Large-Scale Land Reclamation and Soil Improvement for a City Expansion

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LARGE-SCALE LAND RECLAMATION AND SOIL IMPROVEMENT FOR A CITY EXPANSION

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ABSTRACT

This paper describes the geotechnical experience gained over the last 25 years at a 21-square kilometers waterfront development along the east coast of Doha city, Qatar. The development process involved filling a shallow bay using approximately 53 million cubic meters of calcareous sand with gravel and limestone fragments, making it one of the largest land reclamation projects in the world. The 2 to 3-meter-high filling was placed on natural seabed deposits that typically consist of a 1.0 m to 1.25 m top layer of soft plastic silt followed by an approximately 3.5 meters of loose to medium sand and an extended layer of weathered limestone. The subsurface layers that posed engineering problems such as excessive settlement to constructions in the filled area were primarily the soft plastic silt and loose sand. Field and laboratory tests conducted before and after filling showed a significant soil improvement due to fill loading and the consequent soil aging. This is attributed to primary and secondary consolidation of plastic silt and secondary compression of sand. Settlement analysis considering this improvement has led to the use of shallow foundations for low-rise and relatively high-rise structures changing the general practice of overusing the costly and time-consuming deep foundations and soil replacement.

INTRODUCTION

In 1975, a major development has started to expand Doha, capital of Qatar, by filling a shallow bay along the city coast. With the exception of some low-rising outcrops in the north and north west of the country, Qatar has a flat rocky (limestone) terrain covered with sand flats, dunes, and sabkha. Located about midway along the country eastern cost, Doha is mainly covered with calcareous sand and limestone fragments (Fig. 1). This surface material was used in the filling process to overly the natural seabed deposits of the bay area.

Filling the shallow bay was completed in 1980. However, considerable development of the filled area has been started about 13 years later due to economical constrains. Development has involved construction of roadways, utilities, and low and high-rise buildings. Extensive total and differential settlement of such structures due to compressibility of natural seabed deposits and irregularity of the underlain limestone layer has been a concern. As a result, deep foundations and soil replacement have been overused to support structures in the filled area. However, comprehensive subsurface investigations including field and laboratory tests showed that loads imposed by filling precompressed and significantly improved the seabed deposits to allow for construction of most of these types of structures on shallow foundations with tolerable settlement behavior. The following

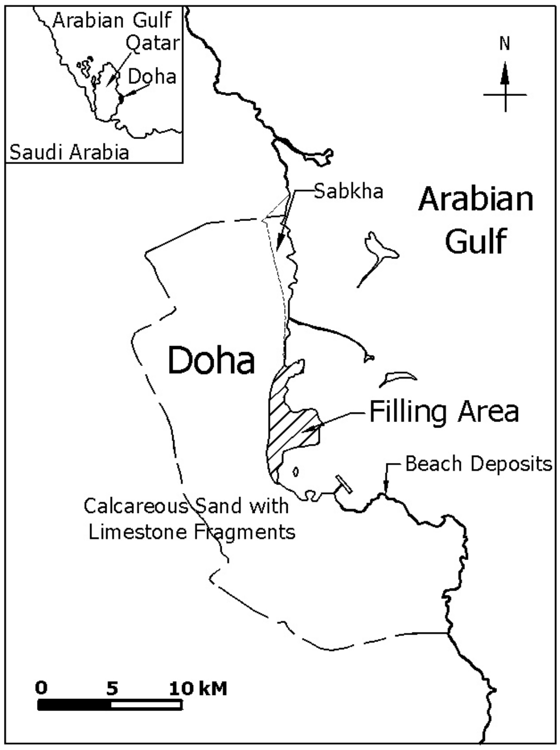

Fig. 1. Site location plane and surface soil description.

sections describe this large-scale land reclamation project, geotechnical testing findings, and settlement prediction techniques and results that has been helping in avoiding the overuse of deep foundations and soil replacement and consequently accelerating the development process.

