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UTILISATION OF ALKALINE ACTIVATED INDUSTRIAL BY-PRODUCTS IN DEEP SOIL MIXING

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ABSTRACT

The use of deep soil mixing (DSM) technique in deep ground improvement projects has increased over the past decade due to being more cost-effective and easier to implement compared to other techniques such as piling, for structures subject to low to medium loads. Currently, Portland cement, lime and their combination are being used as the most common binders in DSM. However, due to the economic and concerning environmental disadvantages of using these binders, there is a need for new environmentally friendly cementing materials. This research attempts to find a way to use stockpiles of industrial by-products, such as fly ash (FA) and slag (S), as new green binders; consequently, reducing the carbon footprint in ground improvement projects. Different contents of FA and S, activated by liquid alkaline activator (L), were added to a soft marine soil to evaluate the changes in its behaviour as well as its microstructure. In addition, mixtures with cement (C), lime (Li) and their combination were prepared and tested for comparison. Binders were added at contents of 10, 20 and 30%, by dry soil mass, and samples were cured for 7 days. The results revealed that these new binders significantly increased the strength and stiffness of the soft soil, and they can be a suitable replacement for C and Li. The optimum mixture was found to be CIS+5% FA+15% S, within the range of binder, L and water content studied in this research. Moreover, recycling FA and S would substantially limit the expansion of landfill sites.

Keywords: Fly ash, Slag, Geopolymer, Alkaline activation, Deep soil mixing

INTRODUCTION

In the southern part of the Central Business District of Melbourne, Australia, which is located in the Yarra Delta, a highly compressible soft marine soil, known as Coode Island Silt (CIS), with high water contents exists. The vast presence of CIS, with poor engineering characteristics such as low bearing capacity (undrained shear strengths up to 80 kPa depending on the depth), in this area causes geotechnical constraints in construction projects. Proper deep ground improvement techniques need to be implemented to improve these soft soil deposits as they extend up to 30 m in depth [1]-[3]. In the recent years, new deep ground improvement technologies such as deep soil mixing (DSM) have drawn the attention of researchers and engineers. For structures bearing low to medium loads such as road embankments, DSM is faster and cheaper with less practical restrictions compared to traditional methods such as piling [2], [4].

In DSM method, an auger-mixing tool is drilled down to the intended depth while injecting a cementitious material, as binder, to mix with the native soil. The result would be circular columns of treated soil with improved engineering properties compared to the native soil. Cohesive soils with high moisture contents, such as CIS, are most suited to be treated with the DSM method. Current major binders being used are cement (C) and/or lime (Li) with the contents of 100-500 kg/m^3 (up to 30% by mass) of soil [5]-[7]. Due to economic and environmental concerns such as high energy and natural resource consumption and CO_2 emission during the production of these binders, attempts are being made to find alternative binders lowering the aforementioned disadvantages in recent years. There is a great potential in utilizing industrial by-products such as fly ash (FA) and slag (S) as they are available abundantly in landfills; consequently, eliminating the production concerns associated with traditional binders. Moreover, this can be a solution to the mentioned wastes disposal problems. These efforts in recent years have led to the introduction of green binders termed as geopolymers. Geopolymer is an inorganic product of blending precursors, materials rich in alumina and silica such as FA and S, with liquid alkaline activators (L) [8]. FA is a by-product of coal combustion in power plants, and S is a by-product of iron and steel manufacturing.

There has been a great deal of research conducted on the use of DSM method and geopolymers, especially utilizing FA and S, in the construction industry [2], [6], [8]-[18]. However, since geopolymers are relatively new in civil engineering, there is limited knowledge on the use of these binders in ground improvement, especially in the DSM application. This study investigates utilizing FA and S based geopolymers, as an alternative sustainable material with low carbon footprint, to improve the properties of soft soils in DSM application. The main objective of this research is to study the reliability of using geopolymers, as green binders, compared to traditional binders in implementing the DSM technology in soft soils with high water contents through unconfined compressive strength (UCS) and microstructure analysis. Utilizing FA and S as industrial by-products, which are often discarded to landfills, to produce geopolymeric binders in a sustainable manner could lead to finding proper replacements for traditional binders, both environmentally and economically.

