Impact of Wick Drain Installation on the Shear Strength Parameters of Cohesive Soils

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ABSTRACT
Application of prefabricated vertical wick drains to reduce post construction settlement of soft cohesive soil is well known in geotechnical industry. However, effect of wick drains on the soil shear strength parameters has not been reported duly in the literature. The changes in soil strength parameters, as a consequence of wick drain installation, are evaluated in this paper for a new electrical substation for the City of New Westminster, BC. The proposed substation is located in the Queensborough area on the east end of Lulu Island within the Fraser River Floodplain to provide electrical supply to the community as well as some additional back-up capacity for the rest of the city. A preload was placed at the substation footprint in August 2019 in order to induce settlement of the upper cohesive soil. SNC-Lavalin used the settlement monitoring data in conjunction with vibrating wire (VW) piezometer data to back calculate and adjust soil consolidation parameters. It was concluded that, in addition to the existing preload, a wick drain installation program is required to speed up the consolidation settlement rate to meet the target construction schedule and reduce the long-term post construction settlement of the site to a tolerable threshold of the structures. Cone Penetration Tests (CPT) were performed before preload placement in 2019, and after wick drain installation in 2021. The CPT data were used to evaluate the impact of wick drain installation on the shear strength parameters of the soft clayey soils. Interpretation of CPT data indicated an average increase of about 20% to 30% for the undrained shear strength of silty clay/clayey silt layer.

RÉSUMÉ
L'application de drains à mèche verticaux préfabriqués pour réduire le tassement post-construction des sols cohésifs mous est bien connue dans l'industrie géotechnique. Cependant, l'effet des drains à mèche sur les paramètres de résistance au cisaillement du sol n'a pas été dûment rapporté dans la littérature. Les changements dans les paramètres de résistance du sol, en conséquence de l'installation de drains à mèche, sont évalués dans cet article pour une nouvelle sous-station électrique pour la ville de New Westminster, BC. La sous-station proposée est située dans la zone de Queensborough, à l'extrémité est de l'île Lulu, dans la plaine d'inondation du fleuve Fraser, et fournira une alimentation électrique à la communauté ainsi qu'une capacité de secours supplémentaire pour le reste de la ville. Une précharge a été placée au niveau de l'empreinte de la sous-station en août 2019 afin de provoquer le tassement du sol cohésif supérieur. SNC-Lavalin a utilisé les données de surveillance du tassement en conjonction avec les données du piézomètre à fil vibrant (VW) pour calculer et ajuster les paramètres de consolidation du sol. Il a été conclu que, en plus de la précharge existante, un programme d'installation de drains à mèche est nécessaire pour accélérer le taux de tassement de la consolidation afin de respecter le calendrier de construction cible et de réduire le tassement post-construction à long terme du site à un seuil tolérable pour les structures. Des essais de pénétration au cône (CPT) ont été réalisés avant la mise en place de la précharge en 2019 et après l'installation de drains à mèche en 2021. Les données CPT ont été utilisées pour évaluer l'impact de l'installation du drain à mèche sur les paramètres de résistance au cisaillement des sols argileux mous. L'interprétation des données CPT a indiqué une augmentation moyenne d'environ 20 à 30 % pour la résistance au cisaillement non drainée de la couche d'argile limoneuse/limon argileux.

1 INTRODUCTION

SNC-Lavalin Inc. was commissioned by the City of New Westminster to design and support the construction of a new electrical substation on the east end of Lulu Island in the Fraser River Floodplain, New Westminster, BC. The proposed substation is located in the Queensborough area and will provide electrical supply to the community as well as some additional back-up capacity to the rest of the City. The substation comprises of a control building, a 12 kV switchyard and a 60 kV switchyard containing two power transformers.

The subsurface of the site consisted of a thick layer of high plastic soft silty clayey soil which was prone to considerable long-term settlement. Preload fill placement was proposed to reduce post construction settlement. However, the settlement monitoring and vibrating wire (VW) piezometer data indicated that it is unlikely that the predicted degree of primary consolidation would be achieved by the construction commencement date leading to a long-term post construction settlement beyond the structural tolerance. Hence, a wick drain installation program was proposed, in conjunction with the existing preload, to accelerate consolidation settlement in order to meet the construction schedule.