PROJECT DESCRIPTION

The project involved filling 21-square kilometers of a shallow bay developing a waterfront-reclaimed land for expansion of Doha city. The bay had a water depth ranging between 0.5 m and 1.0 m creating readily inaccessible area due to the compressible nature of the surface soil. Reclamation took place by end tipping from the landward side with no formal compaction procedure. The filling material consisted of calcareous sand with gravel and limestone fragments. The procedure adopted during land reclamation was to form an offshore bund by placing fill along the proposed new shoreline (Fig. 2). The resulting lagoon, behind the bund, was then filled with same material. The reclaimed land was then left almost undeveloped until 1993.

As a part of the country capital, the filled area is to accommodate high-rise condominiums and administration buildings, 2 to 3-story villas and townhouses, roadways, and infrastructures. The high and low-rise structures are all reinforced concrete constructions. Only one-level underground garage typically accompanies each high-rise structure to avoid the costly and time consuming dewatering process.

Typically about 2 to 3-meter-high filling was required to raise the site to design grades. More fill was required in building areas that set at 0.5 to 1.0 meters above street grades. Consequently, about 53 million cubic meters of fill material was placed to form the reclaimed area.

SUBSURFACE CONDITIONS

Different phases of subsurface explorations have been conducted before and after the filling completion. These explorations included test borings and open pits located across the project area. Both undisturbed tube and core samples and disturbed split-spoon samples with standard penetration tests (SPT) were taken. The exploration showed that the filling material is underlain by natural seabed deposits that typically consists of a 1.0 to 1.25 meter-layer of plastic silt followed by an approximately 3.5 meters of sand. The deposits are in turn underlain by an extended layer of weathered limestone that is occasionally intruded by 1.0 to 2.0 meter-thick layers of weak attapulgitic shale at depths ranging between 24 and 30 meters below the fill surface. Typical depths and thickness of the subsurface materials are given in Table 1.

Because of the close proximity of the site to the sea, the depth of ground water is affected by seasonal fluctuation of sea level. The fluctuation range is approximately 0.5 meter. Boreholes carried out at different times of the year reflected this fluctuation. As a result, ground water was encountered at depths varying from 2 to 2.5 meters below the fill surface depending on time of boring and thickness of fill layer.