MATERIALS AND METHODS

Materials

The CIS was collected at depths of approximately 3 to 5 m in field, and samples were then put in plastic bags and transferred to the laboratory. Soil classification tests, such as Atterberg's limits tests, were conducted on the CIS to determine its physical properties. Figure 1 presents these characteristics including particle size distribution of CIS, through conducting sieve analysis and hydrometer tests. the fine fraction of soil was 90% and the coarse fraction was 10%, with the maximum particle size (D_{max}) being 0.15 mm. the soil had a liquid limit (LL) of 50.4% and a plastic limit (PL) of 23.4%, resulting in a plasticity index (PI) of 27.0%. From the results of these tests, the CIS was classified as a silty clay with high plasticity. The FA, S, C and Li were collected from local suppliers in Melbourne.

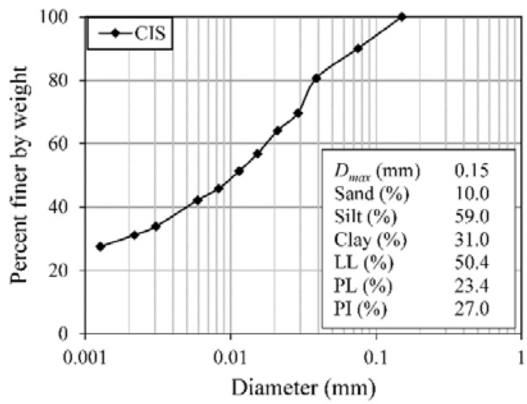


Fig. 1 Properties of CIS.

Figure 1 presents the SEM images of CIS, FA, S, C and Li. It is noted that the CIS particles are almost clustered with irregular shapes, the C and S particles are very similar having irregular shapes and sharp edges, the FA particles are in spherical shapes and have smooth surfaces, and the Li particles show a porous medium.

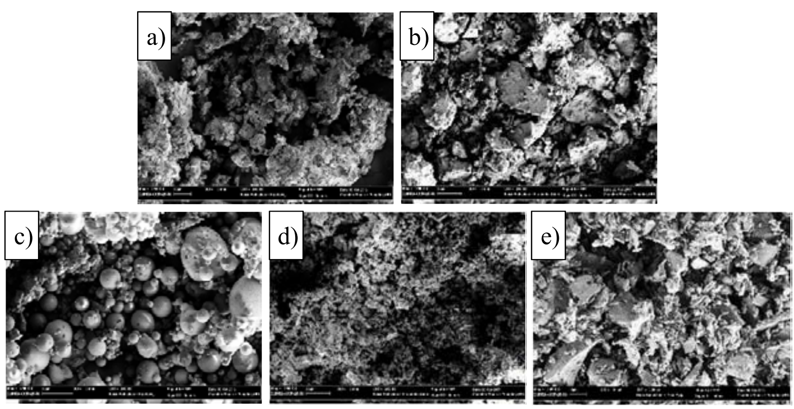


Fig. 1 SEM images of: a) CIS, b) C, c) FA, d) Li and e) S.

A combination of sodium hydroxide (NaOH), obtained in the form of beads, and sodium silicate (Na_2SiO_3), obtained as solution, was used as the L. Following safety and financial factors and previous recommendations, NaOH was prepared to 8 molarity and a Na_2SiO_3:NaOH ratio of 70:30 was used [3], [14], [16], [18].

Sample Preparation and Testing

Following the range of binder content considered for DSM [6], [7], binder or precursor contents of 10, 20 and 30 %, based on dry CIS mass, were used in this study. The mixtures were prepared as presented in Table 1. For the mixtures where traditional binders, C and Li, were used, C:Li ratios of 100:0, 50:50 and 0:100 were considered. For the geopolymeric binders, FA:S ratio was considered as 25:75 since S results in higher strengths, and in less time, compared to FA. Nevertheless, FA is needed to achieve a coexistence of Calcium Silicate Hydrate gel, due to the presence of Calcium in S, and Sodium Aluminosilicate Hydrate gel, geopolymer product, for better improvements of the soil properties [3], [11]-[13], [17]. The ratio of activator to precursor was chosen as 1 based on the recommendations by previous researchers [3], [14]. The water content of the CIS was set at its LL, to replicate the field conditions, before mixing with other materials. Previously, the LL has been reported as the optimum water content for stabilization of high water content clays [3], [6], [10], [14].

Table 1 Mix designs

Mixture	10%				20%				30%			
	C	Li	FA	S	C	Li	FA	S	C	Li	FA	S
CIS+C	10	-	-	-	20	-	-	-	30	-	-	-
CIS+C+Li	5	5	-	-	10	10	-	-	15	15	-	-
CIS+ Li	-	10	-	-	-	20	-	-	-	30	-	-
CIS+FA+S	-	-	2.5	7.5	-	-	5	15	-	-	7.5	22.5

Note: The values are in percent (%) by mass of dry CIS.

