applicable information from the Dewatering Monitoring Scope (Table 12-2, shown below) of the ADM Guidelines is provided.

DEWATERING MONITORING SCOPE

KEY		
Geologic/Hydrogeologic Hazard Zone	A	High potential
	B	Medium Potential
	C	Low Potential
Proximity of Structures	1	Sensitive or large structures nearby
	2	Structures could be impacted by project
	3	No structures that could be impacted
Excavation Depth	Shallow	0-3 m
	Medium	3 m-10 m
	Deep	>10 m
Excavation/Dewatering Type	i	Open Cut (Sumps and Open Pumping)
	ii	Cutoff Structure
	iii	Wells and Ejectors
Pumping Capacity Analysis	a	Analytical Solution
	b	Flow Net
	c	Numerical Analysis
Settlement Analysis	I	Hand Calculation
	II	Numerical Analysis
Third Party Review	X	Third Party Review Required
		Third Party Review Not Required
Field Testing	Slug Test	Shallow to Medium; Low to medium hazard; low structure sensitivity
	Packer Test	Medium to Deep; Low to medium hazard; low structure sensitivity
	Pumping Test	Deep; Medium to high hazard; sensitive structures close
Flow Measurement	X	Flow measurement required
		Flow measurement not required
Visual Inspections	X	Visual inspection of boring is required
		Visual inspection of boring is not required

EXCERPT FROM TABLE 12-2 OF THE ADM GUIDELINES								
HAZARD ZONE	PROXIMITY OF STRUCTURES	EXCAVATION/ DEWATERING TYPE	EXCAVATION DEPTH	WATER LEVEL / TURBIDITY / TDS	SURVEY/ DEFORMATION	SETTLEMENT	SLOPE MOVEMENT	FLOW MEASUREMENT
C	**1,2**	i	Shallow					
		i, ii, iii	Medium	4 [1]	1 [2]	1 [2]	1 [2]	X
			Deep	**4 [1]**	**1 [2]**	**1 [2]**	**1 [2]**	**X**
	3	i	Shallow					
		ii, iii	Medium					
		i, ii, iii	Deep	4 [1]	1 [2]	1 [2]	1 [2]	X

Notes:

[1] One piezometer each at the corners or at a spacing not exceeding 50 m c/c.

[2] One instrument on or near to each sensitive structures/steep section of the slope.

The settlement calculation is provided in ***Appendix E2***.

3.2.2 Excavation Slope Stability

Dewatering to levels below the planned excavation elevation is required to maintain slope stability and reduce the chance of boils in the sandy materials at the site. Groundwater levels in the excavation area will be observed and monitored throughout the dewatering process. Excavation will not be performed to a level closer than 0.5 m from the groundwater table. Shoring will be installed to support the excavation, as implemented by the main contractor. Open excavation may be allowed if it remains within the property limits, i.e., the excavation shall be contained within the plot limits and not encroach on neighboring property.

Instrumentation for measuring slope movements is provided near the sensitive structure, as required by the ADM Guidelines for this type of project.

3.2.3 Migration of Fine Materials

As discussed in Section 3.2.1, migration of fine materials is a cause for settlement due to soil collapse. In addition, discharge of highly turbid water into the adjacent storm sewer system is prohibited by the site permit. Discharge of highly turbid water will be prevented by the filter zone around the sump as described in ***Section 4.0***, and by daily observation of dewatering discharges. Settlement tanks will be used in conjunction with dewatering pumps.

The filter zone will be designed according to ***Section 4.3*** of the Dewatering Guidelines and properties of the silty sand material surrounding the filter zone. Uniformity Coefficient (C_u) and D_{50} are determined from Sieve Analysis results, such as those presented in ***Appendix E4***. In this case, D_{50} of the silty sand is around 0.11 millimeter (mm) and C_u is approximately 2.5. Since C_u is generally less than 3, the D_{50} of the filter pack should be 4 to 5 times D_{50} of the silty sand, and C_u of the filter pack material must be smaller (more uniform) than C_u of the silty sand.

4.0 CONSTRUCTION METHOD

4.1 PURPOSE, SCOPE, AND DESIGN BASIS

This Method Statement describes the activities and procedures for the dewatering of the proposed structure (*location*), Abu Dhabi. The scope involves the dewatering of the proposed site to enable construction to be carried out in dry, safe conditions through a sump and French drain dewatering system.

The works for the proposed development involves the excavation as per the following information:

- Area for Dewatering: 5 m x 1000 m
- Ground Level: +0.00m
- Existing Water Table Level: approximately 2.0 m Below Ground Level (BGL)
- Max Excavation Level: approximately 10.00 m BGL
- Shoring: Impervious Cutoff Structure

4.2 SUMMARY OF PROPOSED DEWATERING SYSTEM

It is proposed to install a progressive sump pump and French drain system to control the groundwater within the excavation area. By definition, a sump is at a low level in relation to surrounding ground surfaces so that any water will flow to it due to gravity. Sump pumping is the most basic of the dewatering methods. In essence, it involves allowing groundwater to seep into the sumps (pits) and then pumping it away for disposal.

4.2.1 Physical Arrangement of Dewatering Equipment

Sumps shall be installed at approximately 100 m intervals along the alignment of the excavation. The sumps and French drains will be arranged as shown in ***Appendix E1***.

4.2.2 Installation Method and Required Equipment

A sump pit is initially excavated when water is encountered, then subsequently lowered as the excavation deepens. A slotted casing is installed into the sump and surrounded by filter aggregate. A submersible pump is fitted inside the slotted casing.

The French drain system shall be constructed as an integral part of the excavation works. A network of French drains can be installed to adapt to the ground conditions. The flexible nature of the system allows the dewatering to be developed in parallel with the excavation. As the excavation deepens, any water ingress encountered at higher levels can be collected in the original trenches and channeled to the lower level using vertical drains in the excavation embankments. Therefore, it is expected that pumping shall only be maintained at the lowest level of excavation at any given time.

Piezometers shall be installed at every 50 m just outside the excavation by use of a self-propelled drilling rig with a 150-millimeter (mm) auger. After completion of drilling, a jetting lance will be inserted and high-pressure water shall be injected into the ground through the jetting lance, widening and cleaning the borehole. This development of wells is important to maximize the effectiveness of the system. A piezometer will be installed into both the sand and the bedrock per the ADM Guidelines.

4.2.3 Operations, Monitoring, and Maintenance during Construction

Instrumentation for measuring the groundwater level, settlement/subsidence, and slope movements will be provided on one or both sides of the trench, dependent on location of sensitive structures. The instruments shall be regularly monitored to ensure safety of excavation and the dewatering system. Instrumentation for monitoring includes:

- One piezometer at a spacing not exceeding 50 m on-center.
- One instrument measuring settlement, slope movement, and deformation at or near each sensitive building or steep section of slope.

In addition to instrument monitoring, visual inspection will be performed by the Dewatering Subcontractor Site Manager during the operations and maintenance of the dewatering system and will consist of the following daily checks:

- Dewatering pump operation and condition, including performance of any required maintenance or repairs.
- Monitoring of dewatering system effluent to identify excessive turbidity in flows. If excessive turbidity is noted, the Dewatering Subcontractor Site Manager will adjust the dewatering system to limit the migration of fine materials by throttling or shutting off specific sump pumps.
- Visual inspection of the surrounding area and excavation slopes for signs of excessive settlement, slope instability, or excessive seepage into the excavation.

No overhead obstructions are expected. Underground structures have been identified and excavation will proceed with caution in the vicinity of expected underground structures.

5.0 REFERENCES

Farrant, A.R., R.A. Ellison, J.W. Merritt, J.E. Merritt, A.J. Newell, J.R. Lee, S.J. Price, R.J. Thomas, and A. Leslie, 2012a, “Geology of the Abu Dhabi 1:100,000 Map Sheet, 100-16, United Arab Emirates,” Ministry of Energy, United Arab Emirates, 69 p.

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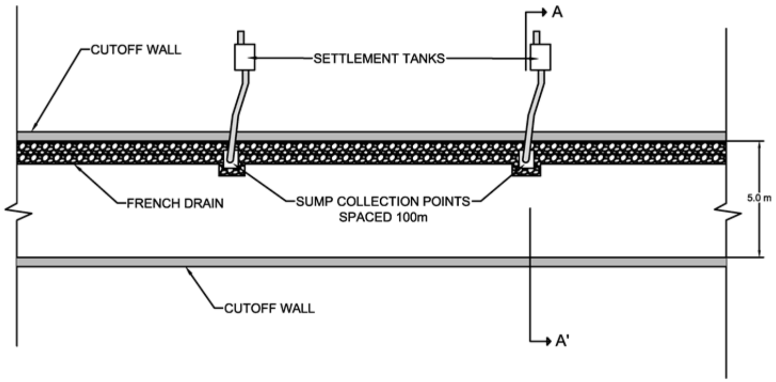
APPENDICES

APPENDIX E1

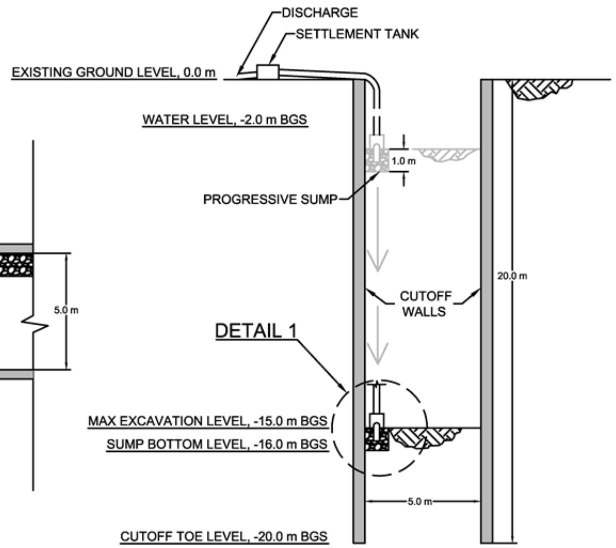
DRAWING

بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

DRAWN BY	ASH	CHECKED BY	EOT		CAD FILE NUMBER	164392–121
	BMW	APPROVED BY	MJE			



PLAN VIEW
TRENCH AND WELL POINT LAYOUT



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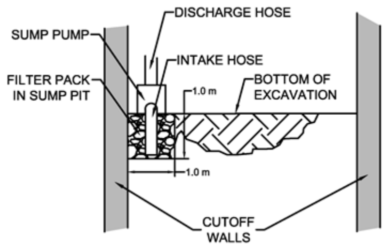
SECTION THROUGH TRENCH



1 DETAIL 1
PROGRESSIVE SUMP

FIGURE 1–1

DEWATERING
EXCAVATION
PLAN AND SECTION
PREPARED FOR

CLIENT
LOCATION
CONTRACTOR, Inc.
ENGINEERS / CONSULTANTS

APPENDIX E2

CALCULATIONS

APPENDIX E2

CALCULATIONS

According to the ADM Dewatering Guidelines for a deep excavation in Hazard Zone C that is near sensitive or large structures, a numerical model is required for pumping capacity design and a hand calculation is required for settlement analysis.

ASSUMPTIONS

1. The depth to the impermeable layer is unknown. The total head in the water table aquifer is assumed equal to 33 m.

2. Analytical solutions assume homogenous, isotropic material whereas the subsurface in Abu Dhabi is layered. The numerical model is therefore expected to more accurately represent actual conditions.

3. These calculations assume lateral homogeneity. This is not necessarily the case in reality. Existence of cavities or more permeable areas (such as un-engineered fill) could affect both flow rate and settlement.

4. This example does not involve wells or wellpoints for dewatering. Water flows into the excavation area due to a head difference and pumping occurs at the sumps, which are essentially at atmospheric pressure. Therefore, equations of flow into a trench are used by directly applying Darcy's law, rather than equations for radial flow, which apply to wells.

5. Dewatering at the sumps occurs as water flows into French drains in the trench due to gravity, no suction is assumed.

6. Elastic settlement in mudstone is negligible and not included in this calculation.

7. One-dimensional consolidation settlement is assumed as an approximation of settlement from dewatering.

CALCULATION INPUT

Refer to ***Appendix E1*** for site geometry. Refer to ***Section 2.2*** and ***Appendix E4*** for subsurface conditions. ***Table 1*** summarizes parameters used in calculation of flowrate/pumping capacity and settlement.

TABLE 1
PARAMETERS USED IN ANALYSIS

PARAMETER	VALUE	JUSTIFICATION
Width of Excavation	5 m	From project drawings
Length of Dewatering	1000 m	100 m distance between sumps
Static Water Depth (m BGL) before dewatering	2 m	From Soils Report
Excavation Depth (m BGL)	15 m	Max, from project drawings
Drawdown Depth (m BGL)	16 m	Required at least 0.5 m below excavation by ADM Guidelines
Required drawdown	14 m	Required at least 0.5 m below excavation by ADM Guidelines
Permeability, Sand	1E-04 m/s	From the Typical Values of Permeability of Saturated Soils **(**Table 10-1) of the ADM Guidelines at the conservative end of the fine sand range
Permeability, Mudstone	1E-06 m/s	From packer test in ***Appendix E4***, verified by ***Reference 2***
Void Ratio, e, Sand	0.42	From consolidation test in ***Appendix E4***
Saturated Unit Weight, Sand	21.0 kN/m^3	From consolidation test in ***Appendix E4***
Dry Unit Weight, Sand	16.8 kN/m^3	From consolidation test in ***Appendix E4***
Compression Index, Sand	0.069	From consolidation test in ***Appendix E4***

ANALYTICAL SOLUTION

Settlement from dewatering occurs from the consolidation of compressive sands with fines due to an increase in effective stress (decrease in pore pressures), even if dewatering is carried out properly. Dewatering removes buoyancy from the soil, increasing the effective stress as total stress remains constant. Estimated settlement is calculated by finding effective stresses at the static and drawn down water levels.