Prefabricated vertical drains (PVD), or wick drains, are a method of ground improvement used in fine-grained soils to accelerate the primary consolidation and reduce the resulting settlement in long term. Although there are a variety of wick drains available in the market, the general design consists of a core, to transmit the drained pore water, wrapped in a geotextile jacket to facilitate rapid flow without increasing the risks of soil migration.

The pore pressure dissipation will also increase the shear strength by increasing the effective stress and was studied in laboratory by different authors such as Bolton and Sharma (1994) and Sharma and Bolton (2001). Other factors such as drain spacing, drain length, cross-sectional area of the mandrel can also result in different outcomes where sensitive soils are anticipated (Holtz et al. 1991).

Application of PVDs to reduce post construction settlement of soft cohesive soil is well-known in geotechnical industry. However, the impact of wick drains on shear strength parameters of soil has not been reported duly in the literature. This paper discusses the impacts of wick drain installation, in conjunction with preload placement, on the undrained shear strength of existing soft cohesive soil underside of the proposed development. Note that the other aspects of the design and associated geotechnical risks are not addressed in this paper.

2 PRE-CONSTRUCTION GEOTECHNICAL FIELD WORKS

Geotechnical field investigation, field monitoring and instrumentation as well as preload placement are summarized in a chronological order as follow:

- January 2019: field investigation completed prior to the preload placement, which comprised of eight auger holes incorporated with Cone Penetration Testing (CPT).
- July 2019: vibrating wire piezometers and settlement monitoring plates were installed. In total, 16 settlement gauges, four vibrating-wire piezometers and three standpipe piezometers were installed across the site.
- August 2019: On average, 3 m height preload at the central portion of the site was placed and ground settlement and groundwater level was being monitored.
- August 2020: preload placement was extended to the northern and southern portions of the site.
- February 2021: installation of PVDs was completed. PVDs were installed to the bottom of silty clay/clayey silt layer to about 16 meters below ground surface (mbgs) with 3 m and 2 m spacing in triangular pattern in the central portion and the expansion areas to the north and south, respectively.
- May 2021: additional field investigation was completed which comprised of two seismic CPTs (SCPTs), one CPT, and six Horizontal to Vertical Spectral Ratio (HVSR) geophysical tests.

3 SUBSURFACE CONDITIONS

Based on the test hole logs and CPT/SCPT plots, the general stratigraphic profile underlying the site consisted of granular fill material to a depth of 0.9 to 2.7 mbsg over a very soft high plastic silty clay to clayey silt. The thickness of this cohesive layer ranged from 12 to 14 mbgs described as normally consolidated to slightly over consolidated. A fine to medium grained loose to compact sand (typical Fraser River sand deposits) was found beneath the clayey layer to about 30 to 38 mbgs, and in turn underlain by non-plastic silt and sand layer to the depth of exploration (Figure 1).

Interpretation of HVSR test results indicated that the post-glacial deposit (till-like layer) expected from about depth 80 m to 94 mbgs across the site.

Based on the available information, groundwater level is expected to be close to depth of about 2 mbgs in the vicinity of the site.