For preparation of geopolymer-stabilized samples, first, the CIS and precursors were mixed in a mechanical mixer for 2.5 minutes, before L was added and mixed for another 2.5 minutes, resulting in a total mixing time of 5 minutes. For the feasibility of comparison, traditional binder-treated mixtures were prepared by adding the binder to CIS and mixed for 5 minutes. After mixing the materials, the mixtures were placed into PVC split molds to prepare cylindrical specimens with 38 mm diameter and 76 mm height. Three samples were prepared for each mix to assure the test results were consistent.

Mixtures were put into the molds in two layers, each layer tapped 25 times on the table to remove the entrapped air. The unit weight of the specimens of all mixtures was checked for consistency. The samples were then wrapped with plastic films, put in a humid room with constant temperature (23±1 °C), dismantled the next day, wrapped again, and put in the humid room again to be cured. The overall curing time was 7 days.

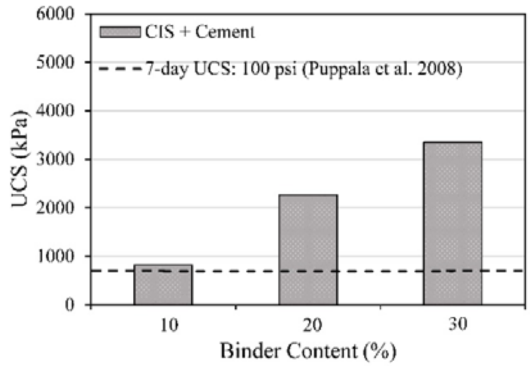
After the curing period, UCS tests were conducted on samples with a 1-mm/min (1.32%/min) rate of displacement and stopped manually after the specimens reached a considerable post-peak strength loss. After conducting the UCS tests, small samples were taken from the failed specimens for further analysis by scanning electron microscopy (SEM) imaging tests. The taken samples needed to be dry before being gold-coated and put in the SEM device. Thus, samples were put in the oven at 50 °C overnight before conducting SEM tests.

RESULTS AND DISCUSSION

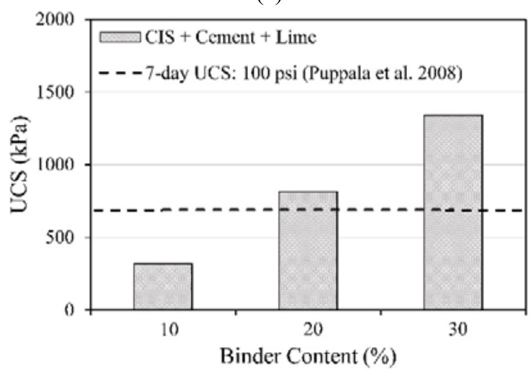
UCS Tests

Figure 2 presents the results of UCS tests on different mixtures of CIS stabilized with various combinations of binders, C and/or Li, at three contents of 10, 20 and 30%. Puppala et al. [19] suggested a minimum 7-day UCS value of 100 psi (689.5 kPa) for DSM ground improvement using C, which is presented in Fig. 2. As noticed, in the mixtures stabilized with traditional binders, CIS-C mixtures show the highest strength improvement, and as Li content increases, there is a significant decrease in the UCS, as previously observed [20]. Furthermore, while the strength development is almost linear with binder content increment in CIS-C-Li and CIS-Li mixtures, the rate of increase reduces after addition of 20% binder in CIS-C mixtures. This demonstrates that the optimum C content for ground improvement in CIS, with high water content, is 20%, as reported earlier [2], [6], [10], [15].

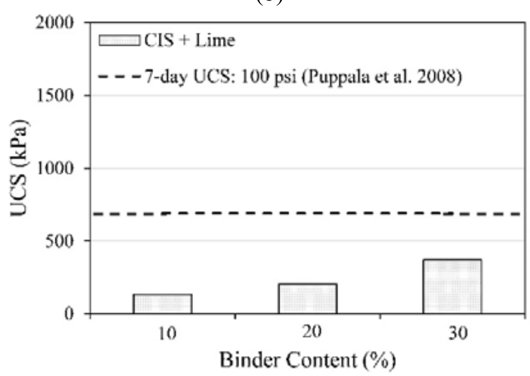
The UCS values of CIS stabilized with precursors, FA and S, are illustrated in Fig. 3. It is noted that there is a significant increase in the UCS of geopolymer-stabilized mixtures when the precursor content is increased from 10% to 20%, followed by a lower rate of strength enhancement when 30% precursor is added. Previously an increase in strength with precursor content increment has been reported; however, 20-25%, by dry mass of soil, has been reported as the optimum [8], [15]. It can be concluded that, within the range of precursor contents studied here, 20% is the optimum content to use.