Total stress and effective stress are defined under initial conditions, before dewatering, as in ***Equations 1 and 2***. Final effective stress, after dewatering, is calculated in ***Equation 3***. A graphical representation of initial and final effective stress is presented on ***Figure 1***. Stresses are estimated outside the cutoff structure. The water levels outside the cutoff structure will not draw down to the same level as the water level inside, but the water level outside the cutoff walls will still be affected by dewatering efforts. As shown on ***Figure 2***, the water level outside the cutoff structure is expected to drop below the top sand layer and into the mudstone, so settlement of the sand layer is estimated.

$$\sigma_0 = h_{WL} * \gamma_d + (h_r - h_{WL}) * \gamma_s \qquad \text{(Equation 1)}$$

$$\sigma'_0 = \sigma_o - (h_r - h_{WL}) * \gamma_w \qquad \text{(Equation 2)}$$

$$\sigma'_f = h_d * \gamma_d + (h_r - h_d) * (\gamma_s - \gamma_w) \qquad \text{(Equation 3)}$$

where,

γ_d = dry unit weight = 16.8 kN/m^3 from ***Appendix E4***

γ_s = saturated unit weight = 21.0 kN/m^3 from ***Appendix E4***

γ_w = unit weight of water = 9.8 kN/m^3

h_{WL} = depth of original water level below ground surface = 2 m

h_d = depth of water level BGL after dewatering = 6 m (set equal to h_r if greater than h_r)

h_r = thickness of soil to calculate settlement = 6 m (depth of soil to rock from ***Appendix E4***)

σ_0 = total stress under initial conditions = 117.5 kN/m^2

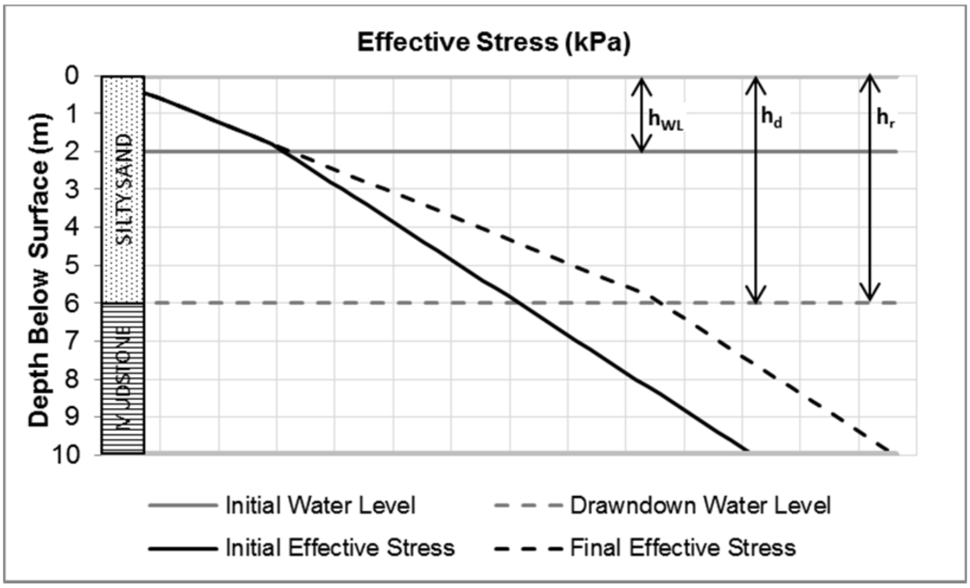
σ'_0 = effective stress under initial conditions, at the bottom of the sand layer = 78.3 kN/m^2

σ'_f = final effectives stress, at the bottom of the sand layer = 100.7 kN/m^2

σ'_{0m} = effective stress under initial conditions, mid depth of the sand layer (h_r=3m) = 44.7 kN/m^2

σ'_{fm} = final effectives stress, mid depth of the sand layer (h_r=3m) = 50.3 kN/m^2

FIGURE 1
GRAPHICAL SCHEMATIC OF INITIAL AND FINAL EFFECTIVE STRESS



Estimated settlement is calculated in ***Equation 4*** from the compression index, void ratio, and the log difference of final and initial effective stresses. Initial and final stresses are calculated at mid layer to balance the lower stresses at the top of the layer and the higher stresses at the bottom of the layer.

$$\Delta S = \frac{Cc * h_r * \log(\sigma'_{fm}/\sigma'_{0m})}{(1+e)} \qquad \text{(Equation 4)}$$

where,

σ'_{0m} = effective stress under initial conditions, mid depth of the sand layer (3m) = 44.7 kN/m^2

σ'_{fm} = final effectives stress, mid depth of the sand layer (3m) = 50.3 kN/m^2

Cc = compression index = 0.069

e = void ratio of sand = 0.42

h_r = height of the full sand layer = 6 m

ΔS = estimated settlement of soils affected by dewatering = 14.9 mm

NUMERICAL MODEL

A numerical model is used to simulate flow into the French drains with sumps every 100 m. The three-dimensional finite-difference program MODFLOW is used for this purpose. MODFLOW is able to simulate steady and non-steady groundwater flow through multiple layers of different hydraulic conductivities (k). The model was run with steady-state conditions using hydraulic conductivity values presented in ***Table 1***. Only one side of the trench is modeled under the assumption that the groundwater flow-field is symmetric on either side of the trench. Constant-head boundaries were assigned to the outer edge of the model to simulate the water table and to the bottom of the trench to simulate the French drain. No-flow boundaries were assigned to the remaining edges and to the cutoff wall. The geometry is modeled at the deepest point of excavation (***Appendix E1***). The model calculates a distribution of hydraulic heads for the entire model domain, such that water mass is conserved (i.e., net water flow into and out of the model domain approaches zero) and the change in head for any individual model cell for successive mathematic iterations approaches zero (closure requirement set to maximum change of 0.01 foot).

For this calculation the model simulates 1000 m to the either side of the trench and 100 m along the trench axis to represent the distance between sumps. The trench cross section is found in ***Appendix E1***. Boundary conditions assign a total head equal to the water level (33 m) on the right edge, away from the trench, and a total head equal to the elevation of the sump and French drains (19 m) on the bottom of the trench. The steady-state result showing total head throughout the cross section is presented on ***Figure 2***. The steady-state flow rate is 2.05E-04 m^3/s.

The drawdown due to dewatering extends to approximately 220 m from the trench. The slope of the water table has potential to cause differential settlement from dewatering where one side of a structure has been dewatered more than the other side. If during the process of dewatering this water table slope is exhibited within the sand layer, the effects will be exaggerated due to the more compressible nature of sand as compared to that of the mudstone.

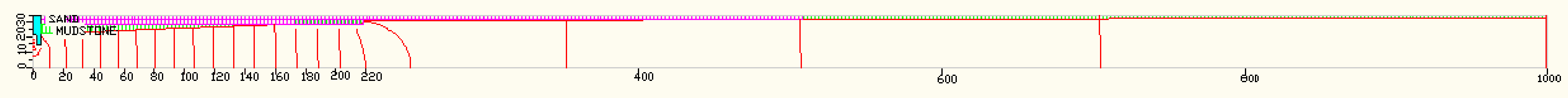
DIFFERENTIAL SETTLEMENT

The steepest horizontal hydraulic gradient from the numerical model is approximately 1 m of head loss in 20 m of distance (***Figure 2***). The difference in calculated settlement between the sand layer, fully dewatered (6 m) and the sand layer with all but 1 m dewatered (5 m) is determined by dividing the sand layer into three layers for calculation. The top 2 meters is always dry, as it is above the water table. The layer from 2 m to 5 m is saturated initially and dry after dewatering. The layer from 5 m to 6 m is saturated initially and either dry or saturated after dewatering, in order to compare the differential settlement from 1 m of head loss. Again this is an approximation using the 1-D consolidation equation. A more accurate calculation of differential settlement can be obtained using a numerical model.

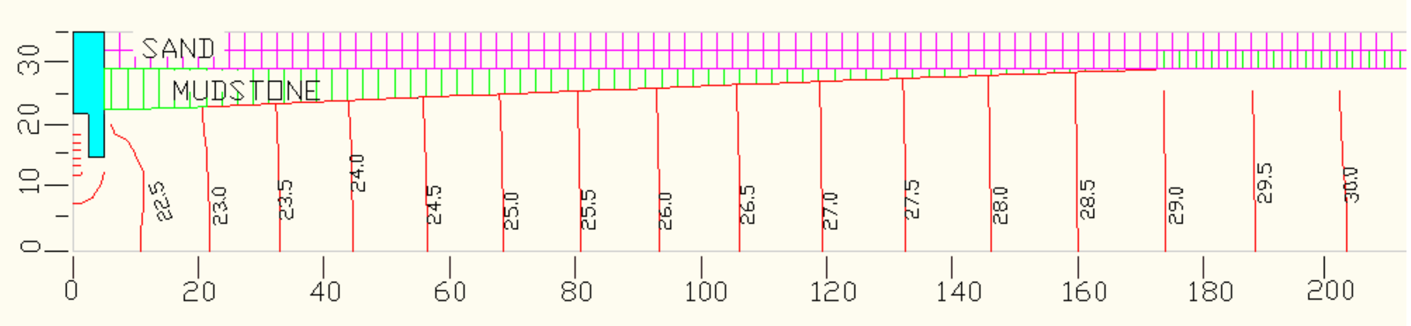
The difference in settlement from the analytical solution when the bottom 1 m is dry versus saturated is 0.65 mm. The ratio of 0.65 mm over 20 m is 1:30800. Compared to the lowest structural limit of 1:2500 for load-bearing concrete block walls, this is acceptable (***Reference 4***, Table 2-3). However, it is noted that the cone of depression is mild. Any additional efforts to pre-drain the excavation (e.g., wells or wellpoints) would have the potential to generate a sharper cone of depression with a larger potential for differential settlement.

FIGURE 2
RESULTS OF NUMERICAL ANALYSIS [1]

Full Cross Section



Zoomed-In Cross Section



[1] All dimensions and heads are in meters (m)

CONCLUSION AND DISCUSSION

Calculated flow rate from both sides of the trench will be 2.05E-04 m^3/s. Maximum settlement (at deepest location of drawdown) from dewatering effects is 14.9 mm. Differential settlement from a 1 m head difference is 0.65 mm in 20 m, or 1:30800.

REFERENCES

1. Bowles, J. E., *Foundation Analysis and Design,* The McGraw-Hill Companies, Inc., 1996, Sections 2.10 and 5.12.

2. Price, D., *Engineering Geology Principles and Practice,* Springer, 2009, p. 35.

3. Fang, H., *Foundation Engineering Handbook,* Second Ed., Chapman & Hall, New York, NY, 1991, p. 95.

4. USACE, "Settlement Analysis," EM 1110-1-1904, September 1990.

APPENDIX E3

RISK ANALYSIS

APPENDIX E3

RISK ANALYSIS

Risk analysis involves considering and rating potential hazards for the duration of the dewatering activity. Hazards are assigned a consequence and likelihood rating according to the following guidelines:

RISK ANALYSIS RATING

Consequence Rating	Safety	Health	Environment
1	First Aid	Immediate	Minor
2	Medical	Temporary	Short term
3	Lost time	Short term	Long term
4	Disability	Long term	Serious
5	Fatality	Fatal	Catastrophic

Likelihood Rating	Frequency
1	Highly Unlikely
2	Remote
3	Possible
4	Probable
5	Certain

The risk factor is the product of the consequence and likelihood rating. A risk factor 1 to 4 is low risk, 5 to 12 is medium risk, and 13 to 25 is high risk.

[Address the consequence and likelihood of the following events and any other relevant situation with potential to be hazardous. Suggest control measure and address the consequence and likelihood with the proposed control measures in place.]

The following hazards are considered for this project:

1. Mobilization and Demobilization

    a. Loading/unloading of equipment on site

        i. Objects falling from lifting equipment

ii. Sudden failure of wire rope/chain or hydraulic/mechanical system

iii. Materials falling from vehicle

iv. Injuries by vehicle door opening/closing

v. Injury to personnel and property damage by manual handling

vi. Run over/stacked/crushed by a vehicle

vii. Poor lighting

viii. Poor ground/road condition

2. Well Installation

a. PVC casing erection

i. Personnel getting wet

ii. Slip trip and fall due to wet surfaces

iii. Risk of PVC casing/pump falling from lifting equipment

iv. Eye contamination

v. Personnel falling into drilled hole

b. Well hole filled with aggregate

i. Dust pollution from aggregates

c. Electrical equipment installation

i. Serious injury to personnel by electric shock

ii. Fire hazard

3. Discharge Installation

a. Discharge tank installation

i. Objects falling from lifting equipment while placing the discharge tank

ii. Seepage of water on site

iii. Overflow of water from tank

b. Discharge pipe installation

i. Risk of back injury

ii. Temporary and permanent back injury by lifting heavy objects

c. Person walking over loose sand

i. Slip, trip, and fall while walking over loose sand

d. Person working near/in the water

i. Personnel drowning and shock

ii. Personnel getting wet

4. Discharge Installation, Operation, and Maintenance

a. Discharge tank installation and flexible hose connection

i. Objects falling from crane

ii. Crane overturning, causing crash injuries

iii. Serious injury to personnel and property damage

iv. Sudden failure of wire rope/chain and hydraulic/mechanical system

v. Seepage of water on site

vi. Overflow of water from brake tank

5. French Drain System

a. Excavation of trench and preparation of French drain system

i. Objects falling into trench

ii. Noise pollution from excavator

iii. Personnel falling into the excavation

iv. Serious injury to the personnel and property damage

6. Construction of French Drains

a. Pump switched off (temporary)

i. Electrocution

b. Pump removal and reinstall (temporary)

i. Injury to personnel and property damage

ii. Water spillages

c. PVC casing removal

i. Sudden failure of ware rope

ii. Property damage

iii. Risk of back injury

iv. Temporary and permanent back injury by lifting heavy objects

d. PVC pipe connection (for French drain system)

i. Risk of back injury

ii. Temporary and permanent back injury by lifting objects

iii. Property damage

e. Pump switch on

i. Electrocution

ii. Water spillages

7. French Drain System – Sump Pump Installation

a. PVC pipe erection, excavated French drain area filled with aggregate, pump erection, power connection

i. Risk of PVC casing/sub-pump falling from lifting equipment

ii. Personnel getting wet

iii. Eye contamination

iv. Personnel falling into drilled hole

v. Personnel injured by electric shock

8. French Drain System – Sump Pump Operation and Maintenance

a. Noise pollution by generator

b. Pump failure

c. Spillage of diesel/engine oil

9. Observation Points – Drilling Hole

   a. Movement of drilling rig, drilling operations

      i. Toppling over when travelling on sloped ground

      ii. Risk of collision

      iii. Injury due to rotating equipment

10. Observation Points – Pipe Installation

   a. Water jetting, installation of well casing

      i. Personnel getting wet

      ii. Eye contamination

      iii. Risk of equipment falling from lifting equipment

11. Material storage

   a. Injury to personnel

   b. Manual handling injuries

   c. Materials damage

   d. Object falling from height

   e. Slip, trip, and falls

   f. Chance of fire and explosion hazard

   g. Spillages (chemicals/oil)

12. Fuel Storage

   a. Personnel injury or property damage by fire/explosion hazard

   b. Spillage and environmental impact

   c. Partial/total power loss

13. Working in Hot Weather

   a. Potential for heat stress, dehydration, sunburn, and heat stroke, etc.