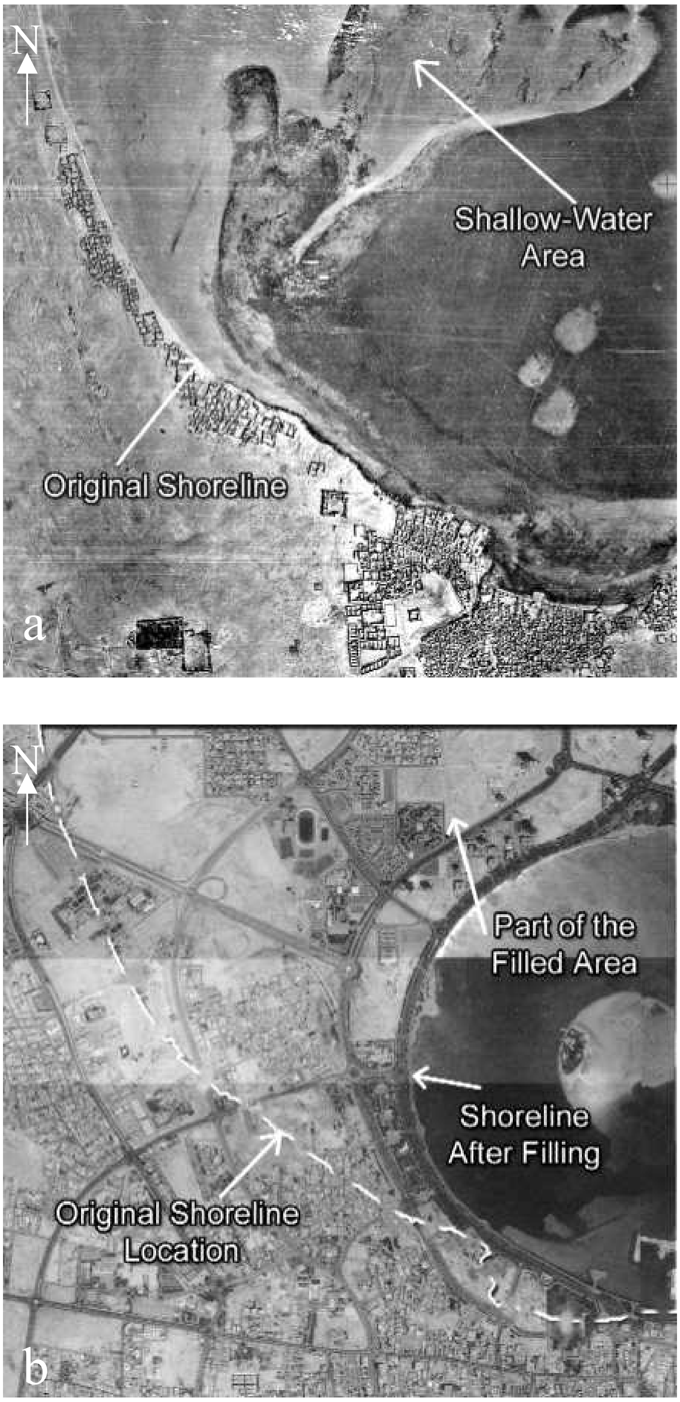

Fig. 2. Aerial photographs showing part of the site, (a) before filling, and (b) after filling.

Particle-size analyses conducted on the fill material showed that it consists mainly of poorly graded sand and gravel with cobble-size fragments of limestone and traces of silt. Standard penetration test indicated that the fill layer is of medium density with an average 'N' value of 22. This layer varies in thickness from 2 to 3 meters.

Table 1. Generalized Subsurface Conditions After Filling

Type of Material	Typical Depth to Top of Layer (m)	Typical Thickness (m)	Typical N-Value[a]
Fill	0	2 - 3	15 - 30
Plastic Silt	2 - 3	1 - 1.25	7 - 9
Sand	3.25 - 4	3 - 4	17 - 35
Limestone	6.5 - 8	Variable	NA[b]

a N-Value: Standard penetration resistance in blows/foot.
b NA: Not applicable

Figure 3 shows particle gradation of the natural seabed deposits. It can be seen that the deposits consists of an upper layer of clayey silt to sandy silt followed by a sand to gravelly sand layer. Consistency and relative density and consequently the mechanical properties of the upper and lower layer soils, respectively, changed due to the fill loading as described in the subsequent section.

The thickness of silt layer varies across the project area from a maximum of 1.25 meters to complete absence. However, the typical thickness is more than 1.0 meter. Results of Atterberg limit tests conducted on the fine-grained fractions of this layer soil indicated that the material is plastic with average liquid limit and plastic limit of 39% and 21%, respectively. The natural marine deposit that was encountered underneath the silt layer comprises a poorly to well-graded fine to coarse carbonate sand to gravelly sand. In some of the project area, the material has become cemented to form a calcarenite layer known locally as 'Caprock'. The cementation is however very irregular and the boreholes would not suggest a laterally continuous layer.