(a)



(b)



(c)

Fig. 2 UCS values for mixtures stabilized with: a) C, b) C+Li and c) Li.

Overall, comparing traditional and geopolymeric binders reveals that, except for 10% binder content, using geopolymeric binders results in considerably higher strengths. Moreover, although CIS+10% C and CIS+10% C+10% Li meet the minimum UCS requirement, it should be noted these are the values achieved in the lab, while the values obtained in the field can be 2 to 3 times lower [9]. Therefore, CIS+20% C and CIS+5% FA+15% S seem to be the optimum mixtures in terms of strength. Considering the economic and environmental factors, mentioned before, using FA-S-based geopolymeric binder in DSM ground improvement of CIS is a beneficial replacement for C and/or Li binders.

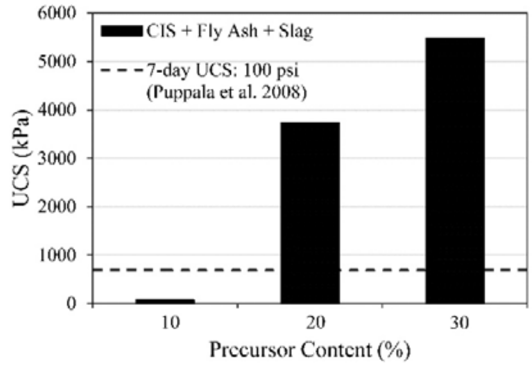


Fig. 3 UCS values for mixtures stabilized with FA and S.

SEM Tests

To analyze the microstructure of the mixtures, SEM tests were conducted on the samples after the UCS tests. The SEM images of CIS stabilized with C and/or Li and geopolymeric binder, with 20% binder or precursor content, are presented in Fig. 3. Figure 3 (a) shows the SEM image of CIS+20% C. It is noticed that the C particles are almost dissolved in the mixture, resulting in a dense medium with strong bonds. As C is gradually being replaced with Li, voids start to appear in the structure, Figs. 3 (b) and (c), to the extent that in Fig. 3 (c) where the binder is merely Li, a completely porous matrix is observed. These images further confirm the considerably higher UCS values of CIS+C mixtures compared to those of CIS+C+Li and CIS+Li mixtures.

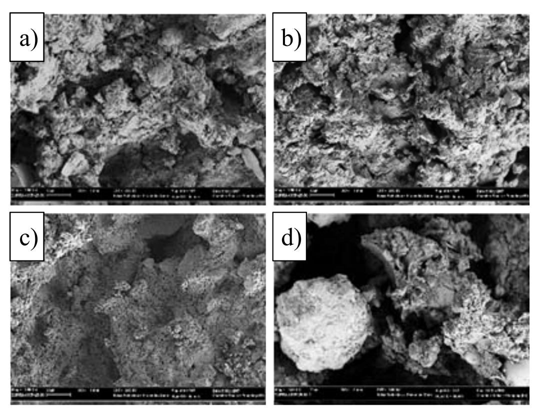


Fig. 3 SEM images of: a) CIS+20% C, b) CIS+10% C+10% Li, c) CIS+20% Li and d) CIS+5% FA+15% S.

In Fig. 3 (d), reacted FA and S particles are clearly evident that leads to a compact morphology with strong geopolymer gel bonds. This strong structure proves proper activation of FA and S with the used L and water content; and accordingly, the high strength achieved by utilizing geopolymeric binder.

CONCLUSION

A series of UCS and SEM tests were conducted on a clayey soil, CIS, treated with different contents of geopolymeric binders, obtained by activation of industrial by-products such as FA and S, and traditional binders, C and/or Li. The aim was to investigate the performance of utilizing these by-products in DSM application and comparing their reliability with that of the traditional binders.

UCS tests results revealed that increasing the binder content to 30% increased the strength in all mixtures. However, the rate of increase was lower when the binder content was increased from 20% to 30%, especially when using the geopolymeric binder. In terms of strength development, using geopolymeric binder resulted in the highest improvement, followed by using C, C+Li and Li as binders. This was evident from the microstructure of the mixtures using SEM images.

Overall, CIS+5% FA+15% S was found to be the optimum mixture, within the range of binder, L and water content studied in this research. Utilizing stockpiled FA and S in landfills in ground improvement technologies will not only reduce the financial and environmental consequences of using traditional binders, but also will be a solution to the problems regarding the disposal of these wastes.