14. Out-of-hours Maintenance

a. Lane watching, poor lighting, difficult access

15. Watch man working at night

    a. Inadequate lighting

    b. Injury to the personnel or property damage

16. Unauthorized entry to dewatering work activity area

    a. Injuries to personnel

17. Hand tools

    a. Personnel injury and property damage

    b. Cuts, burns, heat disorder, eye injury

Using activity 3d as an example, two potential hazards are personnel drowning or shock and personnel getting wet. The consequences of these hazards are potentially fatal (Consequence Rating = 5) and the likelihood is possible (Likelihood Rating = 3). The risk factor before control measures of this hazard is 15, which falls in the high-risk category.

The following control measures for hazards in 3d are suggested:

1. Personal Protection Equipment (PPE) rule must be enforced (life jacket/buoy, swimming glass, life line, high visibility clothing, hard hat, gloves, and safety footwear). Personnel working without proper PPE will be removed from site permanently at all times.

2. Full time watch keeping is necessary by foreman.

3. Reduce the personnel and work time near water.

4. First aider (from Main Contractor) shall be placed upon request.

If these control measures are implemented the possible consequences are reduced to medical (Consequence Rating = 2) and the likelihood of the event occurring is highly unlikely (Likelihood Rating = 1). The risk factor of this hazard after implementing control measures is now 3, which falls in the low risk category.

APPENDIX E4

GEOTECHNICAL INVESTIGATION REPORT

APPENDIX E4

GEOTECHNICAL INVESTIGATION REPORT

		Borehole:

Equipment: T-30	**Start Date:** **End Date :**	**Coordinates:**
Method: Rotary Boring/Coring	**Bore Diameter (mm):** 142	**Surface Elevation (m):** 4.63 NADD
Fluid Flush: Guar Gum water mixture	**Core Diameter (mm):** 83	**End Borehole Depth (m below surf.):** 32.60

Borehole Progress			Depth	Graphic	Strata Description	Datum	Test	Test Records				Sample/ Test	Rock Core Quality		
Flush ret. %	W.D. [m]	Date	[m]	Symbol			No.	N	Seating Drive	Blwct 1	Blwct 2	No.	TCR [%]	SCR [%]	RQD [%]
		15/10	0.0		0.00 m to 2.00 m - Light brown, silty, fine to medium, SAND, with gravel size fragments of slightly cemented sand.							B1			
			2.0		2.00 m to 6.00 m - Medium dense to dense, light greyish brown, silty, fine to medium, SAND.		S2	27	10	13	14				
												B3			
			3.0				S4	31	6	12	19				
												B5			
			4.0				S6	36	12	17	19				
												B7			
			5.0				S8	37	14	18	19				
												B9			
			6.0		6.00 m to 14.45 m - Extremely weak to very weak, very thickly bedded, light brown to dark grey, MUDSTONE, slightly to moderately weathered. Close to medium spaced, horizontal to sub-horizontal fractures. from 6.00 m to 6.80 m - Layer of weak Gypsum. from 6.60 m to 6.80 m - Extremely weak, non intact core.		S10	50	25	50	-				
80%			7.0									C1	100	45	15
			8.0		from 7.30 m to 8.50 m - Low recovery, sample washed out during drilling.							C2	33	21	21
80%			9.0		from 9.10 m to 9.50 m - Layer of very weak Sandstone. from 9.65 m to 10.00 m - Extremely weak, non intact core.							C3	87	77	77
80%			10.0												

Project: Location Client: Job No.:	Logged by:MS Checked by: MI Plate:

ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Borehole:

Equipment: T-30
Start Date:
End Date :
Coordinates:

Method: Rotary Boring/Coring
Bore Diameter (mm): 142
Surface Elevation (m): 4.63 NADD

Fluid Flush: Guar Gum water mixture
Core Diameter (mm): 83
End Borehole Depth (m below surf.): 32.60

Strata Description

6.00 m to 14.45 m - Extremely weak to very weak, very thickly bedded, light brown to dark grey, MUDSTONE, slightly to moderately weathered. Close to medium spaced, horizontal to sub-horizontal fractures.
from 10.75 m to 11.00 m - Extremely weak, non intact core.

14.45 m to 17.30 m - Weak to medium strong, very thickly bedded, light grey, GYPSUM, moderately weathered. Close to medium spaced, sub-horizontal fractures.
from 14.45 m to 14.65 m - Non intact core.
from 14.85 m to 15.20 m - Non intact core.
from 15.20 m to 15.35 m - Band of very weak, mudstone.
from 15.90 m to 16.50 m - Non intact core, low recovery.
from 16.50 m to 17.30 m - Weak to medium strong.

17.30 m to 30.60 m - Very weak, very thicly bedded, light brown to light grey, SILTSTONE, slightly weathered. Close to medium spaced, horizontal to sub-horizontal fractures.

Borehole Progress

Flush ret. %	W.D. [m]	Date	Depth [m]
80%			11.0
		16/10	
80%			12.5
80%			13.5
80%			15.0
80%			16.5
80%			18.0
80%			19.5

Depth [m]: 10.0, 11.0, 12.0, 13.0, 14.0, 15.0, 16.0, 17.0, 18.0, 19.0, 20.0

Datum: -6.0, -7.0, -8.0, -9.0, -10.0, -11.0, -12.0, -13.0, -14.0, -15.0

Rock Core Quality

Sample/Test No.	From	To	TCR [%]	SCR [%]	RQD [%]
C4	10.00	11.00	100	53	53
C5	11.00	12.50	87	83	83
C6	12.50	13.50	100	80	70
C7	13.50	15.00	100	67	63
C8	15.00	16.50	83	13	12
C9	16.50	18.00	93	93	93
C10	18.00	19.50	100	100	100
C11	19.50				

Project:
Location:
Client:
Job No.:

Logged by:MS
Checked by: MI
Plate:

GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

		Borehole:

Equipment: T-30
Method: Rotary Boring/Coring
Fluid Flush: Guar Gum water mixture
Start Date:
End Date :
Bore Diameter (mm): 142
Core Diameter (mm): 83
Coordinates:
Surface Elevation (m): 4.63 NADD
End Borehole Depth (m below surf.): 32.60

Flush ret. %	W.D. [m]	Date	Depth [m]	Graphic Symbol	Strata Description	Datum	Test No.	N	Seating Drive	Blwct 1	Blwct 2	Sample/ Test No.	TCR [%]	SCR [%]	RQD [%]
			20.0		17.30 m to 30.60 m - Very weak, very thicly bedded, light brown to light grey, SILTSTONE, slightly weathered. Close to medium spaced, horizontal to sub-horizontal fractures.	16.0						C11	97	97	97
80%		17/10	21.0			17.0						C12	93	83	83
80%			22.0			18.0									
			23.0		from 23.30 m to 23.60 m - Layer of weak Gypsum.	19.0						C13	100	100	97
80%			24.0			20.0						C14	100	93	83
80%			25.0			21.0									
			26.0			22.0						C15	100	100	100
80%			27.0		from 27.30 m to 27.80 m - Layer of very weak Sandstone. from 27.65 m to 27.80 m - Extremely weak, non intact core.	23.0						C16	100	81	69
80%			28.0			24.0									
			29.0			25.0						C17			
			30.0												

Project:	Logged by:MS
Location:	Checked by: MI
Client:	Plate:
Job No.:	

GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

		Borehole:

Equipment: T-30 | **Start Date:** | **Coordinates:**
End Date :
Method: Rotary Boring/Coring | **Bore Diameter (mm):** 142 | **Surface Elevation (m):** 4.63 NADD
Fluid Flush: Guar Gum water mixture | **Core Diameter (mm):** 83 | **End Borehole Depth (m below surf.):** 32.60

Borehole Progress			Depth	Graphic	Strata Description	Datum	Test	Test Records				Sample/ Test	Rock Core Quality		
Flush ret. %	W.D. [m]	Date	[m]	Symbol			No.	N	Seating Drive	Blwct 1	Blwct 2	No.	TCR [%]	SCR [%]	RQD [%]
80%			30.0		17.30 m to 30.60 m - Very weak, very thicly bedded, light brown to light grey, SILTSTONE, slightly weathered. Close to medium spaced, horizontal to sub-horizontal fractures.							C17	87	80	80
					30.60 m to 32.60 m - Weak to medium strong, very thickly bedded, light grey to grey, GYPSUM, slightly weathered. Close to medium spaced, sub-horizontal fractures. from 30.80 m to 31.70 m - With pockets of silt.	-26.0									
			31.0									C18	93	93	83
80%						-27.0						31.70			
			32.0									C19	100	94	83
80%					at 32.60 m - End of borehole.	-28.0						32.60			
			33.0												
						-29.0									
			34.0												
						-30.0									
			35.0												
						-31.0									
			36.0												
						-32.0									
			37.0												
						-33.0									
			38.0												
						-34.0									
			39.0												
						-35.0									
			40.0												

Project:	Logged by:MS
Location:	Checked by: MI
Client:	Plate:
Job No.:	

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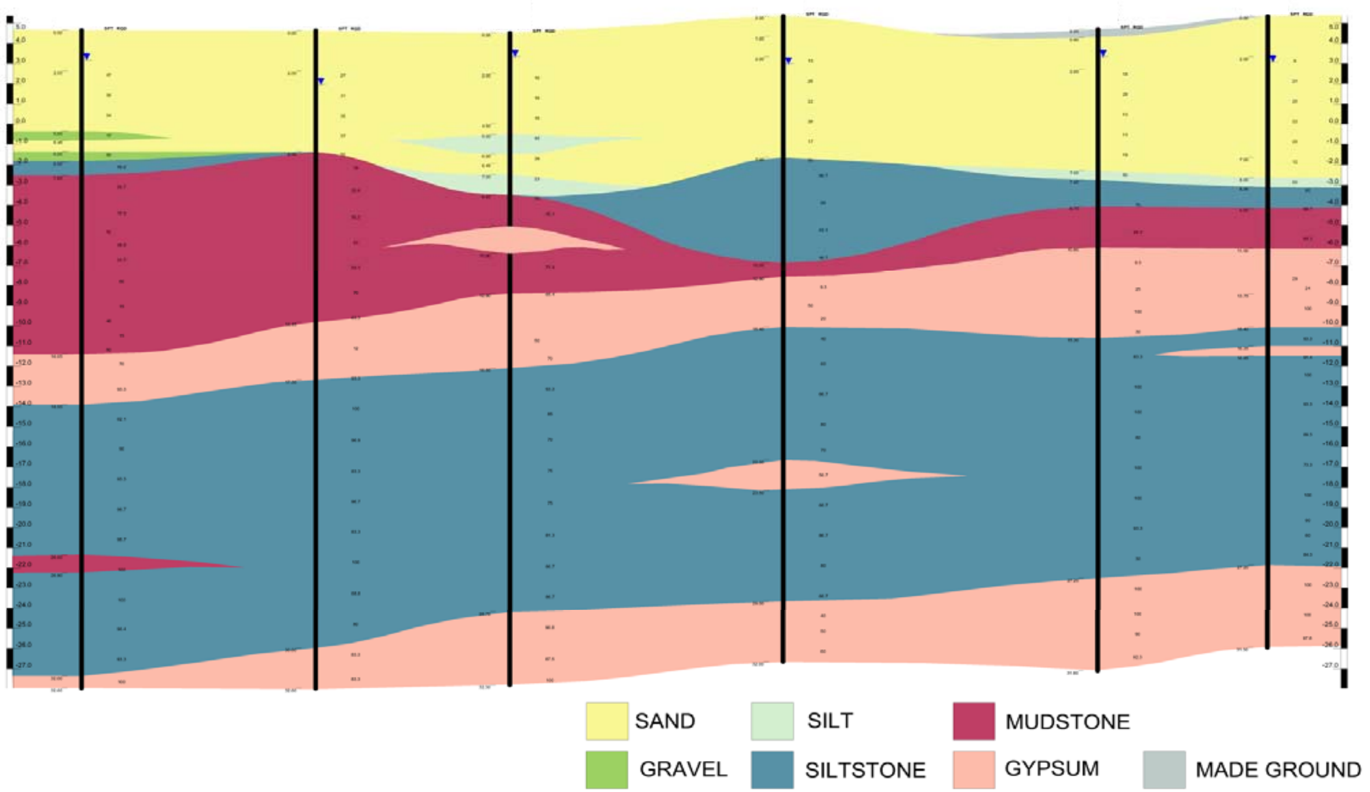
GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)



