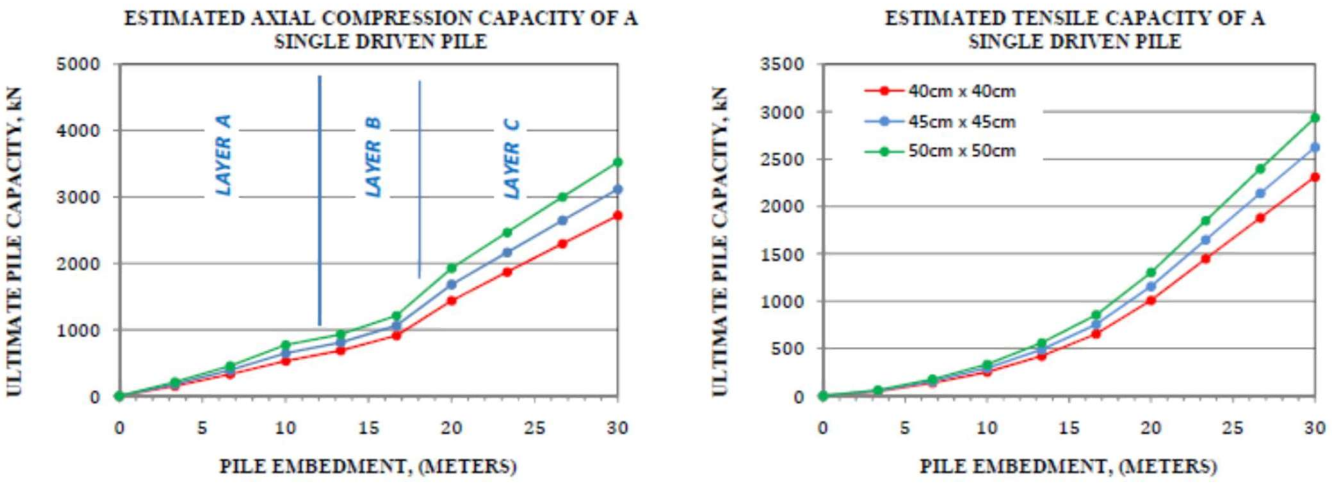
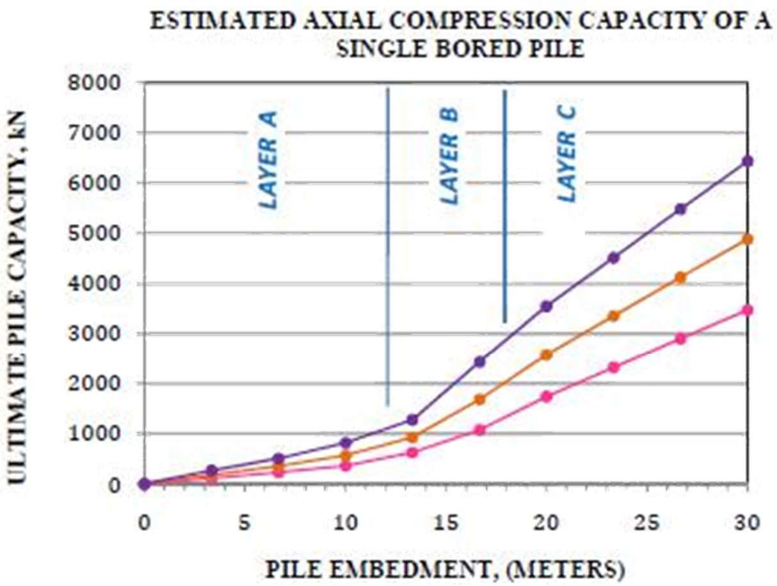


Figure 7-2. Estimated ultimate capacity of single-stick driven piles



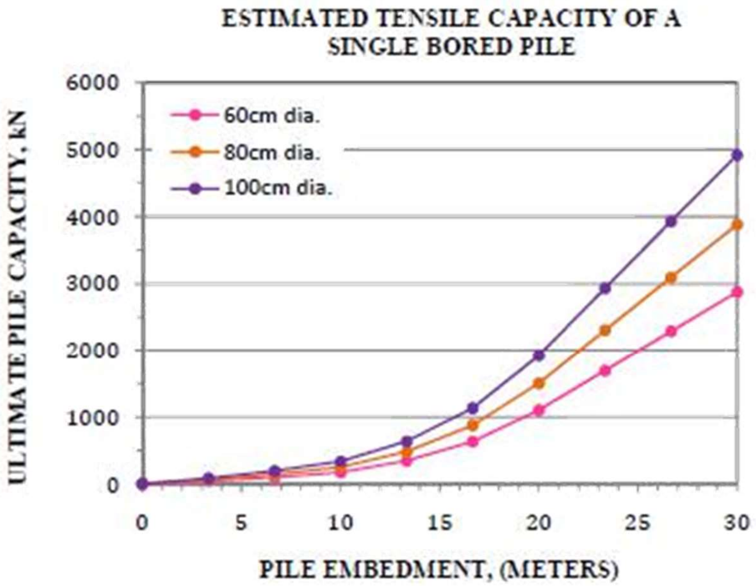


Figure 7-2. Estimated ultimate capacity of a single bored pile

7.5.1.1.Other Pile Design Considerations

Verification of Actual Pile Capacity& Integrity

The tabulated pile capacities are based purely on theoretical computations. The actual capacity of the piles will have to be confirmed / determined by actual pile load tests - either by the Static Test (ASTM D1143) or the Dynamic (ASTM D4945) Testing Procedures. The latter will be the more practical choice as more piles can be tested at a much lesser time and cost.