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DEEP FOUNDATION REFERENCE FOR METRO MANILA, PHILIPPINES

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ABSTRACT

The study focuses on the analysis of the pile capacity and pile length for various cities of Metro Manila. Standard Penetration Test (SPT) N-values from soil reports were used to compute for the geotechnical parameters such as the undrained shear strength and the angle of internal friction which were directly applied in the computation of the allowable pile capacity. The estimation of the pile length, on the other hand, was done by means of determining the depth of the refusal or rock layer. The proposed minimum pile length and the allowable pile capacity values for each city are plotted to establish a contour map. By means of the collected borehole data, the allowable pile capacity was computed, which was shown in the reference as a series of contour maps. The contour maps were provided to show an overview of the soil's pile capacity at various locations in Metro Manila, Philippines. The contour maps presented vary by means of the design of pile, the size of the pile and the proposed pile length for a specific city or municipality and for the entire Metro Manila. A Geographic Information System (GIS) database was made so as to have storage for the collected borehole data and their locations. The database can be updated for the availability of new data.

Keywords: pile capacity, pile length, deep foundation, foundation reference

INTRODUCTION

Metro Manila's buildings are comprised of at least 70% high-rise buildings and skyscrapers, excluding for the cities of Caloocan, Navotas, Las Piñas, Pateros, and Malabon due to the absence of data [1]. High-rise buildings and skyscrapers, compared to low-rise buildings and residential dwellings, would usually require pile foundation. With that, soil exploration or excavation would be needed for every construction.

With such a large amount already allocated to the construction of the structure itself, a great sum of money must again be allotted to the necessary soil explorations, which are not just costly but time consuming. Reducing the total construction expense would greatly benefit the structural engineer and the owner.

There have been some similar studies related to this research [2]-[8], however, there is not one compiled published or commercially available source of data for the entire Metro Manila, just scattered and separate ones. Due to this, if a structural or geotechnical engineer desires a second reference for the soil data of a specific area in Metro Manila, they would need to acquire them from different sources. This, in turn, would take tremendous amount of time and effort. Thus, this paper will try to help structural engineers by giving them a reference for the pile capacity and pile length needed for the construction of a structure in need of pile foundation in any specific place of Metro Manila. The main objectives of this paper are to estimate the necessary pile lengths for the different areas of Metro Manila and to determine the proposed pile capacity in the entire Metro Manila.

METRO MANILA

Metro Manila, Philippines is bounded by the provinces of Bulacan in the northern part, Rizal in the eastern part, Cavite and Laguna in the Southern part. Manila Bay is located on the western part of Metro Manila, while Laguna de Bay is on the eastern part. Metro Manila has a total land area of 615.39 square kilometers. Basing on geographical coordinate system, the entire Metro Manila lies between 120° 54' and 121° 9' longitudinally and 14° 20' to 14° 47' along the latitude direction.

At some geologic past, the Metro Manila was submerged underwater, which extends up to the mountains in the Eastern part. Intermittent volcanic activities occurred which resulted to the deposition of volcanic materials. During intervening period of inactivity, a layer of sediments are placed on top of the previously laid out volcanic materials which resulted to the common characteristic of the geologic deposit which is alternating beds of tuffaceous materials and transported sediments.

Majority of the sediments present in the geologic deposit of Metro Manila are due to the bodies of water surrounding it which include Manila Bay, Pasig River, and Laguna de Bay. The sediments that were transported consist of sands, pebbly gravels, silts and clays with various traces of fossil remains or marine shells and several organic particles. The presence of organic particles and fossil remains gives an idea of a swampy environment which prevails during a time which has a shallow water level.

The Guadalupe Tuff formation, the underlying rock formation of Metro Manila, was generally well-consolidated and well-cemented. The tuff formation extends from Quezon City and Novaliches up to the province of Cavite in the south. Majority of its composition is lithified volcanic ash, lapilli and sands. When it comes to the thickness of the tuff formation, it remains to be uncertain. Several areas where the tuff formation is present are overlain by layers of sediments which generally thicken as it approaches the west side of Metro Manila, which is Manila Bay.

Overall, the composition of the geologic deposit can be attributed to its elevation. For highly elevated locations, it is composed of dense sands and tuffaceous clays. For low – lying areas of Metro Manila, it is generally composed of loose sands and soft clays [9].