Geological Cross Section
Project:
Location:
Client:
Job No.:

Logged by: IF
Checked by: MI
Plate:

ABU DHABI CITY MUNICIPALITY

DEPARTMENT OF MUNICIPAL AFFAIRS

GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Grain Size Analysis (Sieve Test)

Project Name:	Proj. Ref. No.:
Test Date:	Lab. Room Temp.: 23°C

Borehole No.:	
Depth (m):	
Sample Ref. No.:	3
Sample Type:	Small Disturbed
Sample Description:	Very Silty Sand
Dry Sample Weight (g):	201.1

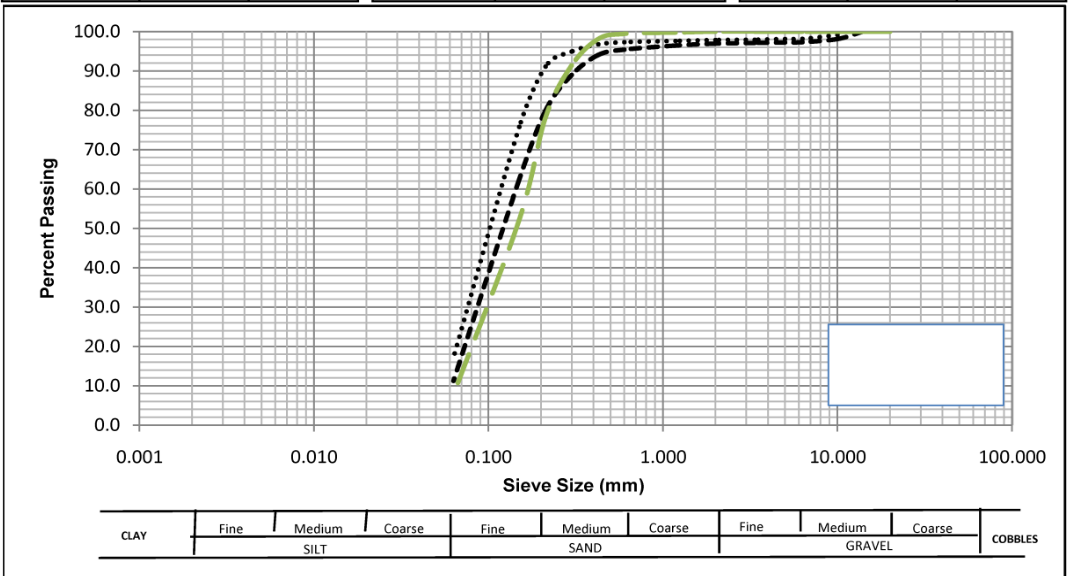
Sieve Size (mm)	Retained Weight (g)	% Passing
50.00	0.00	100.0
37.50	0.00	100.0
28.00	0.00	100.0
20.00	0.00	100.0
14.00	0.00	100.0
10.00	1.90	99.1
6.30	3.80	98.1
5.00	3.90	98.1
3.35	4.20	97.9
2.00	4.50	97.8
1.18	4.80	97.6
0.60	5.40	97.3
0.43	6.70	96.7
0.30	10.30	94.9
0.21	17.80	91.1
0.15	48.80	75.7
0.06	166.80	17.1

Borehole No.:	
Depth (m):	
Sample Ref. No.:	1
Sample Type:	Small Disturbed
Sample Description:	Silty Sand
Dry Sample Weight (g):	130.7

Sieve Size (mm)	Retained Weight (g)	% Passing
50.00	0.00	100.0
37.50	0.00	100.0
28.00	0.00	100.0
20.00	0.00	100.0
14.00	0.00	100.0
10.00	2.50	98.1
6.30	3.50	97.3
5.00	3.70	97.2
3.35	3.80	97.1
2.00	4.00	96.9
1.18	4.70	96.4
0.60	6.00	95.4
0.43	7.80	94.0
0.30	14.50	88.9
0.21	26.30	79.9
0.15	49.20	62.4
0.06	116.10	11.2

Borehole No.:	
Depth (m):	
Sample Ref. No.:	2
Sample Type:	Small Disturbed
Sample Description:	Silty Sand
Dry Sample Weight (g):	107.1

Sieve Size (mm)	Retained Weight (g)	% Passing
50.00	0.00	100.0
37.50	0.00	100.0
28.00	0.00	100.0
20.00	0.00	100.0
14.00	0.00	100.0
10.00	0.00	100.0
6.30	0.00	100.0
5.00	0.00	100.0
3.35	0.00	100.0
2.00	0.00	100.0
1.18	0.30	99.7
0.60	0.60	99.4
0.43	2.00	98.1
0.30	9.20	91.4
0.21	23.30	78.2
0.15	51.20	52.2
0.06	98.90	7.7



SJ-F-IMS-RRF-012, R-01, D-16/02/10

ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Packer Test Permeability

Project Name:		Client:	
Project Ref. No:		Date:	

Top of test section (m):	17.5	Borehole No.:		Test No.:	13
Bottom of test section (m):	22.0	Packer pressure (bar):		Packer Type:	Double Packer
Length of test section (m):	4.5	Hose length (m):	8.2	Hose Dia. (mm):	25
Bottom of hole at time of test (m):	77.0	Dia. of hole in test area (mm):	99		
Bottom of casing (m):	15.7	Ground level (NADD):	18.005		
Initial ground water level (m): *(1)*	1.3	Crew/operator:			
Gauge height above ground level (m): *(2)*	0.9	Type of rock:	Calacrenite, Musdstone and Gypsum		

Test Record

Period	Time (min)	Gauge Pressure (bar)	Flowmeter Reading (m³)	Water Intake (m³)	Average Flow, Q (m³/s)
1st period	0	0.6	25.4468		0.000096
	5		25.4785	0.0317	
	10		25.504	0.0255	
	15		25.5331	0.0291	
2nd period	0	1.2	25.5396		0.000140
	5		25.586	0.0464	
	10		25.6267	0.0407	
	15		25.666	0.0393	
3rd period	0	2.4	25.6806		0.000172
	5		25.7335	0.0529	
	10		25.7841	0.0506	
	15		25.8351	0.0510	
4th period	0	1.2	25.8376		0.000106
	5		25.8731	0.0355	
	10		25.903	0.0299	
	15		25.9327	0.0297	
5th period	0	0.6	25.9329		0.000063
	5		25.9514	0.0185	
	10		25.9701	0.0187	
	15		25.9893	0.0192	

Computation Record

$k(m/s)= (Q /2\pi Hl) \log_e (l/r)$

k-The Permeability in meters per second
Q-is the rate of injection in cubic metes per second
h-is the pressure head of water in the test in meters
l-is the length of the test section in meters
r- is the radius of test section in meters

L = (100/l) (Q/H) in lugeon units

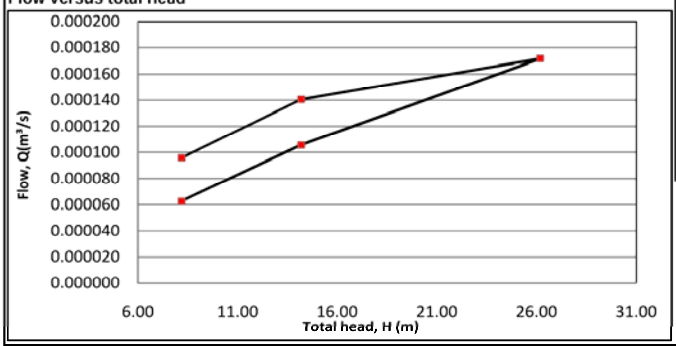
l- is length of test section in metres
Q/H- is slope of graph, Q in litres/min & H in metres

H= (1)+(2)+(3)-(4)-(5)

H- Total head

Period	Head of water, m (3)	Friction losses: in basic pipeworks	Friction losses: in extra rods &
1st	6	0.001544	0.00223
2nd	12	0.001544	0.00784
3rd	24	0.001544	0.01151
4th	12	0.001544	0.00455
5th	6	0.001544	0.00167

Flow versus total head



Period	Total Head, H (m)	Flow (m³/s)	Lugeon units	k (m/s)
1st	8.20	0.000096	15.6	1.87E-06
2nd	14.19	0.000140	13.2	1.58E-06
3rd	26.19	0.000172	8.7	1.05E-06
4th	14.19	0.000106	9.9	1.19E-06
5th	8.20	0.000063	10.2	1.22E-06

Remarks: There is very less flow in 3 rd peroid

SJ-F-IMS-RRF-035, R-01, D-16/02/10

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Samp	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND

Diameter-D (mm)	71.30
Area-A (mm²)	3992.72
Height-Hs (mm)	15.50
Dry Wt.+Ring (g)	249.64
Ring Wt. (g)	99.40
Particle Density (Gs)	2.43
Degree of Saturation S_o (%)	146.55

	Initial	Final
Wet Wt.+Ring (g)	287.69	279.62
Height-H (mm)	22.00	20.59
Volume-V (cm^3)	87.84	82.21
Wet Density (g/cm^3)	2.14	2.19
Moisture Content (%)	25.33	19.95
Void ratio (e)	0.42	0.33
Dry Density (g/cm^3)	1.71	1.83

Increment From/To (kPa)	0-50	50-100	100-200	200-400	400-800	800-1400	1400-700	700-0
Pressure (kPa)	50	100	200	400	800	1400	700	1
Deformation of apparatus (mm)	0	0	0	0	0	0	0	0
Consolidated height,$\hat{H}$ (mm)	21.77	21.59	21.40	21.16	20.89	20.60	20.686	20.876
Voids ratio (e)	0.41	0.39	0.38	0.37	0.35	0.33	0.34	0.35
Height change-H (mm)	0.23	0.18	0.19	0.24	0.27	0.29		
Pressure change-p (kPa)	50	50	100	200	400	600		
Volume compress-m_v (m^2/MN)	0.21	0.17	0.09	0.06	0.03	0.02		
t_{100} (min)	4.00	7.84	8.41	15.21	16.00	30.25		
Average specimen height (mm)	21.89	21.68	21.49	21.28	21.02	20.74		
Coeff. of consol-C_v (m^2/year)	1.48	0.74	0.68	0.37	0.34	0.18		

Increment	Pressure (kPa)	Test Date
0-50	50	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.56	0.0	0.00
0.1	8.56	0.3	0.00
0.2	8.55	0.4	-0.01
0.5	8.48	0.7	-0.08
1	8.43	1.0	-0.13
2	8.40	1.4	-0.16
4	8.36	2.0	-0.20
8	8.34	2.8	-0.22
15	8.33	3.9	-0.23

Height before loading (mm)	22
Height after loading (mm)	21.77

Increment	Pressure (kPa)	Test Date
50-100	100	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.33	0.0	0.00
0.1	8.31	0.3	-0.02
0.2	8.28	0.4	-0.05
0.5	8.24	0.7	-0.09
1	8.21	1.0	-0.12
2	8.19	1.4	-0.14
4	8.18	2.0	-0.15
8	8.17	2.8	-0.16
15	8.16	3.9	-0.17
30	8.15	5.5	-0.18

Height before loading (mm)	21.77
Height after loading (mm)	21.59

GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Sample No.:	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND

Increment	Pressure (kPa)	Test Date
100-200	200	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	8.15	0.0	0.00
0.1	8.13	0.3	-0.02
0.2	8.10	0.4	-0.05
0.5	8.04	0.7	-0.11
1	8.01	1.0	-0.14
2	8.00	1.4	-0.15
4	7.99	2.0	-0.16
9	7.98	3.0	-0.17
15	7.97	3.9	-0.18
30	7.96	5.5	-0.19

Height before loading (mm)	21.59
Height after loading (mm)	21.40

Increment	Pressure (kPa)	Test Date
200-400	400	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.96	0.0	0.00
0.1	7.92	0.3	-0.04
0.2	7.88	0.4	-0.08
0.5	7.82	0.7	-0.14
1	7.79	1.0	-0.17
2	7.78	1.4	-0.18
4	7.76	2.0	-0.20
8	7.75	2.8	-0.21
15	7.74	3.9	-0.22
30	7.73	5.5	-0.23
60	7.72	7.7	-0.24

Height before loading (mm)	21.40
Height after loading (mm)	21.16

Increment	Pressure (kPa)	Test Date
400-800	800	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.72	0.0	0.00
0.1	7.69	0.3	-0.03
0.2	7.65	0.4	-0.07
0.5	7.56	0.7	-0.16
1	7.53	1.0	-0.19
2	7.51	1.4	-0.21
4	7.50	2.0	-0.22
8	7.49	2.8	-0.23
20	7.47	4.5	-0.25
30	7.46	5.5	-0.26
60	7.45	7.7	-0.27

Height before loading (mm)	21.16
Height after loading (mm)	20.89

Increment	Pressure (kPa)	Test Date
800-1400	1400	20/08/2011

Elapsed time (min)	Height sensor reading (mm)	Root of time (min)	Change in height (mm)
0	7.45	0.0	0.00
0.1	7.43	0.3	-0.02
0.2	7.41	0.4	-0.04
0.5	7.32	0.7	-0.13
1	7.27	1.0	-0.18
2	7.25	1.4	-0.20
4	7.23	2.0	-0.22
8	7.21	2.8	-0.24
15	7.20	3.9	-0.25
30	7.19	5.5	-0.26
60	7.17	7.7	-0.28
120	7.16	11.0	-0.29