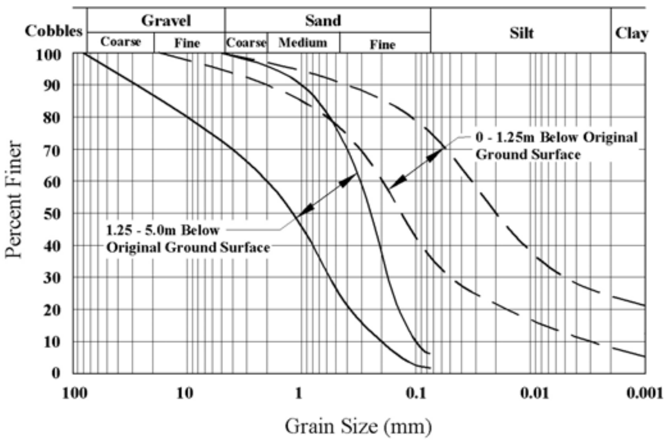

Fig. 3. Grain size distribution for natural seabed deposits

An extended layer of weathered limestone underlies the sand layer. The limestone layer belongs to the Simsima member of upper Dammam subformation. A general description of the Simsima limestone, as presented by Cavalier et al. (1970), would be a fine to medium grained off-white to pale brown, poorly bedded, chalky crystalline calcareous and dolomitic limestone with numerous irregular joints often filled with weathered siltstone. Thin layers of pale green or red brown attapulgitic clays are occasionally present.

Figure 4 shows the change of rock quality designation (RQD) of limestone with depth. It can be seen that the average RQD ranged between 25% and 55%. According to the classification system presented by Deere (1963), quality of the limestone can be described as very poor to fair. Figure 4 also shows unconfined compressive strength (q_u) values of limestone samples extracted from different depths. These values were determined using uniaxial compression tests and generally lead to describing the limestone as moderately weak to moderately hard.

It is clear that limestone of the upper 5 meters is harder and has less values of RQD than that at larger depths. This can be related to the limestone bimodal nature comprising hard-recrystallised predominantly calcareous dolomitic limestone and a variable percentage of secondary material. The engineering properties, including the strength, are largely influenced by the percentage and type of the secondary material. Limestone of the upper 5 meters comprises silt as a secondary material that makes it harder and more breakable than that at larger depths which comprises attapulgitic clay.

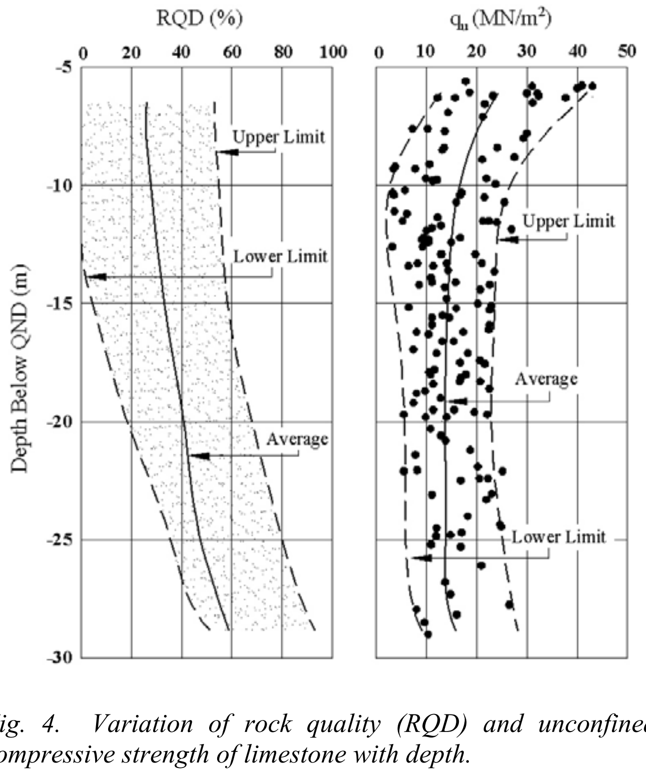

Fig. 4. Variation of rock quality (RQD) and unconfined compressive strength of limestone with depth.

