For better appreciation, a plot showing the recorded uncorrected N-values vs. Depth is shown below.

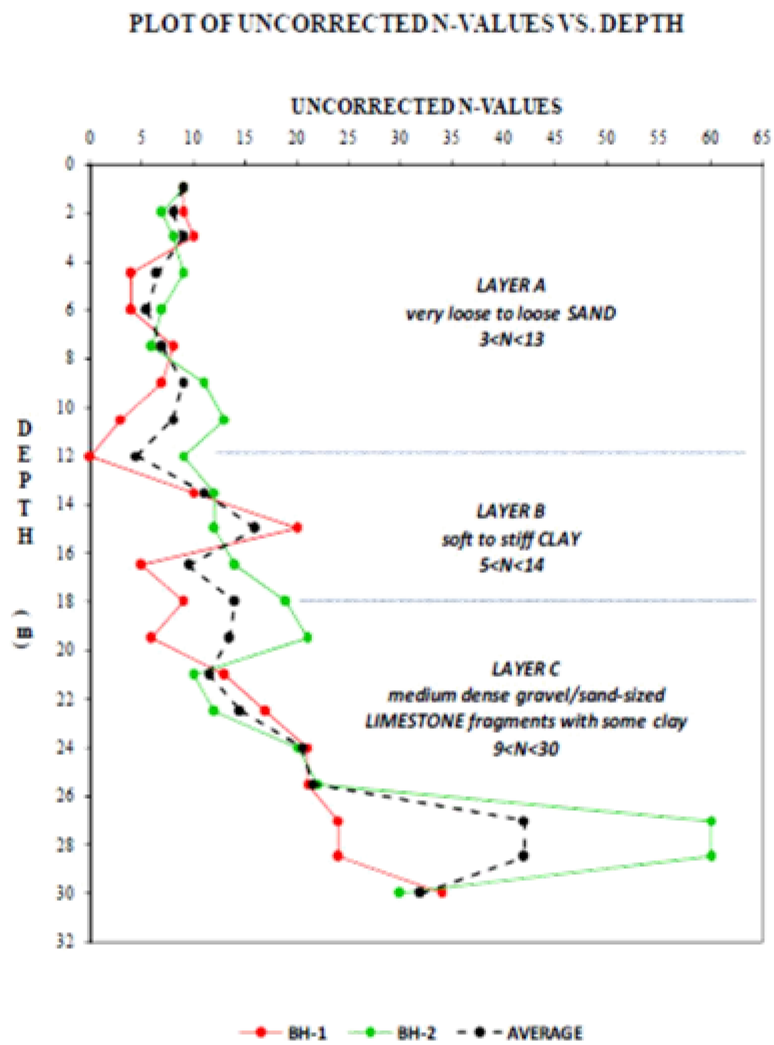

Figure 6-1. Plot of uncorrected N-values vs. depth

6.1. GROUND WATER TABLE MEASUREMENT

The groundwater table was reported in the logs at depth ranging from 0.5m – 1.5m from the existing ground surface. The information is presented below for easy reference:

Table 6-1. Groundwater Table Logs

DATE MEASURED	BH-1	BH-2
9/18/15	1.50m (5PM)	-
9/19/15	0.47m (7AM)	-
	0.47m (5PM)	-
9/20/15	1.5m (7AM)	0.97m (5PM)
9/21/15	-	1.2m (7AM)
	-	1.2m (5PM)

6.2. CHEMICAL TESTS

Representative soil samples were also tested for the determination of the chloride, sulphate and organic content. The results of the following tests are shown below:

Table 6-2. Summary of Chemical Test Results

BH NO.	DEPTH (M)	RESULT OF CHEMICAL TESTS		
		CHLORIDE (mg/kg)	SULPHATE (mg/kg)	ORGANIC CONTENT (%)
BH-1	9.0-9.45	34	3500	-
	16.5-16.95	130	68	-
	19.5-19.95	-	-	3.5
BH-2	2.0-2.45	-	-	3.9
	4.5-4.95	80	1500	-
	24.0-24.45	170	73	-

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. General Findings

Based on the results of the investigation, it is concluded that the project site is underlain by relatively thick loose soil sediments, consisting mainly of an uppermost 12.0m of mostly Sand, and very loose to loose in consistency; followed by the soft to stiff clay that extends to about 18.0m depth. The final layer encountered is described as medium dense gravel/sand-sized Limestone fragments with some clay extending down to the bottom of the borehole at 30.45 meters depth.

The following soil parameters may be assumed based on the results of the soil borings.