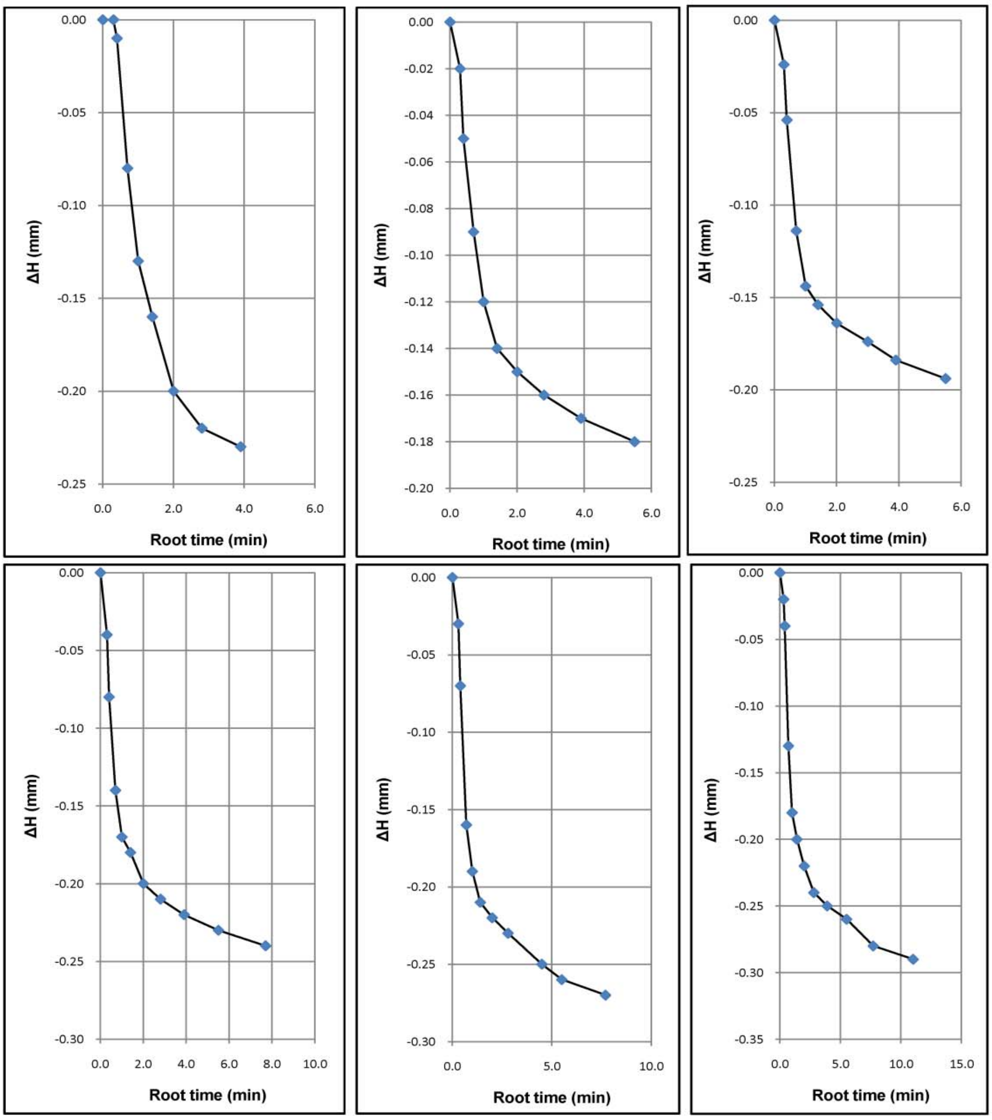
Height before loading (mm)	20.89
Height after loading (mm)	20.60

GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Sample No.:	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND



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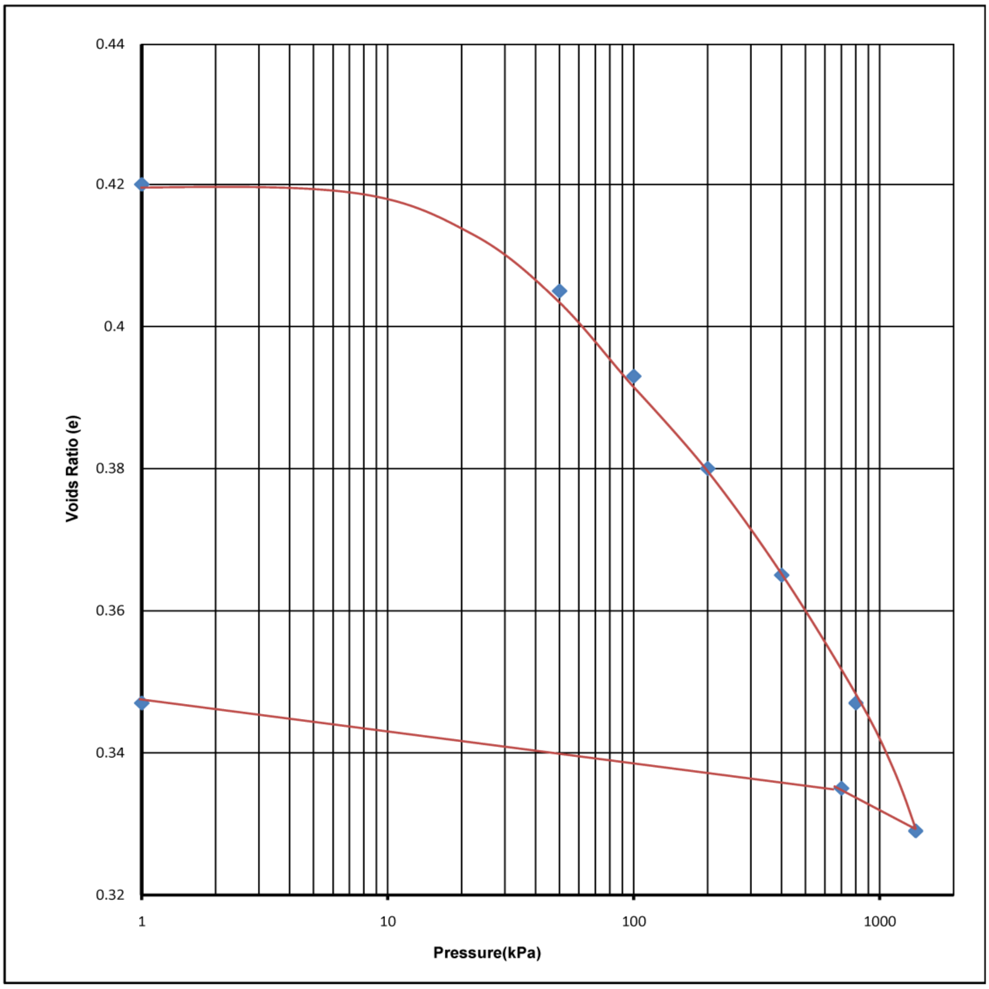
GEOTECHNICAL INVESTIGATION REPORT
(CONTINUED)

Hydraulic Consolidation Test

Project Name:		Lab. Room Temp.:	23°C
Proj. Ref. No:		Test Date:	
Orientation:	Parallel To The Vertical.		

Borehole No.:		Sample No.:	
Depth (m):		Sample Type:	Undisturbed Sample
Moisture Condition:	-	Sample Description:	Silty SAND



Preconsolidation Pressure (σ_c') = 85 (kPa)

Recompression Index (C_r) = 0.0022

Compression Index (C_c) = 0.069

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APPENDIX F

GEOTECHNICAL INVESTIGATION PERMITTING REQUIREMENTS

دائـرة الـشـؤون الـبـلـديـة
بلديــة مدينــة أبوظبــي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

الترخيص الإلكتروني لأعمال استكشاف التربة
e-Permitting of Geotechnical Investigation Works

Presenting Division :الجهة المقدمة	Presentation by: مقدم العرض
• إدارة تراخيص البناء	• د. مازن أديب - مستشار
• Construction Permit Division	• Dr. Mazen Adib – Advisor

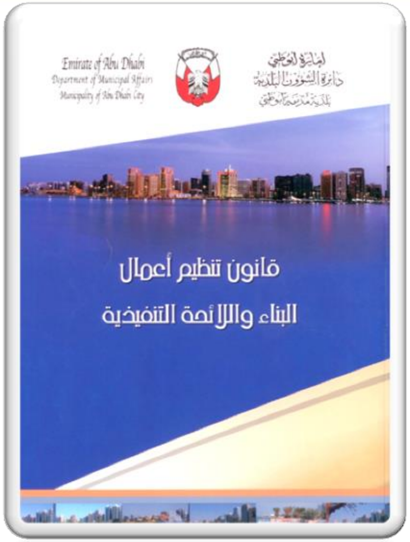
October 8, 2013

Content

- Objectives of Abu Dhabi City Municipality (ADM) service
- Resources
- Information and follow up
- Legal framework
- Requirements for performing geotechnical works in Abu Dhabi City
- List of tests and activities for accreditation
- Requirements timeline
- Permitting process
- Registration on the permitting system (CDP)
- General guidelines for online submittal
- Online application steps for permitting geotechnical works
- Required documents
- Typical documents content

Objectives of ADM service

- Help entities comply with the regulations
- Ensure smooth implementation of the permitting process
  - ✓ Online registration on the permitting system (known as CDP)
  - ✓ Online application on CDP
- Ensure awareness with the requirements and submittal process
  - ✓ Press release on 23 September, 2013
  - ✓ Informative Brochure
  - ✓ Workshop on 8 October, 2013
  - ✓ Documentations and presentations available on ADM website
- Assist in clarifying and explaining issues during implementation
- Provide customers with continuous support during the implementation to solve issues that may arise
  - ✓ Follow up is available during business days from 2:00 pm to 3:30 pm (please visit Construction Permit Division on 1st Floor of ADM bldg. located on Sheikh Zayed Str.)

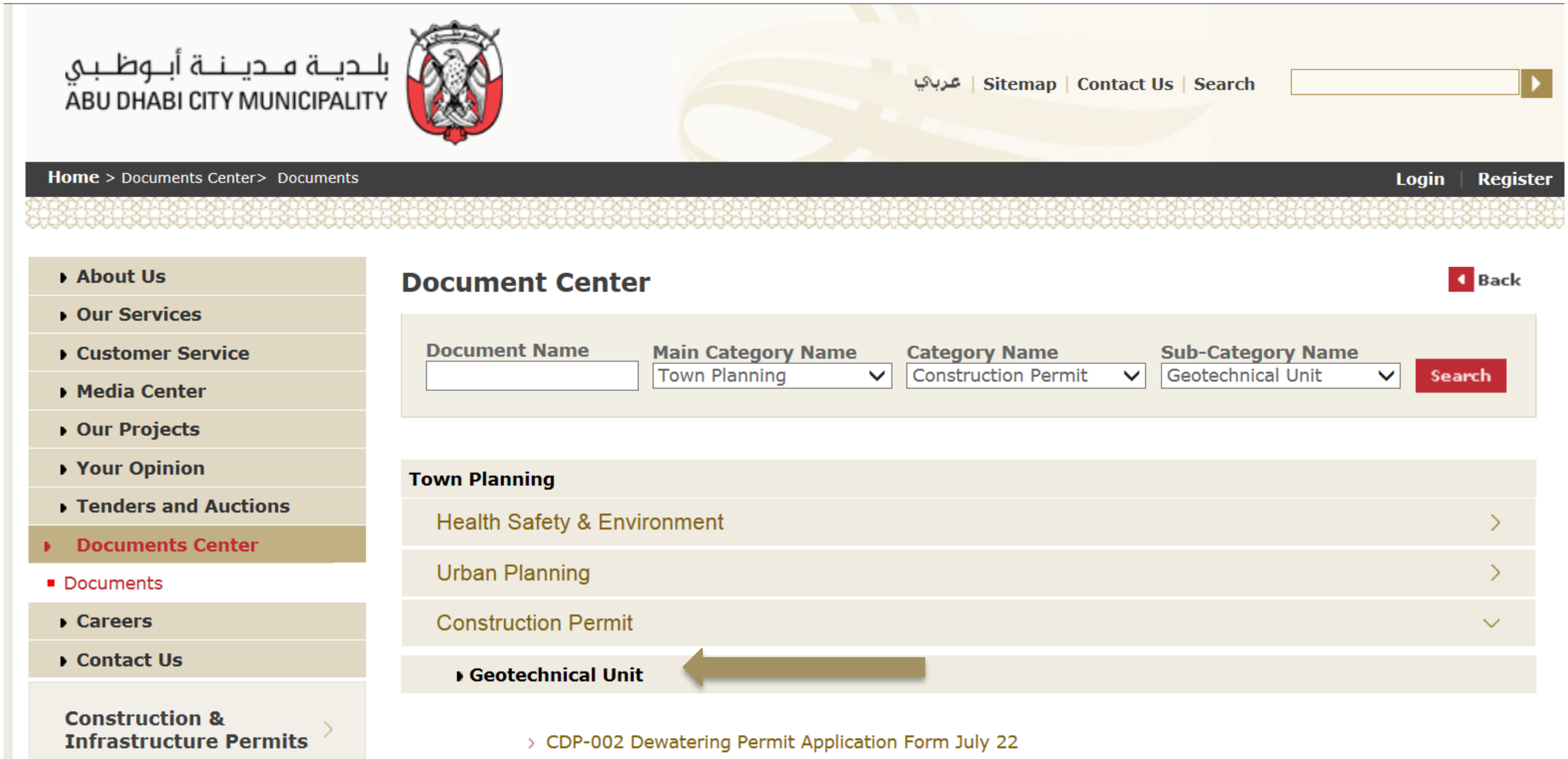


Resources

دائـرة الـشـؤون الـبـلـديـة
بلـديـة مدينـة أبوظبـي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

All necessary forms, presentations and guidelines for the geotechnical and dewatering permits are available online from Abu Dhabi City Municipality website (adm.gov.ae) in the following sequence:

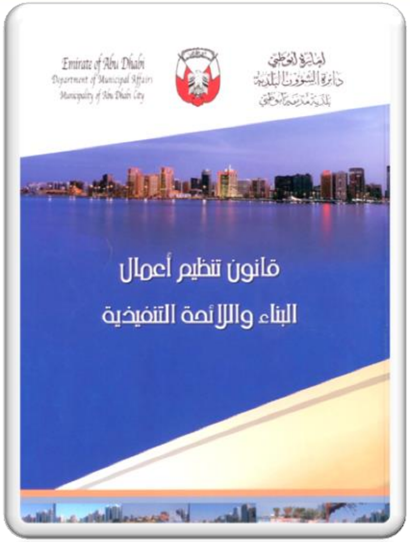
Documents Center/Documents/ Town Planning /Construction Permit/ Geotechnical Unit



Legal framework

دائرة الشؤون البلدية
بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

- **Law No. (4) of 1983 for the regulation of construction works, (pages 98 to 103)**
  - ✓ Sets minimum requirements for investigation, sampling, testing, and reporting associated with geotechnical works.
  - ✓ Requires a supervision by an engineer knowledgeable in geotechnical discipline.
  - ✓ Requires submittal of scope of work to ADM



- **Decision No. 181/2012 - Department of Municipal Affairs**
  - ✓ Requires **all entities** to obtain a permit from the Municipality before executing any geotechnical investigation works in the Emirate of Abu Dhabi.
  - ✓ Identifies submittal requirements
    a. Valid Commercial License
    b. Scope of Work approved by Consultant
    c. Accreditation Certificates of the laboratory from the concerned entities
    d. Equalization by MOHE of academic credentials of supervising engineer
    e. Environment, Health, and Safety (HSE) Risk Assessment.