This difference is also reflected on the measured Young's modulus (E_{lab}) values that were recorded while conducting the uniaxial compression tests (Fig. 5). Values of E_{lab} generally ranged between 0.7 and 5.2 GN/m^2, with an average value of about 2.25 GN/m^2. For settlement calculations, upper, lower, and average trend lines of Young's modulus values for limestone mass (E_{mass}) were estimated using correlation presented by Bieniawski (1978) utilizing the RQD average trend line and measured values of E_{lab} shown in Fig. 4 and 5, respectively. Because of the low measured RQD, values of the estimated E_{mass} are approximately tenth of E_{lab} values at the same depth with an average E_{mass} of about 0.28 GN/m^2 (Fig. 5).

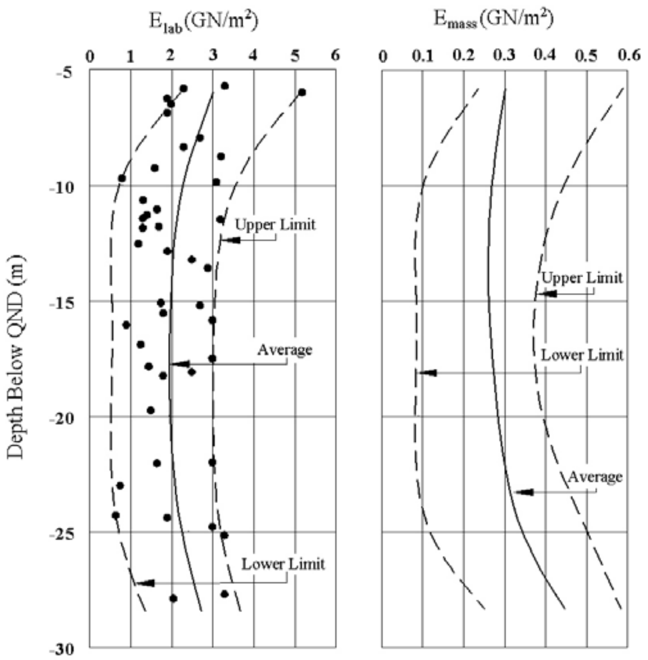

Fig. 5. Young's modulus measured for intact limestone specimens (E_{lab}) and estimated for limestone mass (E_{mass}) at different depths.

Cavities in Simsima limestone were not encountered in the drilled boreholes and no loss of air/water circulation was reported through the drilled depth for any borehole. However, as any relatively soluble bedrock with water accessibility, cavities and cutter-and-pinnacle karst landforms are expected (White, 1988). The potential of these landforms is enhanced because some cavities may have migrated up to Simsima limestone from the underlying dissoluble anhydrite/gypsum beds of Rus Formation.

SOIL IMPROVEMENT

Field and laboratory tests conducted before and after filling showed a significant improvement in the natural seabed deposits due to fill loading and aging. The time-dependent increase in stiffness and strength following loading was thoroughly described in the literature. The increase has been attributed to primary and secondary consolidation for clays (Leonards and Ramiah, 1960; Bjerrum, 1972; Daramola, 1980, Schmertmann 1991; Terzaghi et al., 1996), and secondary compression for sands (Mitchell and Solymar, 1984; Mesri et al., 1990; Schmertmann, 1991). The literature also showed that magnitude of time-dependent improvement, excluding that due to primary consolidation, has a typical range of 50% to 100% usually presented as an increase in measured standard and cone penetration resistance. Most of this increase occurs in the first several months and years for sands and clays, respectively. Unfortunately, the timely nature of soil improvement was not monitored in the project described in this paper because of the 13-year delay in developing the project area. As a result, only the original and almost the fully improved properties of the seabed deposits are presented and compared subsequently.

Figure 6 shows a typical relation between end-of-primary void ratio and logarithm of effective stress for two samples taken from the middle of the plastic silt layer at its original condition and 23 years after filling completion. As an improvement indicator due to fill loading, the silt has developed a relatively significant preconsolidation pressure of 63 kPa that is even more than the overburden pressure after filling which is approximately 45 kPa (Fig. 6). As a result, the silt can be described as slightly overconsolidated. This overconsolidation is attributed to secondary consolidation or aging. The preconsolidation pressure was estimated using the graphical construction proposed by Casagrande (1936). It should be noted that the effective overburden pressure at the middle of the plastic silt layer was approximately 5 kPa before placement of the fill.

