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Prediction of Shaft Resistance of Large Diameter Bored Piles Socketed in Guadalupe Tuff Formation

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Abstract – *In this study, data analytics was utilized to gain insights into the geotechnical engineering properties of the Guadalupe Tuff Formation (GTF). The data came from both the in-situ and laboratory test results of 392 intact rock core samples extracted from the upper layer of the rock mass (0 ~ 30m). By organizing, processing, and analyzing the data, the descriptive statistics of the uniaxial compressive strength (q_u (kPa): m = 3,876, median = 3,496, Q1 = 2,500, Q2 = 5,115), Rock Quality Designation (RQD (%): m = 74, median = 75, Q1 = 60, Q2 = 90), dry unit weight (γ(kN/m^3): m = 16.8, median = 16.7, Q1 = 16.1, Q2 = 17.4), and elastic modulus of intact rock mass (Ei (kPa): m = 7,497, median = 7,500, Q1 = 5,000, Q2 = 10,000) of the GTF were determined. The shaft resistance of 38 large-diameter bored piles (2.0m–3.0m) was predicted using empirical, analytical, and theoretical methods and compared to mobilized unit shaft resistance determined by high-strain dynamic testing (ASTM D4945). Results suggest that the theoretical method underestimates unit shaft resistance in large-diameter piles socketed in the GTF. Similarly, while the analytical method provides closer predictions of ultimate shaft resistance, it also underpredicts actual values. To achieve economical pile design, developing an empirical method specific to the GTF is recommended. This can be achieved by creating a comprehensive database of load test results of large-diameter bored pile socketed in GTF.*

Keywords: *rapid thermal annealing, electrophoretic deposition, high-temperature superconductors, supporting electrolytes, superconducting films*

I. INTRODUCTION

Most of the significant infrastructures in Metro Manila are supported by large diameter bored piles socketed into the Guadalupe Tuff Formation (GTF), a thick sequence of well-stratified andesitic tuff, and tuffaceous sandstones, shales, and agglomerates located at the edge of the Central Physiographic Province of Luzon (Figure 1). GTF is composed of two parts, the lower Alat Conglomerate, and the upper Diliman Tuff [1].

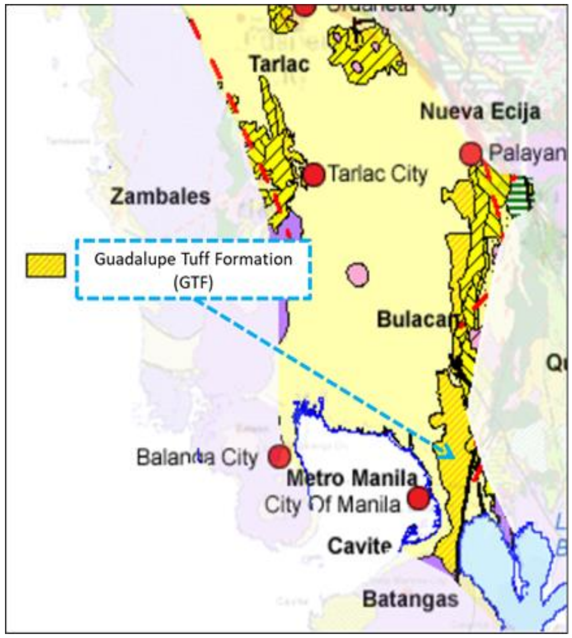

Figure 1. Central Physiographic Province showing the extent of the Guadalupe Tuff Formation

According to Reyes [2], GTF can be categorized as very weak rock or very hard or dense soil. Bored piles socketed into soft rocks are generally designed to carry the load by shaft resistance only. The end bearing capacity of the bored pile is usually neglected. For one, cleaning the base of the excavation is difficult. Second, a relatively larger settlement is needed to mobilize the ultimate end bearing as compared to the settlement needed to fully mobilize the shaft resistance. According to Shong, Ir. Liew Shaw [3], a base displacement of approximately 5%-10% of the pile diameter is generally required to mobilize the ultimate end bearing capacity provided that the base is properly cleaned and checked. For large diameter bored piles, around 2500 mm to 3500 mm in diameter, this ranges from 125 mm to 350 mm which is way larger than the typical allowable settlement of structures (25mm) as permitted by the commonly used structural codes.