Requirements for performing geotechnical works

دائرة الشؤون البلدية
بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

Performing entities to comply with the following:

1. Obtain a geotechnical permit for each investigation; service is available online from October 20, 2013
2. **No field work** without a permit after 31st December, 2013 (i.e, permit is **MANDATORY** after this date)
3. Register in ADM EHSMS by 31st December, 2013
4. Obtain EHS approval of entity health, safety and environment (HSE) manual by 31st December, 2014
5. Achieve accreditation in geotechnical testing, acceptable to Abu Dhabi Quality and Conformity Council (ADQCC) by 31st December, 2014

I. Current ongoing work (or already committed but-not started work) **DOES NOT** require a permit
II. Field work starting **AFTER** 31st December 2013 **DOES** require a permit
III. Typical scope variations (i.e, change in scope by -10 to +25%) **DOES NOT** require a new permit
IV. Significant scope variations (i.e, by more than 25%) or new phases in a large project **DOES** require a new permit. Geotechnical Laboratory/consultant to decide if a variation warrants a new permit
V. Permit is valid for the duration of the works
VI. Meetings may be requested by ADM to clarify submittals.

List of tests and activities for accreditation

Field works:

1. Standard Penetration Test (SPT) and split-barrel sampling of soil
2. Description and identification of soils (visual-manual procedure)
3. Determining Rock Quality Designation (RQD) of rock core
4. Determining subsurface liquid levels in a borehole or monitoring well (observation well)
5. Sampling ground-water monitoring wells

Laboratory tests:

1. Particle-size distribution (gradation) of soils using sieve analysis
2. Liquid limit, plastic limit, and plasticity index of Soils
3. pH, Sulphates and Chlorides content in soil samples and groundwater samples
4. Unconfined compressive strength of intact rock core specimens
5. Determination of water (moisture) content of soil and rock by mass.
6. Preparation of rock core as cylindrical test specimens

Report:

1. Preparation of soil report

Requirements timeline

20 October 2013

- Service Available Online

31 December 2013

- Registration in ADM EHSMS is required after this date
- No field work without permit (i.e, a permit is **MANDATORY after this date**)

30 April 2014

- Equalization of academic degrees from Ministry of Higher Education is required after this date

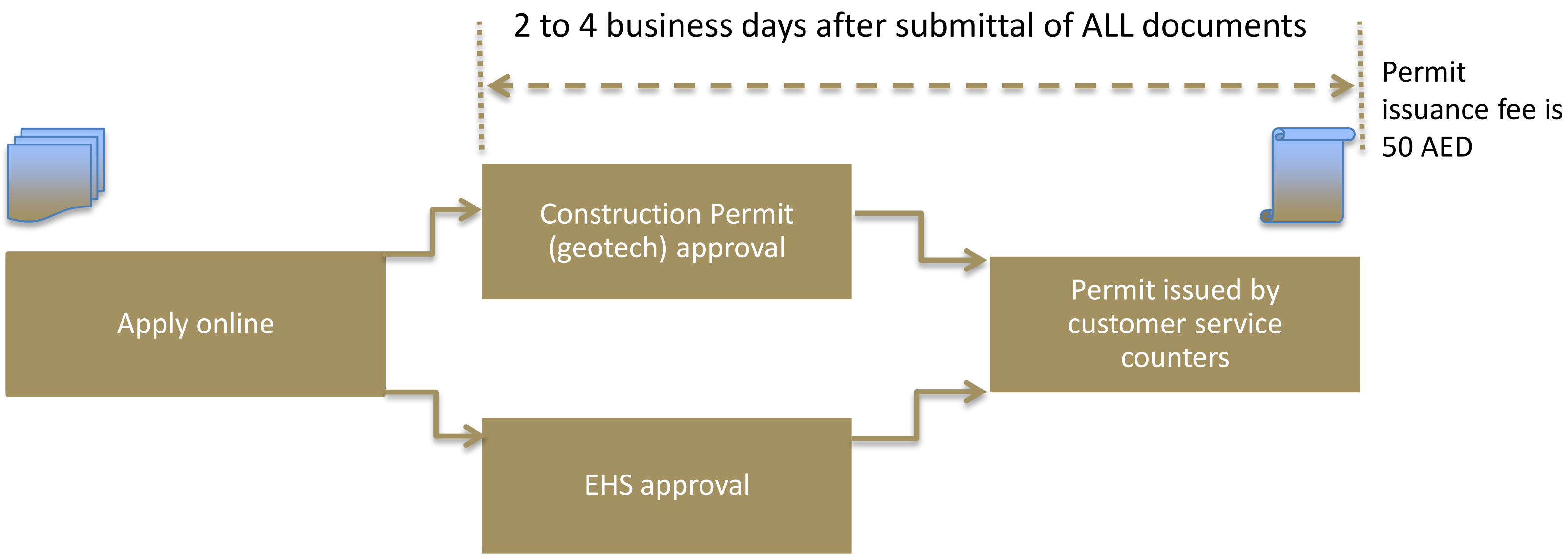
31 December 2014

- Accreditation Approved by ADQCC is required after this date
- Approval of EHS Manual is required after this date

Permitting process

Procedures for obtaining the permit

- Submit an application online through the CDP
- Upload the required documents and drawings
- Follow up on application status through the CDP
- Obtain approval from Construction Permit and EHS Divisions
- Obtain the permit through customer service counters

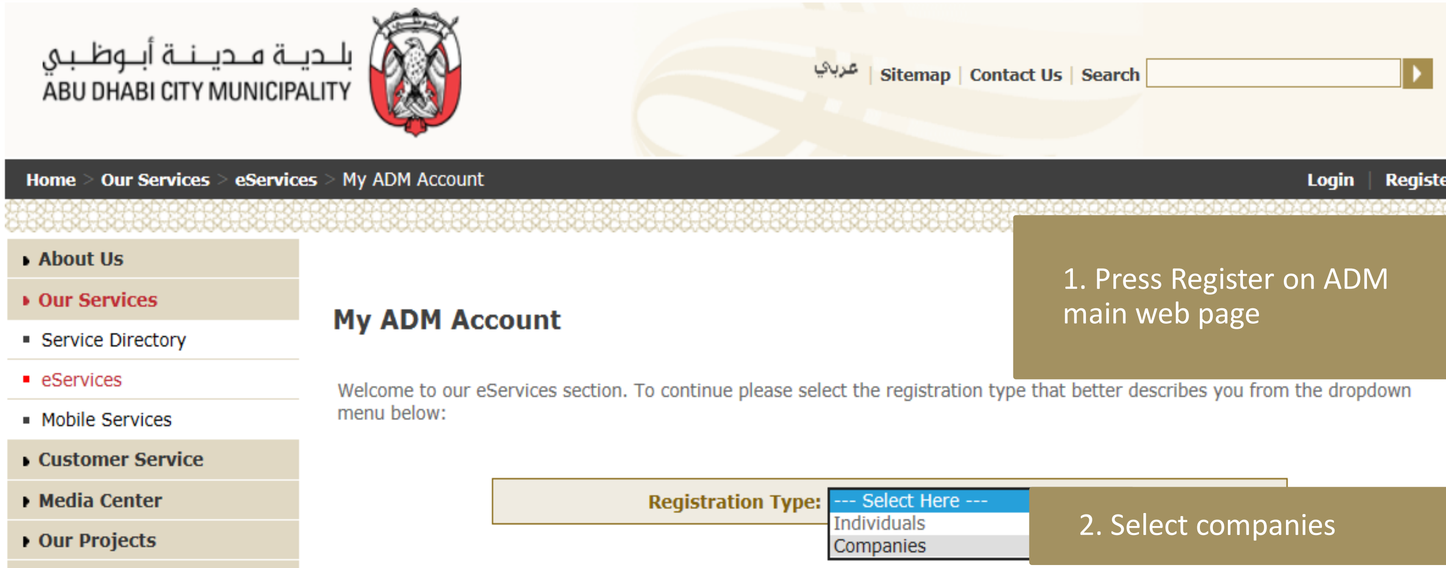


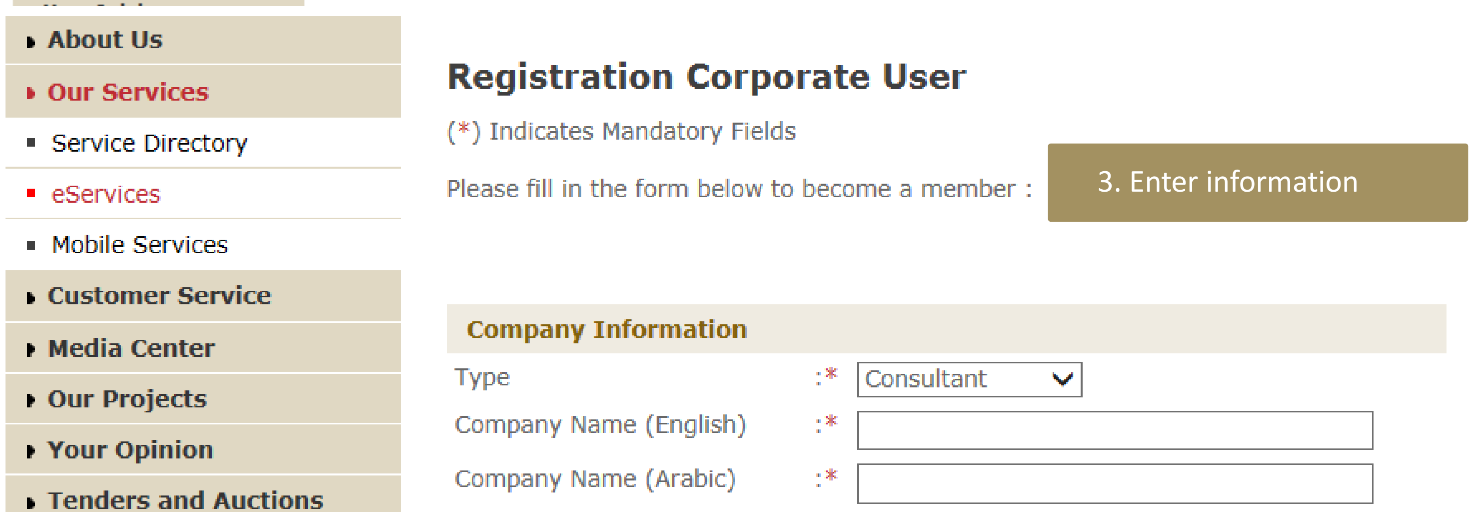
Registration on CDP system – 1st step

ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS

Two-Step Process

1. Register in the eServices (you select your own username and password)





Registration on CDP system – 2nd step

2. Having registered in eServices, you can register in the CDP (username and password is provided by ADM). You will receive the username and password to the email provided during registration.

The eServices of Construction Permits

To use the eServices of Construction Permits you need to obtain an approva Construction Permits Division in The Municipality of Abu Dhabi City.

- To obtain an approval click here to apply.
(You will receive a username and password onc rove

- If you have already received your username a onst Division click here .

5. Press click here to apply

- About Us
- Our Services
- Customer Service
- Media Center
- Our Projects
- Your Opinion
- Tenders and Auction
- Documents Center
- Careers
- Contact Us

Construction & Infrastructure Permits

Our Vision

4. From ADM main web page, hit Construction & Infrastructure Permits

eService User Login

To use our eServices, please "Login" into your eServices Account or Create an Account if you are a new member.

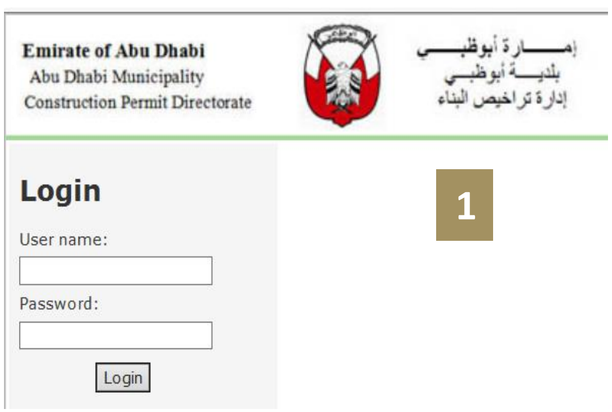
Customer Type:* Company
Username:*
Password:*
Forgot your password?
Login

6. Log-in with the username and password from Step 1 and continue registration process

General guidelines for online submittal

1. Apply online through the CDP system (*https://cdp001.dma.ae/cdpabudhabi/cdppublicportal/loginpage.aspx*).
2. Use “New Engineering Permit” for geotechnical permitting of infrastructure projects.
3. Use “Add New Permit” for building projects. **Request New Project only if one does not exist already for another type of permit.**
4. Consultant should apply for geotechnical permitting of building projects.
5. Geotechnical laboratories can apply for geotechnical permitting of infrastructure projects; or for geotechnical permitting of building projects (if a project was not created already for another type of permit).