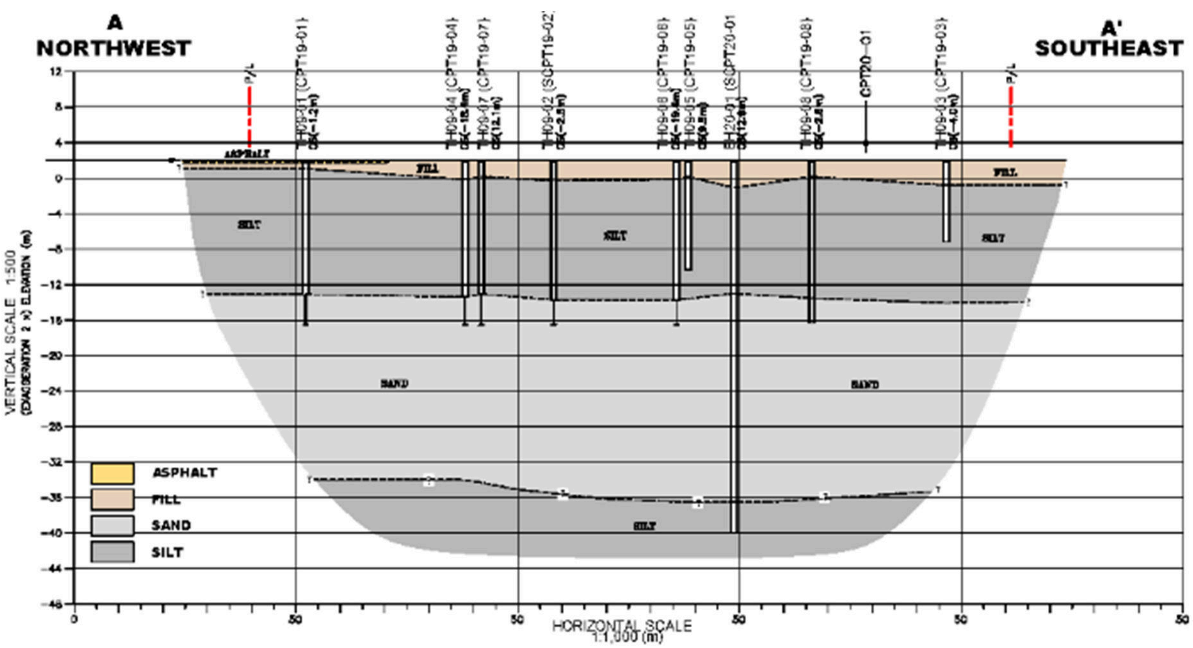

Figure 1: Generalized soil profile across the site

4 ANALYSES

4.1 CPT and SCPT Data Interpretation

To evaluate the impact of the PVDs to the strength of the cohesive silty clay layer, undrained shear strength estimated from the CPTs done in 2019, prior to preload placement, were compared with those completed in 2021 (i.e. after PVD installation). Two adjacent sets of data in the central and northern portion of the site were selected to evaluate the impact of closer spaced PVD, 2 m by 2 m spacing in the north, with wider spacing of 3 m by 3 m in the central zone.

The analyses provided in the sections below were derived from the adjacent CPTs, CPT19-04 and SCPT21-01 in the north along with CPT19-08 and CPT21-02 in the central preloaded areas. CPT data were interpreted using Geologismiki CPeT-IT v.3.

4.2 Soil Behaviour Type (SBT)

To evaluate the soil behaviour in the north and central zones, the normalized Soil Behaviour Type (SBT_n) values were obtained from CPeT-IT using the criteria provided by Robertson (1990). The SBT_n values indicated the soil materials in both central and northern zones showed a similar behaviour of clay to silty clay (SBT_n equals to 3) from around 4 mbgs to 16 mbgs (Figure 2).

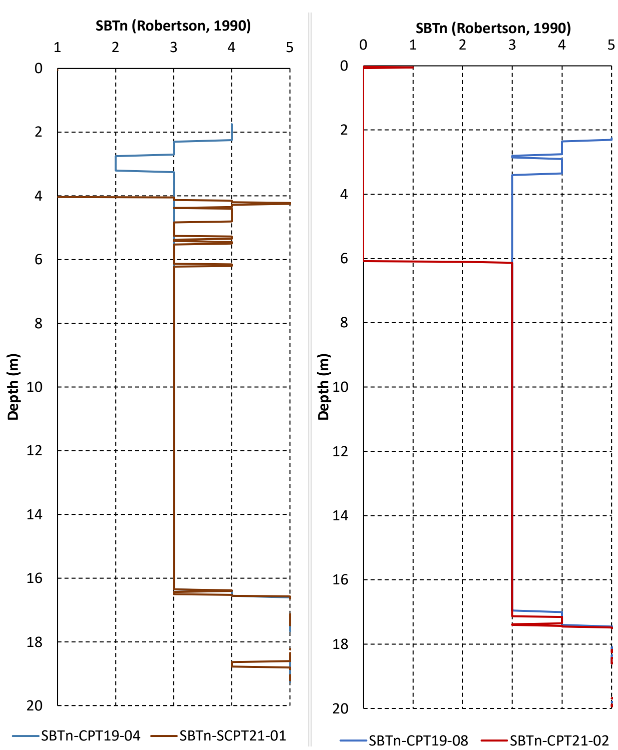

Figure 2- Normalized soil behaviour type (SBT_n) plots of CPTs at northern and central zones

4.3 Soil Sensitivity (St)

The soil sensitivity is defined as the ratio of undisturbed peak shear strength to remolded residual shear strength. For this study, the peak shear strength (S_u) calculated by CPeT-IT using correlation with tip resistance (q_t) and changes in pore pressure (Δu) were used to compare with residual sear strength equal to sleeves friction (f_s) (Robertson 2001, Schmertmann 1978).