METHODOLOGY

The aim of this study is to create a deep foundation reference for the district of Metro Manila. Borehole logs with a ratio of one borehole log per square kilometer were collected and compiled accordingly as shown in Figure 1. The borehole logs were accumulated from the different private companies and government institutions in Metro Manila. A total of 677 borehole locations were collected and mapped. For the attained data or soil reports, it already comprised almost 86% of the total target. However, this number of data does not yet include the outskirts of Metro Manila which are from Rizal, Bulacan, Cavite and Laguna. The outskirt data were used to provide accurate mapping even at the near boundaries of Metro Manila. The data are then analyzed and calculated for the depth of rock formation and geotechnical parameters.

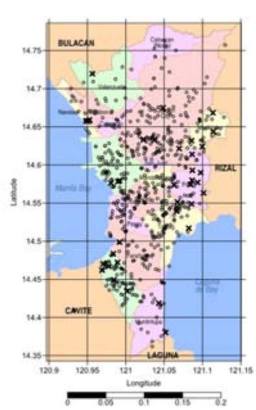

Fig. 1 Map of Manila with borehole locations

The estimation of the pile length and the computation of the allowable pile capacity were performed though an excel program. The soil properties, SPT N-values and RQD were inputted in the program to get some vital geotechnical parameters like the undrained shear strength [10] and the angle of internal friction [11]-[14]. The minimum pile length was estimated depending on the soil condition whether until the refusal layer (SPT-N 50), rock layer (RQD) or even at the last layer of the borehole log in the absence of the refusal or rock layer. The SPT N values that are available in the borehole logs that were collected are the main components used for the computation of the pile capacity. The design of the piles was limited to a range of sizes and shapes. As for the proposed length of the piles, the depth of the rock formation or refusal layer was used as with a one meter embedment on the hard layer or the last soil layer.

The allowable pile capacity, on the other hand, was computed based on the skin friction and end bearing resistance which are both dependent on the geotechnical parameters. The results were summarized in a form of contour maps for easy visualization and interpretation per city. Likewise, to provide a good analysis of the values of the allowable pile capacity, skin-to-tip ratio were also considered and plotted in the maps. This is to provide a support which between the skin friction and the end bearing resistance contributed greater value in the allowable pile capacity, which in turn, describes what kind of soil does a city, in particular, have and how long the pile length is.

The pile capacity was computed by means of the theory of the alpha and beta method**.** The alpha method is used to estimate the pile capacity especially for clayey soil layers. It uses a factor denoted as in approximating the value of the skin friction and a coefficient Nc to compute for the end-bearing capacity. The skin friction for any types of piles using the alpha method includes the coefficient, the undrained shear strength and the lateral surface area of the pile [15]**:**

$$f_S = \alpha_u S_u; \qquad (1)$$

$$Q_f = \sum (f_s) x (\text{perimeter}) x (\text{length}); \qquad (2)$$

where:
f_s= skin friction stress
α_u= coefficient for skin friction
S_u = undrained shear strength .

Beta Method is similar to alpha method in such a way that it uses coefficients but this time, it is denoted as for skin friction and Nq for end bearing resistance. Unlike alpha method, beta method considers both sandy and clayey soils. The general equations using the beta method is quite similar to alpha method but instead of undrained shear strength, the effective stress is used.

The skin friction for any types of piles using the beta method includes the coefficient, the effective stress and the lateral surface area of the pile [16].

$q_f = \sum (\beta\sigma_z)x(perimeter)x(length);$ (3)

where:
β = coefficient for skin friction;
σ'_z = effective stress;
q_f = skin friction .

Some of the borehole data have rock layers designated by RQD or Rock Quality Designation. The computation using the alpha and beta methods are not applicable to rocks anymore. The pile capacity is now based on the end bearing resistance of the rock which is far greater than the soil. Moreover, the skin friction is neglected in the computation of the pile capacity of rock. O' Neill and Reese [7] approximated the ultimate end bearing resistance through the formula:

$q'_t = 4830\,(q_u)^{0.51};$ (4)

where:
q'_t = end bearing resistance;
q_u = unconfined compressive strength of rock
ϕ' = drained angle of friction

For a better visualization of the acquired data and computed values, the allowable pile capacity values are then mapped out by means of contour maps, implementing the kriging method. Verification for both the data accomplished and the produced contour maps were done as well. The computed values of allowable pile capacity were stored in the GIS database as well.