2

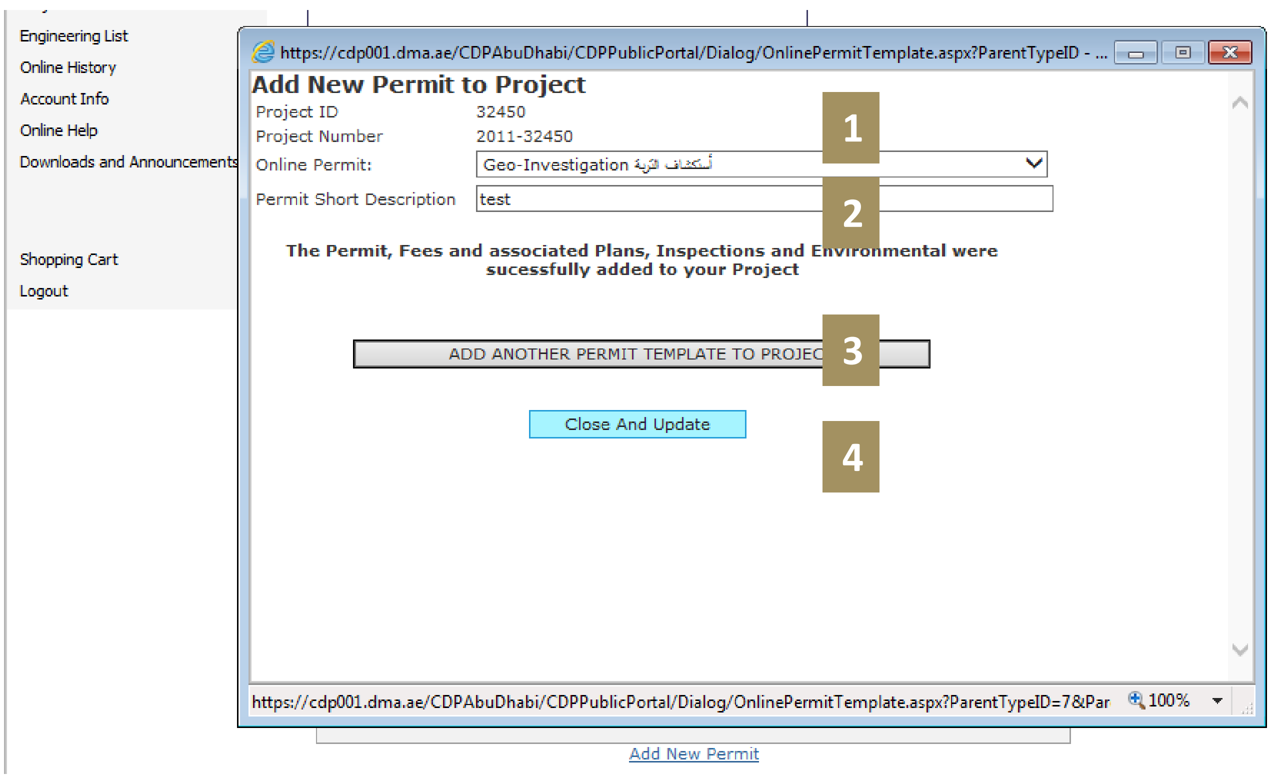


1

3

Online application steps for permitting geotechnical work of building projects

دائرة الشؤون البلدية
بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
DEPARTMENT OF MUNICIPAL AFFAIRS



1 Select Geo-Investigation.

2 Write short description.

3 Press add permit

4 Press close and update

Online application steps for permitting geotechnical work of building projects

Project | Permits | Fees | Inspections | Plans

5

Project : 32450

Permit #	Permit Category	Permit Type	Short Desc	Status	Issue Date	Complete Date
43440	4-Modify w/out ext. add تعديلات دون إضافة خارجية	Modificatio without addition تعديلات دون إضافة	تعديلات حسب المخططات المعتمدة	Issued	20 Sep 2011	12 May 2011
102523	7-Others category فئة المشاريع الأخرى	Geo-Investigati استكشاف التربة	test	Applied For		06 Oct 2013

745302	Geotech Report	6	07 Oct 2013	09 Oct 2013
745303	HSE-Risk Assess	7	07 Oct 2013	07 Oct 2013

5 Press “Plans”

6 Press Geotech Plan No. to upload (a pop-up window will appear).

7 Press HSE-Risk Assess Plan No. to upload (a pop-up window will appear).

Online application steps for permitting geotechnical work of infrastructure projects

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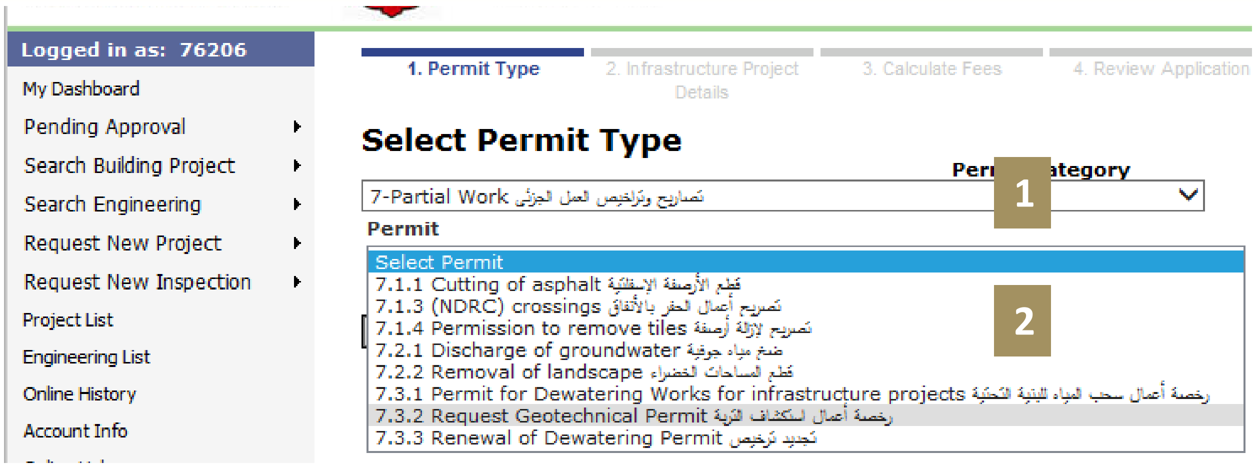
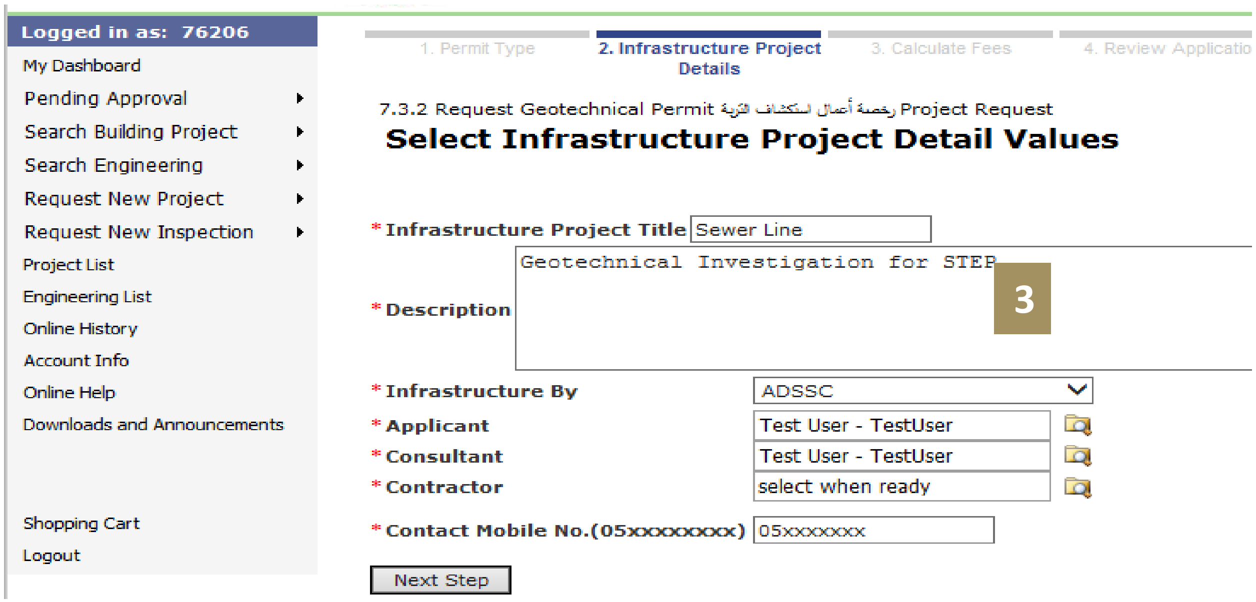


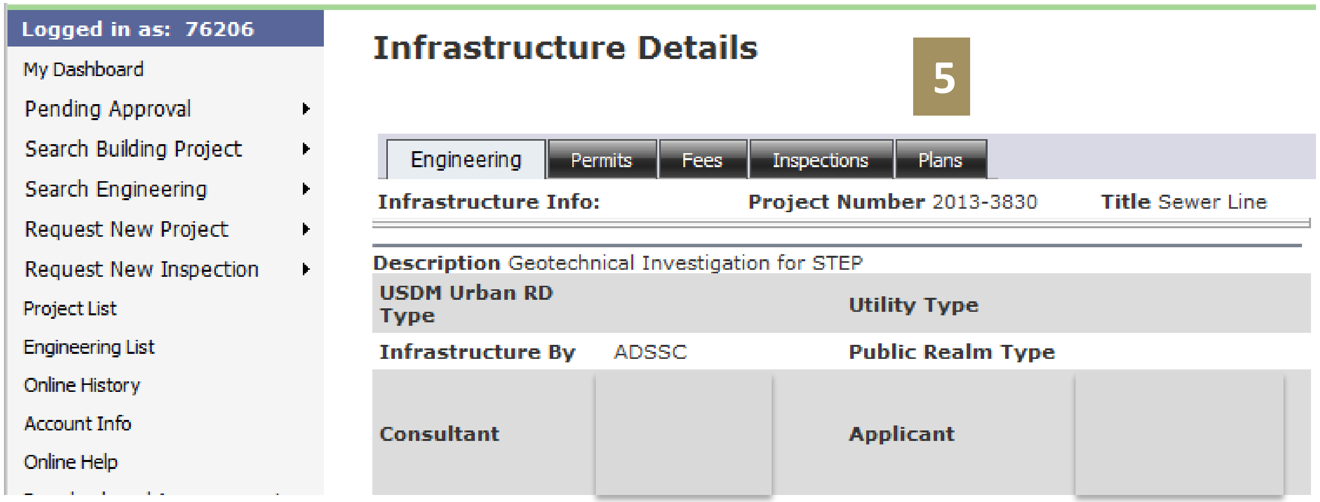


1 Select 7-Partial Work

2 Select 7.3.2-Request Geotech. Permit, Press “Next”

3 Enter Project Details, Press “Next Step”

4 Press “Submit”



5 Press “Plans”

6 Press Geotech Plan to upload (a pop-up window will appear).

7 Press HSE-Risk Assess to upload (a pop-up window will appear).

745302	Geotech Report		6	07 Oct 2013	09 Oct 2013
745303	HSE-Risk Assess		7	07 Oct 2013	07 Oct 2013

Required documents

1. Signed Application Form (available on ADM website)
2. Authorization letter from owner assigning the geotechnical laboratory
3. Valid commercial license of geotechnical laboratory
4. Accreditation certificate acceptable to Abu Dhabi Quality and Conformity Council, ADQCC (mandatory after 31^{st} December, 2014)
5. Scope of work approved by Consultant (concise summary)
6. Equalization certificates (mandatory after 30 April, 2014) of assigned staff (for field supervisor) and brief curriculum vitae (CV).

Upload in Geotechnical Plan on CDP

7. HSE Risk Assessment Matrix

Upload in the HSE Risk Ass. Plan on CDP

Typical documents content

Application form lists the required documents. It is available online from Abu Dhabi City Municipality website (adm.gove.ae) in the following sequence:

Documents Center/Documents/ Town Planning /Construction Permit/ Geotechnical Unit

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Application Form No. CDP-003 For Approval of Geotechnical Investigation

نموذج رقم CDP-003 لطلب موافقة على أعمال استكشاف التربة

S/N رقم	Item البند	Applicant Input إفادة مقدم الطلب			CPD Review مراجعة الإدارة
1.0	Project Information:			معلومات المشروع:	
1.1	Work Location موقع العمل (لمشاريع البنية التحتية)	Area		المنطقة	
		Zone(s)		الحوض أو الأحواض	
		Plot		القسيمة	
	مرحلة المشروع أو المنشأة		Master Plan	المخطط العام	
			Conceptual/Prelim. Design	التصميم المبدئي	
			Final Design	التصميم النهائي	
			Construction	التنفيذ	
			Operation	التشغيل	

S/N رقم	Document المستند	Upload Location مكان التحميل	Building Project مباني	Infrastructure Project بنية تحتية	Applicant's Compliance Check تحقق مقدم الطلب	CPD Review تدقيق الإدارة
2.0	Submittal Requirements الوثائق المطلوبة					
2.1	Form CDP-003 نموذج CDP-003		✓	✓		
2.2	Authorization Letter تكليف من المالك		✓	✓		
2.3	Valid Trade License رخصة تجارية سارية المفعول		✓	✓		
2.4	Accreditation certificate approved by QCC (required from Jan 1, 2015) اعتماد مقبول من مجلس أبوظبي للجودة والمطابقة (ضروري اعتباراً من 1 يناير 2015)		✓	✓		
2.5	Scope of work approved by Consultant نطاق الأعمال والفحوصات الحقلية والمخبرية والمعتمد من الإستشاري		✓	✓		
	Equalization Certificates (required starting May 1st, 2014) of assigned staff (for both field supervisor and	Geotech. الهندسة الجيوتكنيكية				

Typical documents content

Field Work (Attach Plot Plan)
- Boreholes: x No. to xx m and x No. to xx m.
- Test Pits: x No. to x m.
- Cone Penetration Test : x No. to xx m.
- Standpipes: x No. to xx m
- (*list all field tests*)

Laboratory Tests
- Sieve Analyses: x No.
- Atterberg Limits: x No.
- pH, Sulfates, Chlorides on Soil: x No.
- Chemical (*list*) tests on water: x No.
- Unconfined Compressive Strength on Rock: x No.