AVE. DEPTH (meter)	SOIL DESCRIPTION	MEAN N-VALUE	SATURATED UNIT WEIGHT, γ_s (kN/m^3)	EFFECTIVE COHESION, C' (kPa)	EFFECTIVE PHI ANGLE, ϕ' (degree)
0 – 12.0	Very loose to loose Sand	8	15.5	0	28
12.0 – 18.0	Soft to stiff Clay	12	16.0	10	30
18.0 – 30.0	Medium dense Limestone fragments with some clay	24	17.0	20	36

Table 7-1. Recommended Soil Parameters

From the above findings, it is apparent that the uppermost 12m thick of soil is compressible and has low bearing capacity, as may be inferred from the low SPT N values (Ave=8). This layer is also potentially liquefiable in the event of strong ground motion (major earthquake).

The results of the liquefaction analysis are presented in the next section.

7.2. Liquefaction

The presence of very loose to loose Sand found in the uppermost 12mdepths would indicate that the project site is susceptible to liquefaction phenomenon. Liquefaction refers to the significant loss of strength and/or stiffness due to cyclic pore pressure generation which is generally exhibited by sands and non-plastic silts.

Liquefaction analysis was conducted using the empirical method of Seed and Idriss (1971), and assuming ground acceleration of 0.25g. Cyclic stress ratio (CSR) and cyclic resistance ratio (CRR) were computed followed by the factor of safety against liquefaction, by dividing CRR by CSR.

The results of the liquefaction study are graphically presented in the next page. It shows the liquefaction potential along the depth of the study (CRR and CSR), where the red shaded areas represent potential liquefiable zones. The factor of safety against liquefaction and the degree of settlement are also plotted with respect to the soil depth. The corresponding soil profile is then shown in the next page.

The settlement was estimated to be about 24cm based on the results of BH-1& BH-2 using the procedure developed by Ishihara and Yosemine (1990).