Since the soil disturbance during PVD installation could have an adverse impact on the shear strength of highly sensitive clays, the soil sensitivity prior to wick drain installation was calculated for the clayey material using CPT data from 2019 (prior to the preloading). The criteria proposed by Skempton et. al. (1952) was used to evaluate the soil sensitivity (Table 1).

The results are presented in Figure 3, indicating the soil sensitivity index is all less than 3 in both north and central sections, showing the clay material at the site generally has low sensitivity.

Table 1.Classification of Sensitive Clay (Skempton et. al. 1952)

Soil Sensitivity (St)	Classification
1	Insensitive
1-2	Low Sensitivity
2-4	Medium Sensitivity
4-8	High Sensitivity
> 8	Extra Sensitive
>16	Quick Clay

4.4 Undrained Shear Strength (Su)

As stated above, undrained shear strength was estimated from CPT raw data, collected during the field investigations, using CPeT-IT. The undrained shear strength was defined as the difference between the tip resistance (q_t) and vertical stress (σ_v) over cone factor value (N_{kt}).

Changes of estimated S_u from 2019 to 2021 were examined and compared to evaluate the impact of the wick drains on the undrained shear strength and presented as histograms in Figure 4 and Figure 5. The results demonstrated that the shear strength of the clay was increased from an average of 25 – 35 kPa to 30 – 40 kPa in the northern zone and from 25 – 35 kPa to 35 – 45 kPa in the central zone.

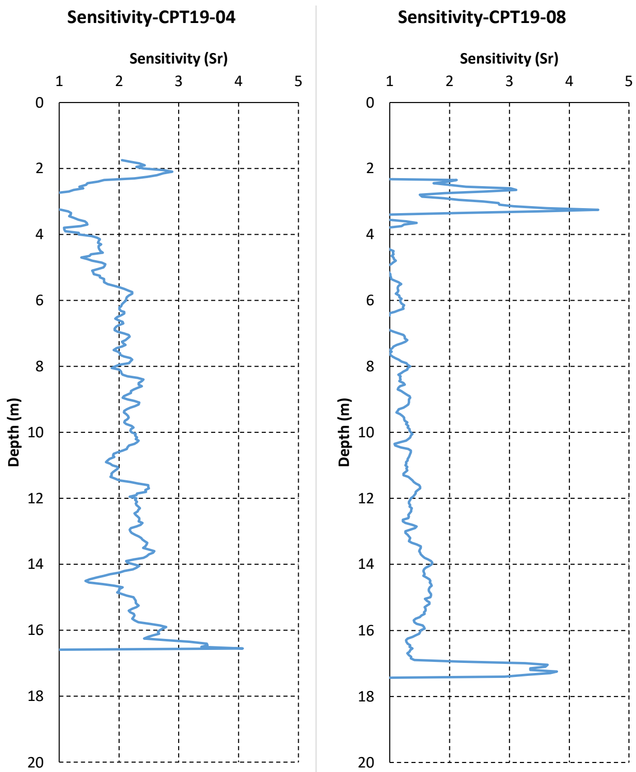

Figure 3. The soil sensitivity at the north and central zone prior to preloading

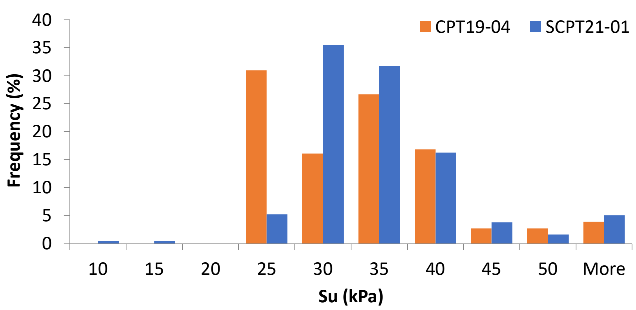

Figure 4. Frequency of S_u values measured in northern zone (CPT19-04 vs. SCPT21-01)

5 PILE DYNAMIC ANALYSIS

To evaluate the pile capacity, four Pile Dynamic Analysis (PDA) tests were done in November and December 2021, ten months after installation of the PVDs.