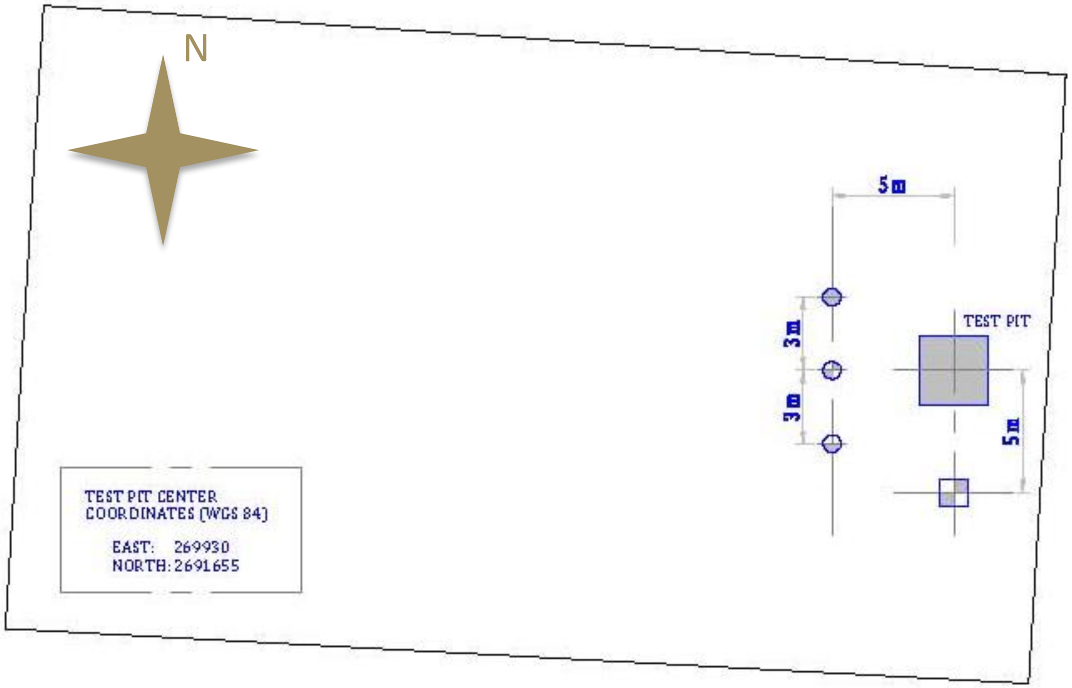
Name: __________
Education: (*Degree and date obtained*)
Experience:
- Current company: *Date*-present
- Previous companies:
  - (*Name of Company and country*): *Position and dates*
  - *(List all previous companies)*

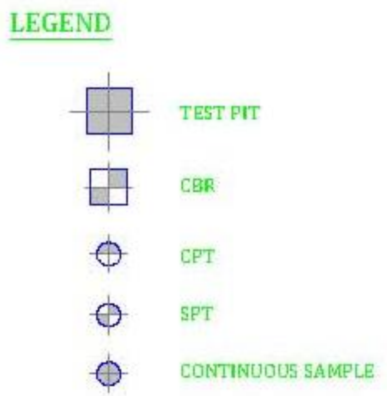
Scope of Work (approved by consultant)

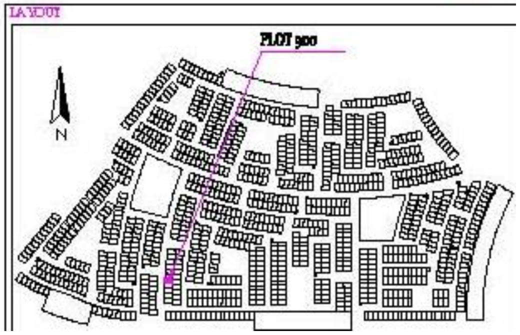
Curriculum Vitae

Typical documents content

Typical plot plan showing location of test points







Thank You

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DEPARTMENT OF MUNICIPAL AFFAIRS

Application Form No. CDP-003 For Approval of Geotechnical Investigation

نموذج رقم CDP-003 لطلب موافقة على أعمال استكشاف التربة

S/N رقم	Item البند	Applicant Input إفادة مقدم الطلب			CPD Review تدقيق الإدارة
1.0	Project Information:			معلومات المشروع:	
1.1	Work Location موقع العمل (لمشاريع البنية التحتية)	Area		المنطقة	
		Zone(s)		الحوض أو الأحواض	
		Plot		القسيمة	
1.2	Phase of project or facility مرحلة المشروع أو المنشأة		Master Plan	المخطط العام	
			Conceptual/Prelim. Design	التصميم المبدئي	
			Final Design	التصميم النهائي	
			Construction	التنفيذ	
			Operation	التشغيل	

S/N رقم	Document المستند	Upload Location مكان التحميل	Building Project مباني	Infrastructure Project بنية تحتية	Applicant's Compliance Check تحقق مقدم الطلب	CPD Review تدقيق الإدارة
2.0	Submittal Requirements الوثائق المطلوبة					
2.1	Form CDP-003 نموذج CDP-003	Geotech. الهندسة الجيوتكنيكية	✓	✓		
2.2	Authorization Letter from Owner تكليف من المالك		✓	✓		
2.3	Valid Trade License رخصة تجارية سارية المفعول		✓	✓		
2.4	Accreditation certificate for the geotechnical laboratory approved by QCC (mandatory after 31st December, 2014) اعتماد لمختبر فحص التربة مقبول من مجلس أبوظبي للجودة والمطابقة (إلزامي بعد 31 ديسمبر 2014)		✓	✓		
2.5	Scope of work approved by Consultant نطاق الأعمال والفحوصات الحقلية والمخبرية والمعتمد من الاستشاري		✓	✓		
2.6	Equalization Certificates (mandatory after 30 April, 2014) of assigned staff (field supervisor) and brief Curriculum Vitae. معادلة الدرجات العلمية (إلزامية بعد 30 أبريل 2014) والسيرة الذاتية للمهندس المشرف.		✓	✓		
2.7	EHS Risk Assessment Matrix جدول تقييم المخاطر	EHS إدارة البيئة والصحة والسلامة	✓	✓		
✓	Required			CPD	Construction Permit Division	

Application Form No. CDP-003 For Approval of Geotechnical Investigation

نموذج رقم CDP-003 لطلب موافقة على أعمال استكشاف التربة

Declaration		إقـــــــــــــــــــرار	
I hereby declare that the information contained in this form and the attached documents, plans and drawings is true and correct in all material particulars and that we will fully comply with the approval conditions of Department of Municipal Affairs including obtaining all required approvals and No Objection Certificates from concerned entities before starting of any works.		وبهذا أصرح بأن المعلومات الواردة في هذا النموذج والمستندات المرفقة والمخططات والرسومات حقيقية وصحيحة في جميع التفاصيل، وأننا سوف تمتثل امتثالا تاما لشروط الموافقة الصادرة عن دائرة الشؤون البلدية بما فيها الحصول على جميع الموافقات وشهادات عدم الممانعة المطلوبة من الجهات المعنية قبل القيام بالأعمال.	
Signature of the Consultant: توقيع الاستشاري:		Official Stamp الختم الرسمي للاستشاري	
Date: التاريخ:			
Signature of the Contractor : توقيع المقاول:		Official Stamp الختم الرسمي للمقاول	
Date: التاريخ:			
Signature of the Geotechnical Laboratory: توقيع مختبر التربة:		Official Stamp الختم الرسمي لمختبر التربة	
Date: التاريخ :			

Service Card

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Service Basic Information

Service Name	Issue Geotechnical Investigation Permit			Service Ref#	--------------
Sector	Town Planning	Division	Construction Permit	Section	Geotechnical Engineering
Service Owner's Name		Dr. Mazen Adib		Position	Advisor – Town Planning Sector
Service Owner's Email address		Mazen.adib@adm.abudhabi.ae		Phone	+97126957713
Service Cycle Time	6 days	Fees	TBD	Related Decree	Decree No. 182/2012 - DMA regarding Geotechnical Permit

ADM Service Classifications

Main Service Categories (Level1)	Construction Permits	Business Service Area (level2)	Construction Permit for Temporary Works
Service Strategic Categories	-	Service User-group Categories	Consultants Contractors Geotechnical Testing Laboratories
Service Outcome Categories	Municipal Permissions	Service Delivery Categories	-

Service Description and Usage Cases

1. Objective

The objective of this service is to regulate the geotechnical investigations in Abu Dhabi to ensure the compliance of the investigation with relevant regulations and to improve the quality of the geotechnical investigations in Abu Dhabi by ensuring consistent and effective oversight by the Municipality personnel during design and execution stages of the investigations.

2. Scope

Any geotechnical investigation with the intent of producing a "geotechnical report" whether factual or interpretative in nature must obtain a permit to perform the investigation even if field work is not required such as desktop studies.

3. Usage Cases

The permit applies to all geotechnical investigations in Abu Dhabi including but not limited to:

a. Residential, commercial and industrial developments
b. Infrastructure and Facility projects
c. Any project or study requiring geotechnical investigations in the City of Abu Dhabi.

4. Relevant References:

1 Building Law, 1983
2 DOT Manual for Geotechnical Investigation Part 1 & 2
3 Guidelines for Geotech Invest. ADM, Road Dept, 2003.
4 Geotechnical Data Submittal Standards, SDD 2012
5 Form CDP-003, 2012
6 Unified Bldg Process No. 4 for Temporary Works, 2012
7 IBC 2009 Section 18

Documents required for the provision of service

S/N	Documents required (soft copies)		
1	Application Form CDP-003	5	Scope of Geotechnical Investigation and laboratory testing program approved by the Project Consultant and Time Schedule for the Works
2	Commercial License of Geotechnical Testing Agency (field and laboratory)	6	Health and Safety Risk Assessment Matrix
3	List of equipment and personnel and quality certificated issued by the concerned entities	7	No Objection from the concerned entities for Infrastructure Projects: DOT, IRI, PRFD, UPD, ADWEA, ADSSC, NCEMA, etc.
4	A copy of the academic and practical qualifications of the engineer supervising the geotechnical investigations and a copy of the equivalency certificates issued by Ministry of Higher Education.	8	Valid Site Plan

Business Rules

Service Card

بلدية مدينة أبوظبي
ABU DHABI CITY MUNICIPALITY
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S/N	Item	Source - Code / Manual & Reference Section	Control / Requirements
1	Required attachments are complete	Decree issued by DMA	All attachment must be complete including the application form, duly signed and stamped.
2	Geotechnical Testing Agency (GTA) has a valid commercial certificate	Decree issued by DMA Chairman	Company must be registered in Abu Dhabi and has a valid commercial license.
3	GTA has the proper quality certificates for the field and laboratory tests in the	Decree issued by DMA Chairman	Company must have a valid quality certificate for the tests it intends to carry out preferably from Abu Dhabi Quality and Conformity Council هيئة الإمارات للمواصفات والمقاييس
4	Qualification of Supervising Engineer	1. Building Law Section 12 Item 2 page 98. 2. DOT Manual for Geotech. Invest. Part 1, Appendix A, Table A1	Supervising Engineer must have the knowledge and the experience to supervise the required investigations as per the applicable guidelines of Building Law and DOT Manual Part 1.
5	Scope of Investigation	1. Building Law (Section 12, item 3 and 4 and 5 and 6) 2. DOT Manual for Geotechnical Investigation Part 2 Section 4.3 3. Guidelines for Geotechnical Investigation, Abu Dhabi Municipality Road Department	For each project category, a minimum requirement must be included in the scope of work either from the Building Permit Law (1983), or IBC 2009 Section 18, or DOT Manual for Geotechnical Investigation Part 1 and 2 or ADM Guidelines for Geotechnical Investigations or from an appropriate published manual on geotechnical investigations. Category of geotechnical projects is explained in DOT Manual for Geotech Investigation Part 1 Section 3.3
6	No Objection Certificates	Guidelines under preparation by Executive Council An NOC is required from entities whose assets will be impacted or whose assets are within a few meters from the boreholes.	Pending completion of guidelines by Executive Council. Coordinator needs to specify based on locations and type of projects and assets being impacted (roads, parks, utilities, etc.). Testing agency is required to identify the NOC requirement for confirmation by ADM

Note
1. The above control / requirements are based on current business practice and approved policies and guidelines and they may be subject to future review.
2. The relevant sections of the prevailing codes / manuals and guidelines apply as the standard control. Exceptional cases which do not meet the standards will be considered on the merits of the case.

Dependent Service Names	
1	Demarcation of Site Boundary for Geotechnical Information Purpose
2	Issue Site Plan (for Building Projects)
3	Provisional Approval of Infrastructure Projects by Urban Planning Division

Service Inbound Channels	Service Outbound Channels
Online Application on the CDP	Permit is issued by Customer Service Counters
✓	✓

End-To-End Service Diagram

1 Day	3 Day	1 Day	1 Day
Receive online request and check submittal completeness	Review compliance with regulations	Geotechnical Section approval	Issue permit (by Customer Service) after all NOCs are obtained