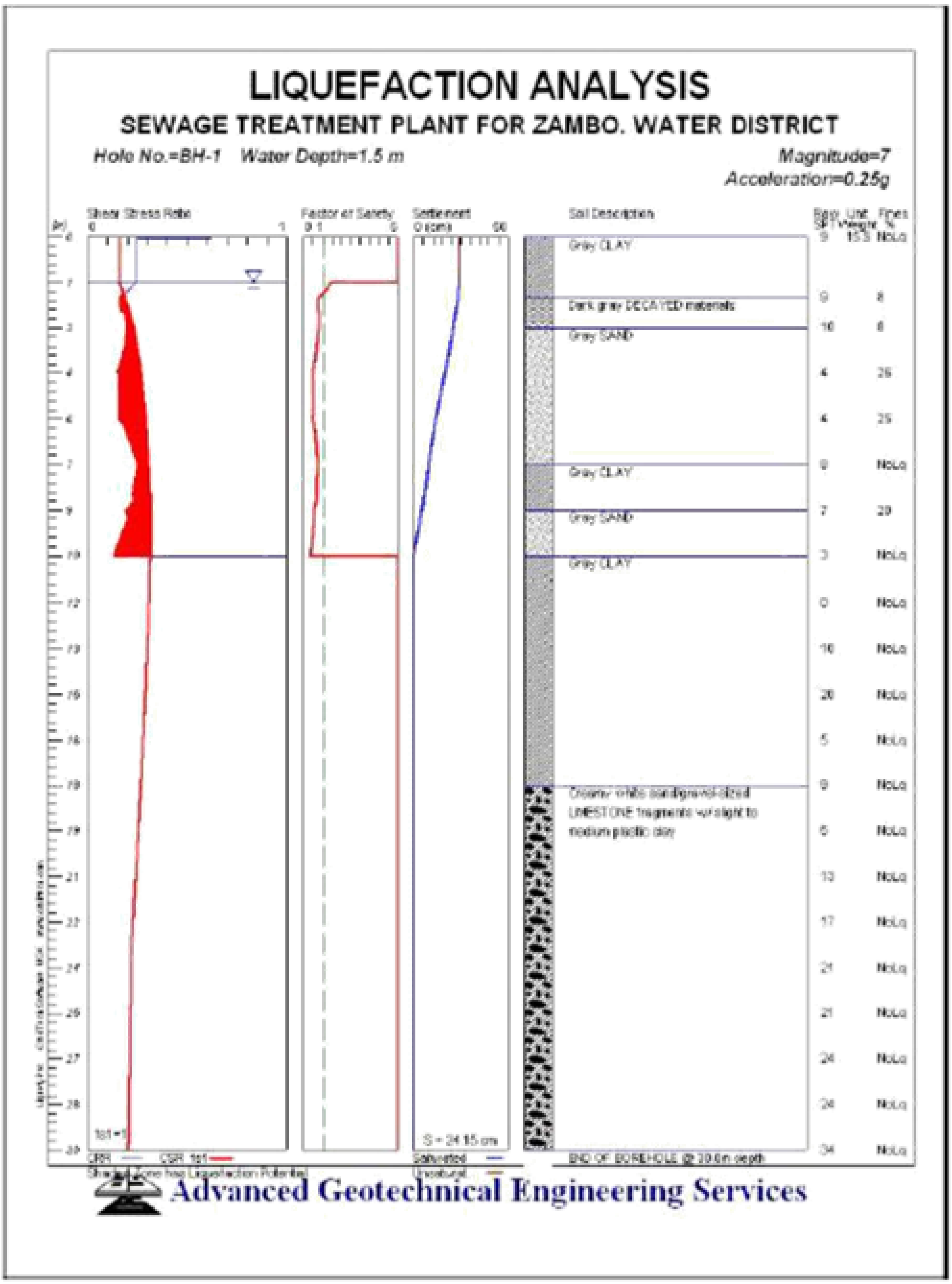

Figure 7-1. Liquefaction Analysis of BH-1

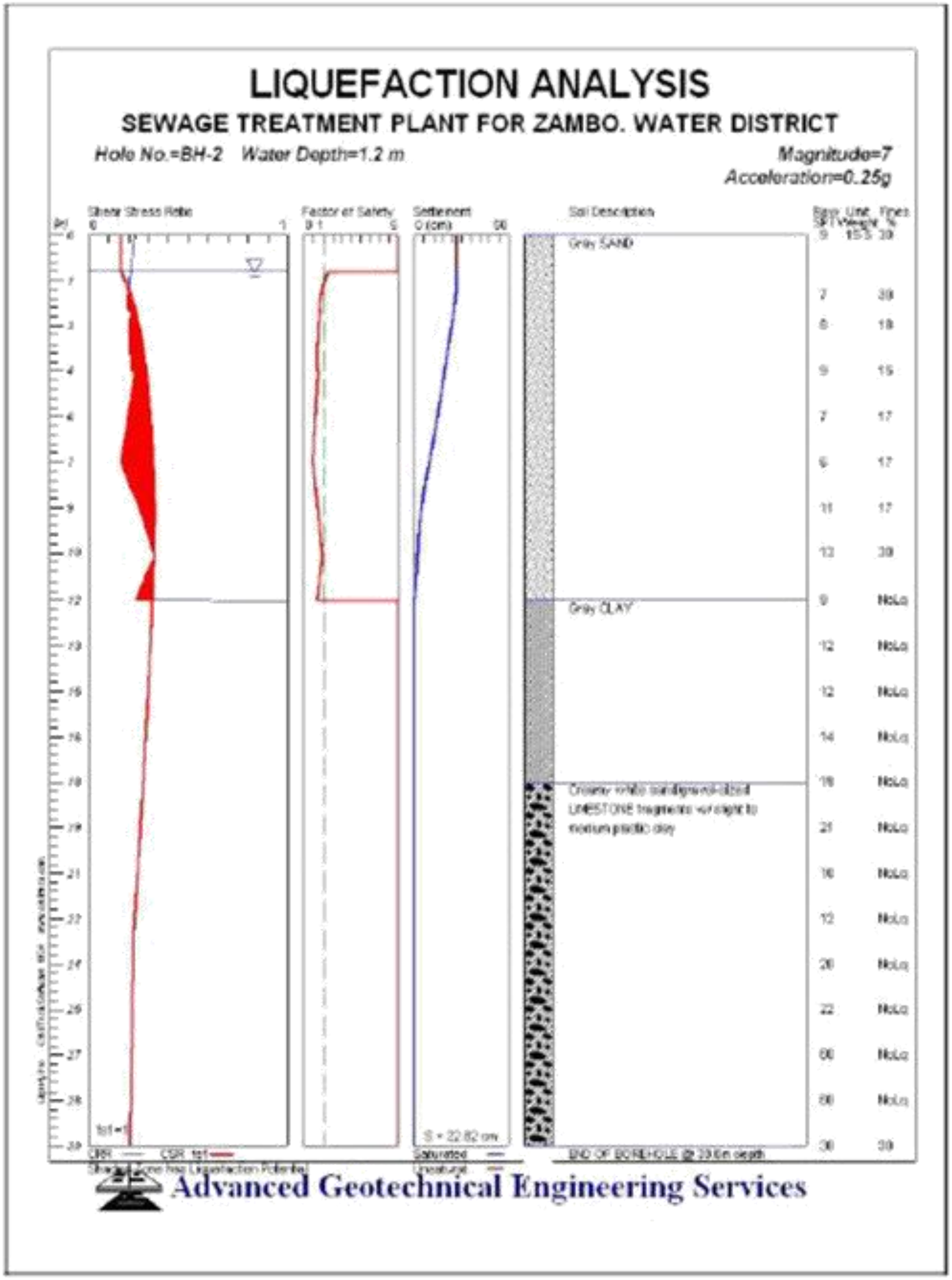

Figure 7-2. Liquefaction Analysis of BH-2

7.3. Lateral Spreading

Lateral spreading is described as the lateral movement of the soil on a gentle sloping ground due to soil liquefaction. Based on the liquefiable potential of the upper soil layer of the project site and nearby shoreline, lateral spreading may occur in an event of a strong earthquake. Documented events are likely on mild slopes of 0.3 to 5%. Horizontal displacement and vertical displacement (settlement and heaving) due to lateral spreading has caused considerable damage to infrastructures and especially underground / utility lines.

In-situ ground solidification technique, such as the deep cement mixing (DCM), is known for mitigation of earthquake-induced lateral spreading. DCM is installed by inserting columns of soil-concrete mixture in the project site. The installation creates a grid of soil-cement columns that produces a stiffer strength to support in-situ soil to reduce lateral spreading. The popularity of this method is indisputable in Japan, followed by the United States and Scandinavia.

7.4. Geotechnical Concerns

The main geotechnical concern of a buried tank below high water table (about a meter from the existing ground) will be the uplift or buoyant force. The uplift force will be exacerbated should liquefaction occur (arising from the excess high pore water pressure) during a strong ground motion (Major earthquake).

A second concern would be the stability of the envisioned vertical cut. Retaining structure is necessary to contain the envisioned 6m vertical cut, and will have to be designed for a) active earth pressure, b) the full hydrostatic forces assuming water level at the ground surface, c) earthquake forces, d) effects of surcharge loads arising from nearby structures or roadways.