DEEP FOUNDATION REFERENCE

Proposed Pile Length

The proposed pile length map for the entire Metro Manila can be seen in Figure 2. The map reflects the type of soil present where majority of the area that is underlain by the Guadalupe Tuff Formation has a pile length that ranges from 5 to 10 meters in length. There are regions, which can be seen as areas shaded with white, have pile lengths of 5 meters for the entire region. These regions are recommended for the use of shallow foundation due to the shallowness of the rock layer or refusal layer. For locations composed of alluvial deposits, the range of pile lengths varies significantly depending on the location. For the western part, the pile length ranges from 10 to 15 meters. Several parts of the region show lengths ranging from 20 to 25 meters. The effect of the Manila Bay, in terms of pile length, is manifested through these results. For the eastern part of Metro Manila, the proposed pile length ranges from 10 to 25 meters, which shows the effect of the location with respect to Laguna de Bay, where majority of the data collected near the said body of water possess thick layers of alluvial deposits, namely sand, silts and clay. Generally, the proposed pile length for Metro Manila ranges from 5 to 15 meters.

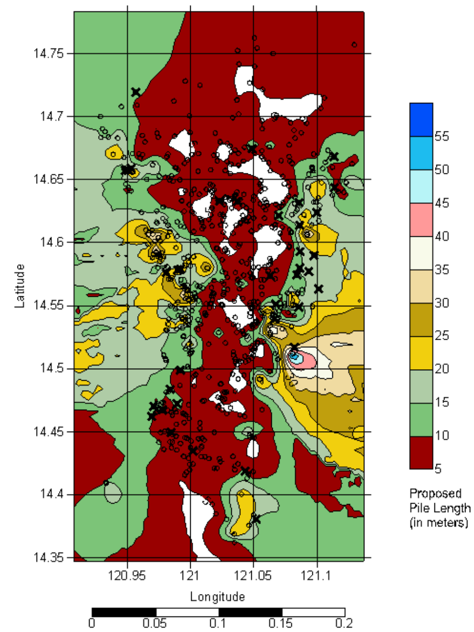

Fig. 2 Proposed Pile Length

Proposed Skin-to-Tip Ratio

The skin-to-tip Ratio is the ratio between the skin friction and the tip resistance or the end bearing resistance. For the skin-to-tip ratio map for Metro Manila, the values can also be reflected by the type of soil present along those areas, as seen in Figures 3 and 4. The areas underlain by the Guadalupe Tuff Formation produces low skin-to-tip ratio whereas a high ratio is observed in areas where alluvial deposits are prevalent. In general, the skin-to-tip ratio has presented that for a certain type of pile, a particular resisting force is dominant over the other, that is, skin friction is greater than the tip resistance, also known as end-bearing resistance, or vice versa. For driven piles, skin friction usually contributes greater resistance than that of the end bearing resistance due to the larger adhesion factors, α and β. Practically, the process of driving the piles really induces greater friction from the soil but the consequence is to use smaller cross-sections only so that the pressure in driving the piles is greater, thus, producing small end-bearing resistance. For bored piles, end bearing resistance dominates the skin friction because the cross-sectional area of the type of pile is quite large. Also, some soil layers are neglected for the computation of the skin friction due to the effect of drilling that make these particular layers disturbed.

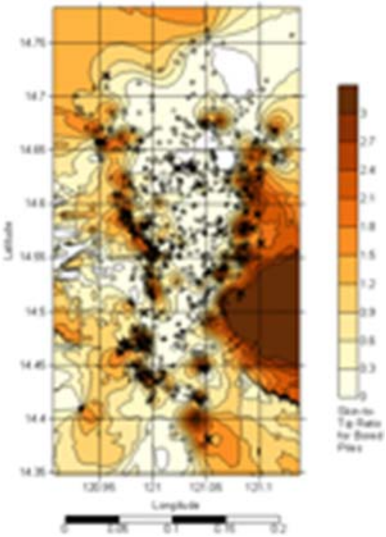

Fig. 3 Proposed Skin-to-tip Ratio (Bored Pile)

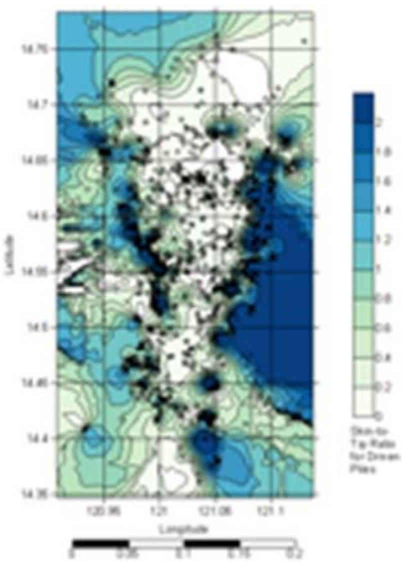

Fig. 4 Proposed Skin-to-tip Ratio (Driven Pile)

Proposed Pile Capacity

The allowable pile capacity maps presented under Figures 5, 6 and 7 show consistency in terms of their distribution of values. A separated sample pile capacity map where the division of the type of soil can be seen under Figure 8 (Guadalupe Tuff Formation) and Figure 9 (Alluvial Deposits). Higher values of allowable pile capacity are found in areas where Guadalupe Tuff Formation is located. On the other hand, areas with alluvial deposits have low allowable pile capacities. Generally, it shows that the pile capacity depends on the type and quality of soil present on a specific area.