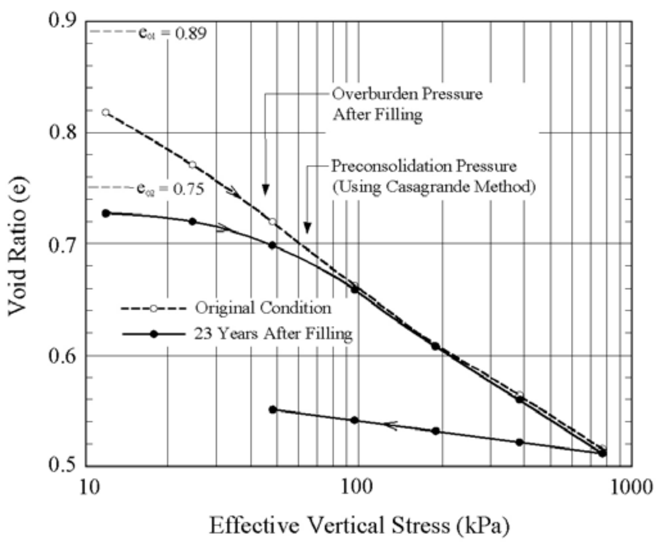

Fig.6. Effect of filling on consolidation curves of silt samples.

For the improved silt represented in Fig. 6, values of recompression index (C_r) and the rebound curve slope (C_s) are found to be approximately the same and equal to 0.023. The compression index (Cc) is also estimated to be 0.185. The coefficient of consolidation (C_v) in the recompression and compression range were evaluated to be 129 m^2/year and 17 m^2/year, respectively, using the logarithm of time fitting method described by Casagrande and Fadum (1940).

Silt improvement through experiencing primary and secondary consolidation is also reflected on the measured SPT N-values. Figure 7 shows that average measured N-value for plastic silt increased from 3 to 8. This changed the consistency of silt from soft to stiff. The increase in measured N-values was also noticed for the underlying sand layer and was more pronounced in the top 1.5 meter of sand changing its relative density from loose to medium (Fig. 7). Sand improvement can be attributed to drained aging or secondary compression due to fill loading. According to Mesri et al. (1990), increase in sand penetration resistance due to secondary compression is related to an enhancement of macrointerlocking of sand grains and microinterlocking of grain surface roughness.

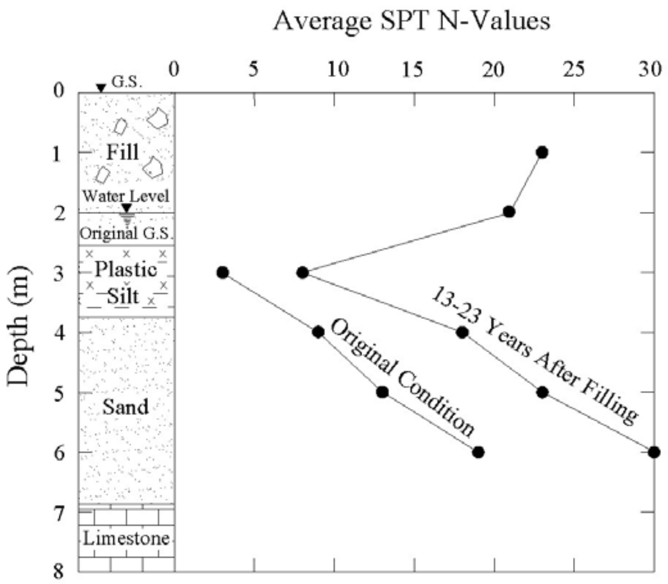

Fig. 7. Effect of fill loading on measured N-values for different soil layers.