Another concern would be the ground subsidence in the surrounding area during excavation works, especially when dewatering (water table is lowered by pumping). Ground subsidence is reasonably expected as the retaining wall moves (or rotates) towards the excavation for the active earth pressure (minimum lateral resistance) to act. Dewatering works in the area will exacerbate ground subsidence unless soil improvement is first provided.

7.5. Foundation Schemes

Based on the above findings, the proposed structure(s) may be supported on deep or piled foundations. In addition to the required compressional load, the piles will have to provide tensile resistance to resist the buoyant forces of the buried tank (i.e., when empty).

Driven piles or bored piles may be considered for this purpose.

Shallow foundation scheme may only be considered for non-essential and low-rise structures, or when soil improvement has been undertaken to arrest possible effects of liquefaction phenomenon and ground subsidence.

Both schemes are discussed in the following sections.

7.5.1. Piled Foundations

Based on the above discussion, the use of single-stick, prestressed, reinforced, precast concrete piles are recommended for the project. The single-stick is underlined for emphasis in anticipation of the required pullout resistance, as jointed piles may have questionable pullout resistance.

Considering the built-up surroundings (1-2 storey residential / commercial buildings), the use of static pile driver is recommended to eliminate noise and air pollutions and unwanted vibrations that may affect the operations of the surrounding residences / businesses / buildings.

Piles will have to bear directly on the medium dense layer (Layer C), with minimum depth of embedment at 24m reckoned from the existing ground surface.

In the event that the use of driven piles might not be plausible, bored piles maybe considered.

The advantage of using bored piles is that it can be drilled and socketed into the more dense material, thus offering higher shaft resistance.

Pile capacity estimates are graphically presented in the next page.

A suitable safety factors, typically 2.5 for compression and 2.75 for tension, may be applied to the calculated ultimate pile capacity to arrive at the allowable pile capacity.