At present, there are three different methods used to predict the shaft resistance of bored piles socketed in rock. These are empirical, analytical, and theoretical methods. Empirical methods are based on full-scale load tests in which the ultimate unit shaft resistance (f_s) is back calculated from the results of the instrumented load tests. The ultimate unit shaft resistance (f_s) is then correlated to the uniaxial compressive strength of the intact rock core (q_u) using empirical constants α and c which is different from author to author. Care and judgment should always be exercised when using this method since these empirical relationships were derived from a different geological setting.

$$\frac{f_s}{p_a} = \alpha\left(\frac{q_u}{p_a}\right)^c \tag{1}$$

Table 1. Empirical Constants Suggested by Different Authors

Authors	a	c
Horvath and Kenney (1979)	0.65	0.50
Carter and Kulhawy (1988)	0.63	0.50
Zhang and Einstein (1999)	1.26	0.50

On the other hand, analytical methods, which are often based on the results of finite element analysis, are generally like empirical methods in terms of form but additional factors were added to address the aspects neglected in the empirical methods for the sake of simplification in calculation. Several analytical methods have been proposed, but among all these, it was the one proposed by O'Neil and Reese that is widely used especially in highway and transportation design codes (AASHTO, FHWA, DPWH DGCS). According to O'Neill and Reese [4], the unit shaft resistance of bored piles socketed in rock may be expressed in the form:

$$\frac{f_s}{p_a} = 0.65\alpha_E\left(\frac{q_u}{p_a}\right)^{0.5} \tag{2}$$

The reduction factor α_E accounts for the degree of fracturing of the rock mass on which the bored pile is socketed to. This reduction factor is a function of the ratio of the elastic modulus of the rock mass (E_m) to the elastic modulus of the intact rock core (E_i).

Lastly, the theoretical method employs Brom's Method to determine unit shaft resistance along the bored pile length. The shear strength parameters of the rock are estimated using the Hoek and Brown Failure Criterion. However, since this method requires a thorough understanding of the geological characterization of the GTF, geotechnical engineers often just take the conservative approach and treat the GTF as dense to very dense sand with an internal angle of friction ranging from 38° – 40° and predict the ultimate shaft resistance using the equation below. Brom's is fundamentally based on effective stress and the friction between the interface of the surface of the bored pile and the surrounding soil.

$$f_s = K\sigma'_v \tan\delta' \tag{3}$$

In this paper, the shaft resistance of 38 large-diameter bored piles (2,500–3,500 mm), all socketed into the GTF to depths of at least three times the pile diameter (>7,500 mm), was estimated using empirical, analytical, and theoretical approaches discussed earlier. These predictions were then compared to the "mobilized" unit shaft resistance measured through high-strain dynamic testing (ASTM D 4945), commonly known as the Pile Driving Analyzer (PDA) Test. The location of the 38 bored piles can be seen in Figure 2.

In a PDA test, "mobilized" shaft resistance is the portion of ultimate shaft resistance along the pile shaft that is activated under dynamic loading. For bored piles, Hussein [5] recommends a drop weight of 1–2% of the ultimate load, while Rausche et al. [6] suggests a drop weight of 1% for shafts socketed in rock. For large-diameter bored piles socketed in GTF with typical capacities of around 40,000 to 60,000 kN, this requires a 40 to 60-ton impact device or a significantly higher drop height. An impact device as heavy as this is usually unavailable and uneconomical to fabricate while increasing the drop height significantly is unsafe and a nuisance especially when piles being tested is in the public areas, say along the road or highway which is often the case for infrastructure projects. Pile Dynamics, Inc. [7] indicates that a pile set of around 2–3 mm reflects full mobilization, with sets over 3 mm indicating failure-level loading or a dynamic test that corresponds to a static test run to failure. In this study, observed pile sets of around 2 mm confirmed measurement of “mobilized” shaft resistance, though not the full "ultimate" shaft resistance.