SETTLEMENT PREDICTION

Previously being a shallow bay surfaced with a highly compressible natural seabed deposits, the filled area has experienced a general practice of overusing deep foundations to avoid excessive settlement. However, considering soil improvement presented in the previous section, the following analysis shows that shallow foundations can be used in supporting low and relatively high-rise structures with tolerable total and differential settlement values. The analysis deals with the worst possible subsurface layering conditions that yield maximum total and differential settlements for low and high-rise structures typically designed for development of the filled area (Fig. 8).

Both the consolidation and elastic (immediate) soil compression were considered in the settlement analysis. For consolidation compression, the recompression and compression indices of plastic silt were utilized in calculating settlement due to stress increase between effective overburden and preconsolidation pressures, and preconsolidation and final pressures, respectively. Because of the silt high coefficient of consolidation, small layer thickness, and double drainage faces, most of the consolidation compression has occurred by the construction completion.

Method presented by D'Appolonia et al. (1970) was used in calculating elastic compression for finite layers of granular soils. Modulus of elasticity for these soils was also estimated using D'Appolonia et al. (1970) correlation to the average SPT N-value. Since almost all of its deformation would be vertical, elastic compression of limestone was simply estimated by multiplying strain times the height of limestone involved in settlement. The strain was calculated by dividing the applied stress by the limestone mass modulus of elasticity.

For a 3-story townhouse as a representative of low-rise structures, isolated footings have been used as foundation. Columns are typically arranged in 4×5 meter-pattern and supported by 2-meter square footings at a depth of 1.0 meter using an allowable fill bearing capacity of 150 kPa. As shown in Fig. 8, the maximum total and differential settlement can be developed if the silt layer underlies in full thickness the fill beneath a footing and totally vanished beneath an adjacent footing. Calculations showed that the maximum total settlement, differential settlement, and angular distortion would be approximately 1.68 cm, 1.13 cm, and 1 to 354, respectively. These values are tolerable considering settlement limits suggested by MacDonald and Skempton (1955) for isolated footings. It should be noted that the settlement values would have exceeded the limits if improvement in plastic silt due to fill loading was not considered.

For high-rise structures, settlement of a typical structure on 30×50 meters raft foundation resting at depth of 2.5 meters and imposing a net pressure of 100 kPa was predicted. Thickness of layers beneath raft contributing to settlement was estimated to be 13 meters using method presented by Burland and Burbidge (1985). As shown in Fig 8, the worst possible subsurface layering conditions that yield maximum total and differential settlement was set for the high-rise structure by assuming the existence of full-thickness silt layer along with a soil filled cutter beneath one side of the raft width, and absence of them beneath the opposite side. Average E_{mass} and N-value of 0.27 GN/m^2 and 24 blows (Fig. 5 and 7) were used in elastic settlement calculations for involved limestone and improved sand layers, respectively. The calculations yielded a predicted maximum settlement and angular distortion of 6.45 cm and 1 to 563, respectively. These values are also tolerable considering limits presented by MacDonald and Skempton (1955) for raft foundation. This means that for such worst subsurface layering conditions considered in the analysis, a raft foundation can support a 16-floor structure with one-story basement without excessive settlement. Better subsurface conditions should allow for higher structures to be supported by raft foundations.