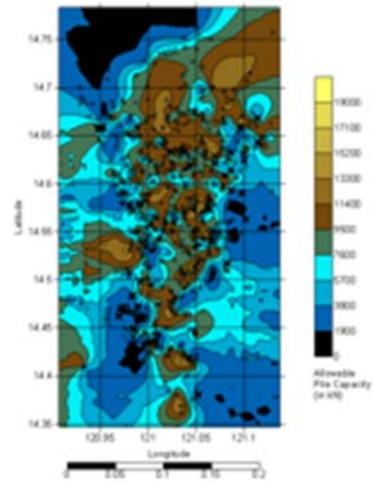

Fig. 5 Pile Capacity (Bored Piles Size 1.50 meter)

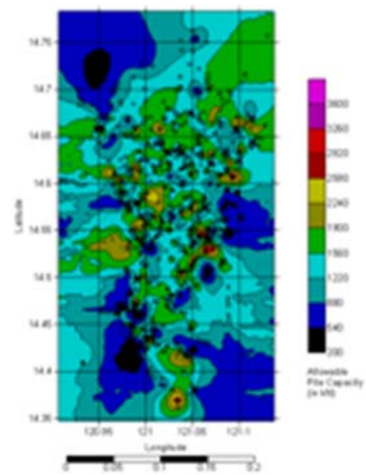

Fig. 6 Pile Capacity (Square Driven Piles Size 0.50 meter)

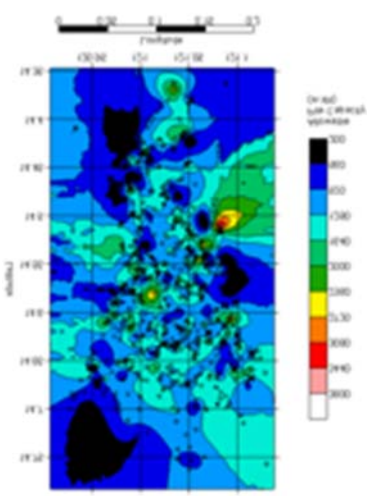

Fig. 7 Pile Capacity (Octagonal Driven Piles Size 0.50 meter)

Fig. 8 Pile Capacity for Bored Pile (Guadalupe Tuff Formation Area)

Fig. 9 Pile Capacity for Bored Pile (Alluvial Deposits Area)

CONCLUSION

In this paper, the pile length and allowable pile capacity are determined through borehole logs. For the cities of Las Piñas, Malabon, Navotas, Pateros and Pasig, they have relatively small allowable pile capacities as compared to the other cities for both driven and bored piles. Pile lengths are shorter beause they have reach the refusal layers at somehow a shallow depth thus, lesser skin friction is induced. Moreover, based on the skin-to-tip ratio, skin friction still plays a great contributor over the end-bearing resistance although in totality, the allowable pile capacity is still small as compared to other cities.

The cities of Makati, Mandaluyong, Paranaque, Makati and Quezon are quite remarkable not only due to the high allowable pile capacities that they produce but to the short pile lengths as well. Having a short pile length, in this case, is quite advantageous because it is really cost-effective. Also, it does not affect too much the allowable pile capacity because it majorly relies on the end-bearing resistance. The Guadalupe Tuff formation is actually the factor which makes the end-bearing resistance greater. These cities have shallow rock layers and usually, shallow foundation is recommended for most of the areas of the aforementioned cities. Skin-to-tip ratio has proven that the end-bearing resistance really governs in these cities.

There are cities in Metro Manila which are not recommendable for shallow foundation just because the top layers are weak especially in bearing capacity. These include Manila, Marikina and Pasay. However, when pile foundation is used for these cities, a large allowable pile capacity is computed. This is because a longer pile length is recommended to induce large skin friction from several soil layers and to reach the refusal or rock layer at great depth. In these particular areas, both the skin friction and end bearing resistance greatly contribute to the allowable pile capacity. This means that high loadings from the superstructure can be resisted by piles considering also its length. The trade-off, however, is that it is not cost effective anymore due to the long piles that require great amount of materials.

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