Foundation Quality Control during Construction

Quality control of piles may be best checked using appropriate testing methods such as Pile Integrity Testing (ASTM D 5882) &Cross-hole Logging Tests (ASTM D 6760) for integrity testing, and High-strain dynamic testing (ASTM D 4945) for capacity verification.

Pile Driving

Pile driving should be done continuously since relatively long stoppages would make re-driving difficult. A wave equation analysis (GRLWEAP) may have to be conducted to verify size of hammers suitable for driving to the prescribed or desired depth, and check driving stresses as well.

Pile Spacing

To minimize stress overlapping, piles should be spaced as far as practicable. A minimum spacing of 2.5 to 3.0D from center to center of piles may be adopted, where D is the diameter of the pile.

Efficiency of Pile Group

Since friction is the major component of the pile capacity, it is recommended that the efficiency of pile groups be calculated using the Converse-Labarre equation calculated as follows:

$$E_g = 1 - \theta \frac{(n-1)m + (m-1)n}{90mn}$$

Where:

n = number of rows

m = number of columns

$\theta = \tan^{-1} \frac{D}{s}$

D = diameter

s = spacing

7.5.2. Shallow Foundations

The use of shallow foundation should be limited to non-essential and light structures, as there is a serious risk associated with liquefaction (described in Section 7.2) and/or ground subsidence.

For the above purpose, footings may be founded on 1.0 – 1.2m below the existing ground level, bypassing any unconsolidated deposits. A conservative net allowable bearing capacity of 50 kPa (1,000 psf) may be assumed in proportioning the footings.

Isolated footings or foundations should be connected with tie-beams to provide structural rigidity and help resist differential settlement especially during strong earthquake.

7.5.3. Coefficient of Lateral Subgrade Modulus, k_h

As a guide, the ranges of k_h that may be used indesigning the piled foundation to resist lateral loads are as follows:

Table 7-2. Coefficient of Lateral Subgrade Modulus, k_h

Layer	General Description	Estimated Lateral Subgrade Modulus (MN/m^3)
A	Very loose to loose Sand (Ave=8)	4 – 6
B	Soft to stiff Clay (Ave=12)	15 - 20
C	Medium dense gravel/sand-sized Limestone fragments with clay (Ave=24)	30 – 40

7.6. Retaining Wall Design

It is apparent that the envisioned 6m depth of excavation will have to proceed with bracing, soil nailing and or tiebacks for temporary support, as the soil is generally weak and is prone to sliding failure.

The design of the wall will have to consider the combined effect of the lateral earth pressure and the surcharge load (if any), the water pressure, and earthquake loads.

In estimating for the lateral earth pressure acting on the retaining wall, an effective friction angle phi of 28 degrees and a saturated unit weight of $15kN/m^3$ may be assumed. Full hydrostatic height of water will also have to be conservatively assumed, unless appropriate fail-proof drainage system can be assured.

7.7. Ground Subsidence

Ground subsidence in the surrounding area is anticipated due to the presence of high water table and loose sand in the uppermost 12m depth, coupled with the anticipated pumping out of the ground water. It is prudent to provide monitoring of ground and wall movements during construction to evaluate its effect and help minimize the damage to the surrounding structures.

Mitigation would involve soil improvement of the surrounding soils prior to excavation and perhaps provision of longer sheet piles. Seepage forces will have to be considered in the analysis of the retaining wall design

7.8. Seismicity

Based on the seismic provisions of the National Structural Code of the Philippines (NSCP2010, Sixth Edition) seismic zone factor is 0.4 for the project area, and the prevailing soil type (alluvial deposits) falls under S_D. The near-source factors Na & Nv are 1.0.

8. LIMITATIONS

This geotechnical report was prepared to aid in the design of this specific project. Its scope is limited to the project and location described herein and represents our understanding of the surface and subsurface conditions at the site, at the time of the investigation.

Should there be appreciable differences found in the soil/rock conditions during the construction phase, or should there be any differences in our understanding of the project requirements, we should be immediately notified so that supplemental recommendations can be provided.

RICHARD C. TAN, MSCE
Principal Geotechnical Engineer
PRC Reg. No. 23248
October 12, 2015
Quezon City, Philippines

TITLE: **BOREHOLE LOCATION PLAN**				
PROJECT: PROPOSED SEWAGE TREATMENT PLANT FOR ZAMBOANGA WATER DISTRICT		LOCATION: MAGAY STREET, ZAMBOANGA CITY		
CLIENT: AECOM International Development, Inc.		DATE: OCTOBER 05, 2015	SCALE: not to scale	SHEET NO.: 1 of 1