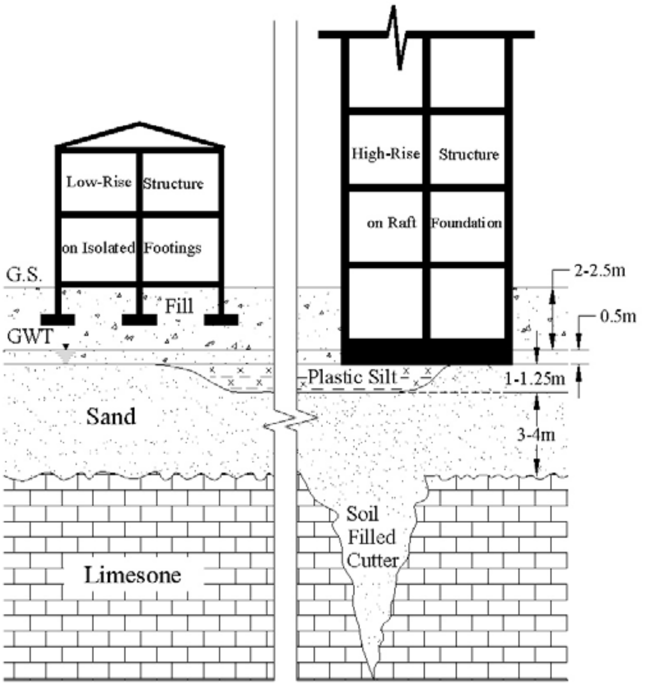

Fig. 8. Schematic drawing showing the worst possible subsurface layering conditions for low and high-rise structures.

Different sizes of structures have been erected on the filled area in the last 23 years (Fig. 9). These structures represent less than 20% of the construction activities targeted to develop the filled area. As an expansion of a growing country capital, the filled area is to be fully developed by the end of next decade. Introducing the settlement analysis described above to geotechnical and structure-engineering practitioners has started to help in avoiding the overuse of deep foundations and soil replacement and consequently accelerating the development of the filled area.

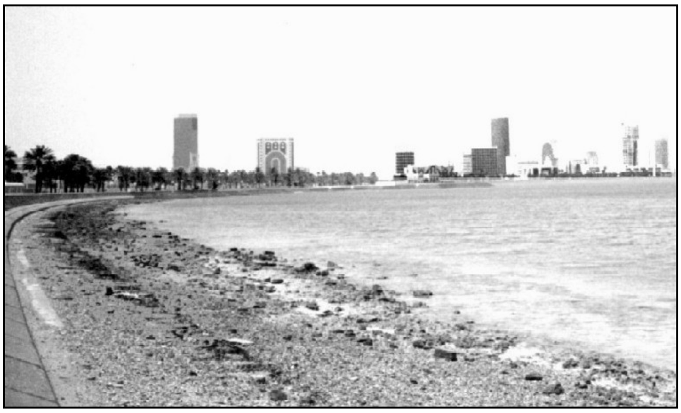

Fig. 9. Different sizes of structures have been erected on the filled area for development of the country capital.

CONCLUSION

A 21-square kilometer waterfront was developed along the east coast of Doha city, Qatar, by filling a shallow bay using approximately 53 million cubic meters of sand with gravel and limestone fragments, making it one of the largest land reclamation projects in the world. The depth of seawater at the bay area ranged between 0.5 and 1.0 meter. Filling had continued for five years starting 1975 with no formal compaction procedures leading to an average 2.5-meter thick layer of medium dense fill. The fill layer was placed on natural seabed deposits of about 1.25 meter of soft plastic silt followed by an average of 3.5 meters of loose to medium sand and an extended layer of limestone. The reclaimed land was left almost undeveloped until 1993. Subsurface investigation conducted before filling and after development started showed a significant improvement in the seabed deposits. The measured average SPT N-values increased from 3 to 8 for the plastic silt layer and from 8 to 18 for the loose sand layer. The preconsolidation pressure of the silt layer also increased from 5 kN/m^2 to 63 kN/m^2. These increases are attributed to primary and secondary consolidation of plastic silt and secondary compression of loose sand. Settlement calculations considering these improvements have yielded tolerable predicted values of maximum settlement, differential settlement, and angular distortion for typical low-rise and relatively high-rise structures resting on isolated footings and raft foundations, respectively, considering the worst possible subsurface layering conditions. This settlement analysis results have been used to avoid the overuse of deep foundations and soil replacement and consequently accelerate the development of the reclaimed area.

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