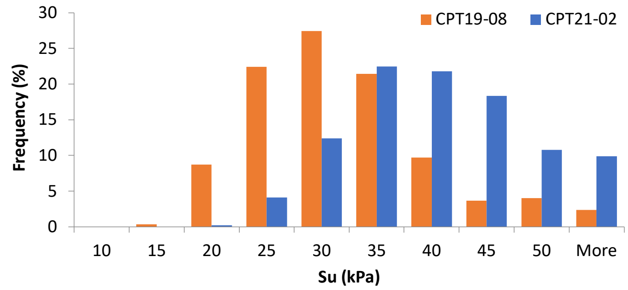

Figure 5. Frequency of S_u values measured in the central zone (CPT19-08 vs. SCPT21-02)

The PDA tests were distributed evenly and was performed on Pile #22 located at the Control Building footprint to, north of the site, Pile #53 located at the centre and Pile #82/Pile #112 located at the footprint of the Power transformers to the centeral and southern portion of the site. The piles tested were similar to the designed pile as open-ended spiral-welded steel pile of 610 mm (24 inch) diameter with 12.5 (1/2 inch) wall thickness. Similarly, a drop hammer of 29.8 kN (6,700 lbs) was used on all piles with a rated energy of 45.4 kJ corresponding to 1.52 m (5 ft) drop height for Pile #22, Pile #82 and Pile #112. The hammer rated energy was increased to 90.9 kJ corresponding to 3.05 m (10 ft) as needed.

Using the CAPWAP analysis, the shaft and Toe resistance were calculated for each pile. The results for the shaft resistance presented only in Table 2 considering its relevance to this paper.

Table 2. PDA test results

Pile Number	Shaft Resistance in Clay (kN)
Pile 22	996
Pile 53	877
Pile 82	841
Pile 112	981

The unit friction measured at each pile were calculated by dividing the measured shaft resistance to the penetration depth in cohesive materials and the surface area of the pile. Considering the anticipated undrained shear strength of the site in range of 30 kPa – 50 kPa, an Alpha value of 0.85 was assumed using the chart recommended by Vesic (1977) and presented in CHBDC Commentary on CSA S9-19. The summary of the back-calculated undrained shear strength were provided in Table 3.

Table 3. Back-calculated undrained shear strength from PDA test results

Pile Number	Pile Penetration in Clay (m)	Shaft Resistance in Clay (kN)[1]	Unit Shaft Resistance in Clay (kN)	Calculated Undrained Shear Strength, kPa[2]
Pile 22	14.7	996	67.8	42
Pile 53	14	877	63.0	39
Pile 82	13	841	64.7	40
Pile 112	13	981	75.5	46

Note 1 – Based on CAPWAP results
Note 2 – Calculated as Unit Shaft Resistance/(3.14 x 0.61 mm x 0.85); Alpha is assumed 0.85 as per Vesic (1977)

The results confirmed that the shear strength of the clay was increased from an average of 25 – 35 kPa to 40 – 50 kPa in the northern zone and from 25 – 35 kPa to 35 – 45 kPa in the central zone. All the PDA tests were performed after end of initial driving and the soils were not set up completely, so the presented values in Table 3 will be increased after soil set up completion.

6 CONCLUSION

Four months after wick drains installation, the instrumentation data (settlement survey points and VW piezometers) indicated that the 95% primary consolidation was achieved throughout the site. It is anticipated that the dissipation of the pore pressure, and hence increase in effective stress, resulted in increase of soil shear strength. However, considering multiple factors impacting such conclusion, a constant increasing rate could not be predicted without supplemental field investigation that compares the soil strength prior and after wick drain installation.

The analysis of other factors such as normalized Soil Behaviour Type (SBTn) and Soil Sensitivity (S_t) indicated that the clay deposits along north and central parts of the site would likely behave similarly under this ground improvement plan. Figure 4 and Figure 5 demonstrated that the undrained shear strength increased after the preload placement and PVD installation. These figures clearly show that at the central part of the site, where the maximum preload existed, the shear strength increase was more substantial compared to the northern part of the site, in which lesser preload was placed. It is worth noting that PVD spacing was 2 m and 3 m at the northern and central part of the site, respectively.