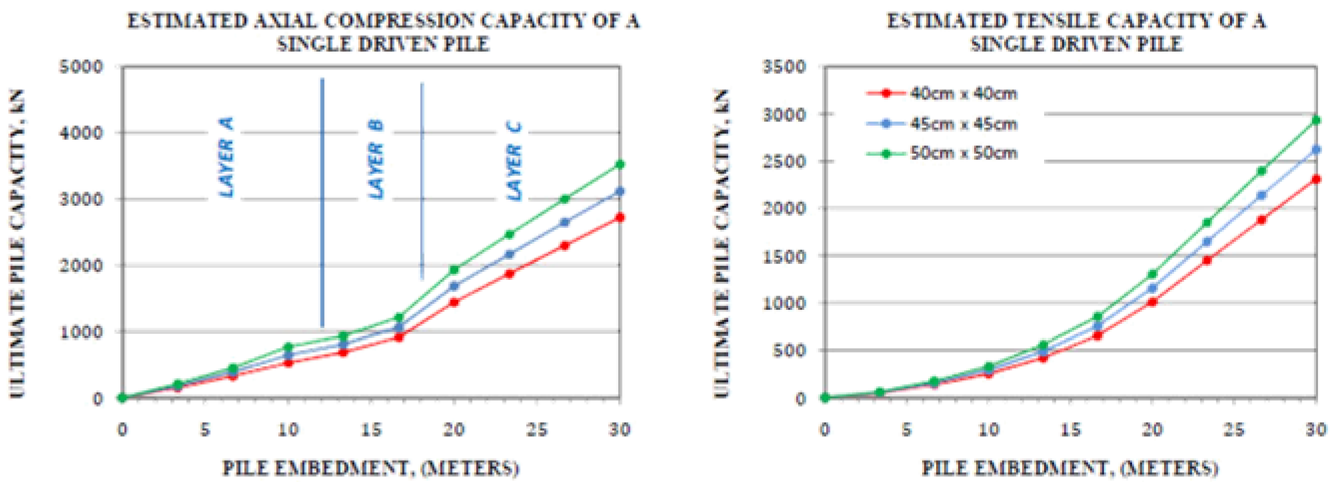

Figure 7-2. Estimated ultimate capacity of single-stick driven piles

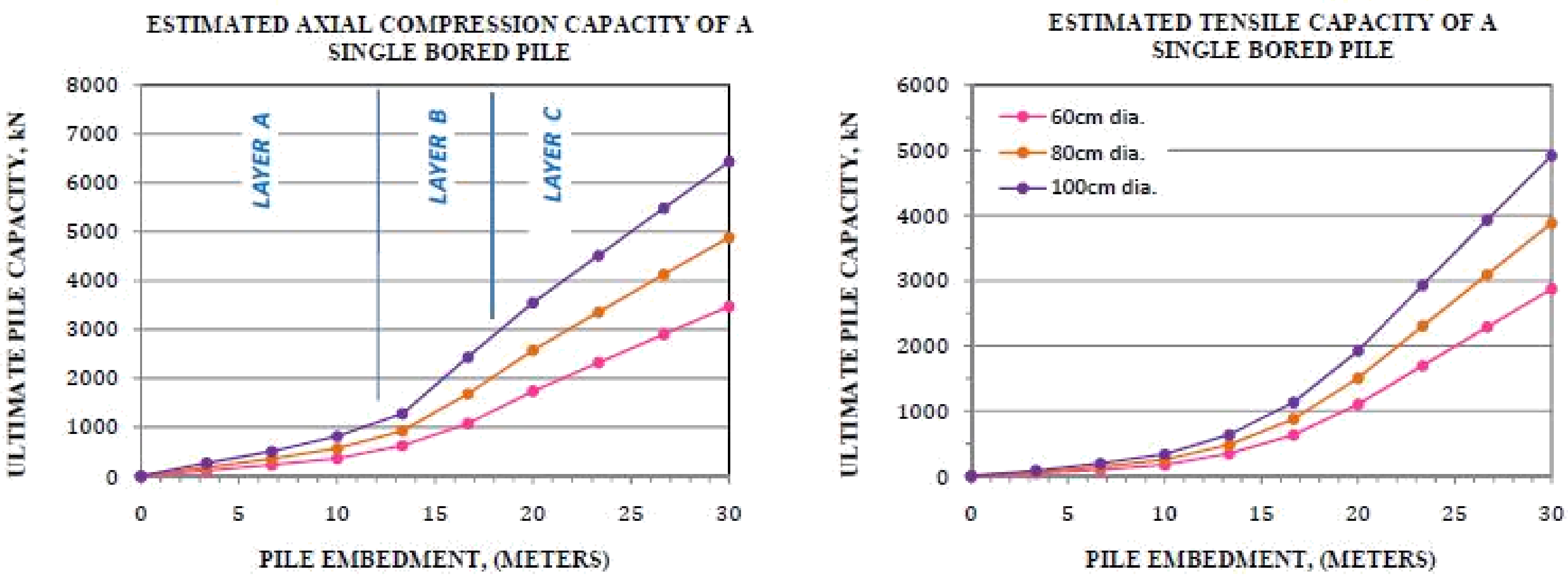

Figure 7-2. Estimated ultimate capacity of a single bored pile

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