The following conclusion can be made from reviewing the results:

1. Where the sensitivity of the clay is low, an increase of 20% to 30% in undrained shear strength could be expected after wick drain installation and at about 95% consolidation rate (Figure 6 and Figure 7);
2. Some of CPT data indicated insignificant increase of S_u at greater depths (more than 10 m), mainly within the north portion of the site, although the soil classification parameters were comparable with the rest of the site. This could be due to poor performance of PVDs in greater depths.
3. The closer spacing of the wick drains would likely have a less positive impact on the increase of shear strength. This could be due to a soil disturbance and should be studied closely prior to the recommendations on wick drain installation.
4. Although the wick drain installation is required to accelerate the primary consolidation settlement rate, its impact on shear strength should be evaluated by a pre- and post-installation in-situ testing to confirm the design shear strength values.
5. The results of the PDA testing indicated that the shear strength of the soil is within the estimated range from CPT. Additionally, since the calculated shear strength is mobilized from peak to residual during the pile driving, the anticipated in-situ peak shear strength should be higher than the values calculated from the PDA tests.

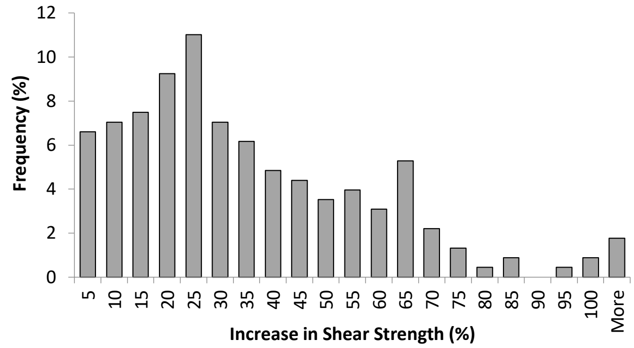

Figure 6. Percent increase of S_u after PVD installation in the central portion of the site

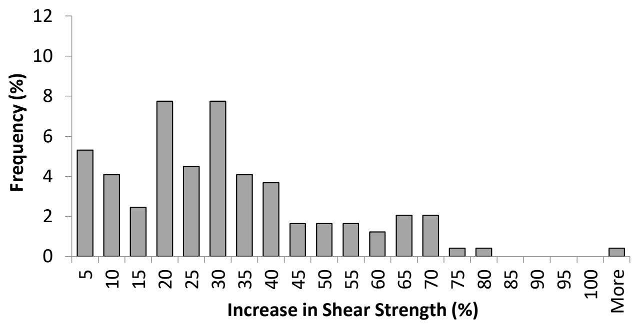

Figure 7. Percent increase of S_u after PVD installation in the northern portion of the site.

7 ACKNOWLEDGMENT

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8 REFERENCES

Bolton M.D., Sharma. J.S., 1994, Embankment with base reinforcement on soft clay, *Centrifuge*, Balkema, Rotterdam, 587-592.

Canadian Standard Association (CSA), *Commentary on CSA S6:19 Canadian Highway Bridge Design Code,* CSA Group, November 2019

Holtz, R.D., Jamiolkowski, M. B., Lancelotta, R., and Peroni, R., 1991. *Prefabricated vertical drains. Design and Performance.* Construction Industry Research and Information Association, CIRIA, London, 131 p.

Robertson, P.K., 1990. Soil classification using the cone penetration test. Canadian Geotechnical Journal, 27(1): 151-158.

Robertson, P.K., 2009. CPT interpretation – a unified approach, Canadian Geotechnical Journal, 46: 1-19

Schmertmann, J.H., "*Guidelines for cone penetration test, performance and design*", Report FHWA -TS -787 -209, Federal Highway Administration, Washington, July 1978

Sharma, J.S., Bolton, M.D., 2001, Centrifugal and numerical modelling of reinforced embankment on soft clay installed with wick drains, *Geotextiles and Geomembrane*, Elsevier, 19, 23-44.

Skempton, A. W. and Northey, R. D., 1952, “The Sensitivity of Clays” Geotechnique, Vol. 3, No. 1, pp 30 -53

Vesic, A. S., 1977, Design of Pile Foundation – *National Cooperative Highway Design Program Synthetic of Practice.* No. 42., TRB, Washington D.C.