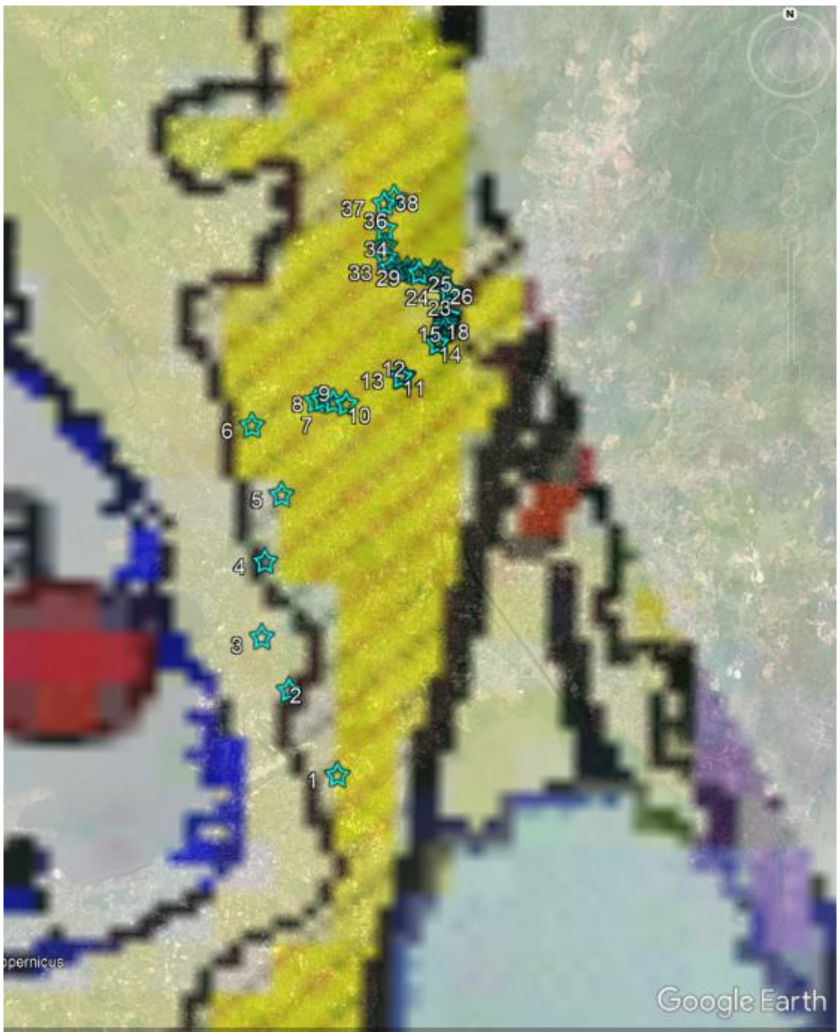

Figure 2. Location of large diameter bored piles tested with high-strain dynamic test

In this study, data analytics was employed to gain insights into the geotechnical engineering properties of the GTF. In-situ and laboratory test results of 392 intact rock core samples extracted from 100 boreholes drilled within the GTF. The borehole location plan can be seen in Figure 3. These boreholes were a part of the subsurface investigation campaign carried out for three significant infrastructure projects in Metro Manila named here as Project X, Project Y, and Project Z. Project X is a 5.5-kilometer segment of a toll road, strategically connecting the FTI to C6/Taguig. In contrast, Project Y is an elevated expressway extending approximately 18.83 kilometers, spanning from Buendia in Makati City to the North Luzon Expressway in Balintawak, Quezon City. Project Z, a rapid transit line, covers a length of roughly 22.8 kilometers and includes 14 stations. The line traverses in a northeast-southwest orientation, extending from San Jose del Monte in Bulacan to the North Avenue in Quezon City.

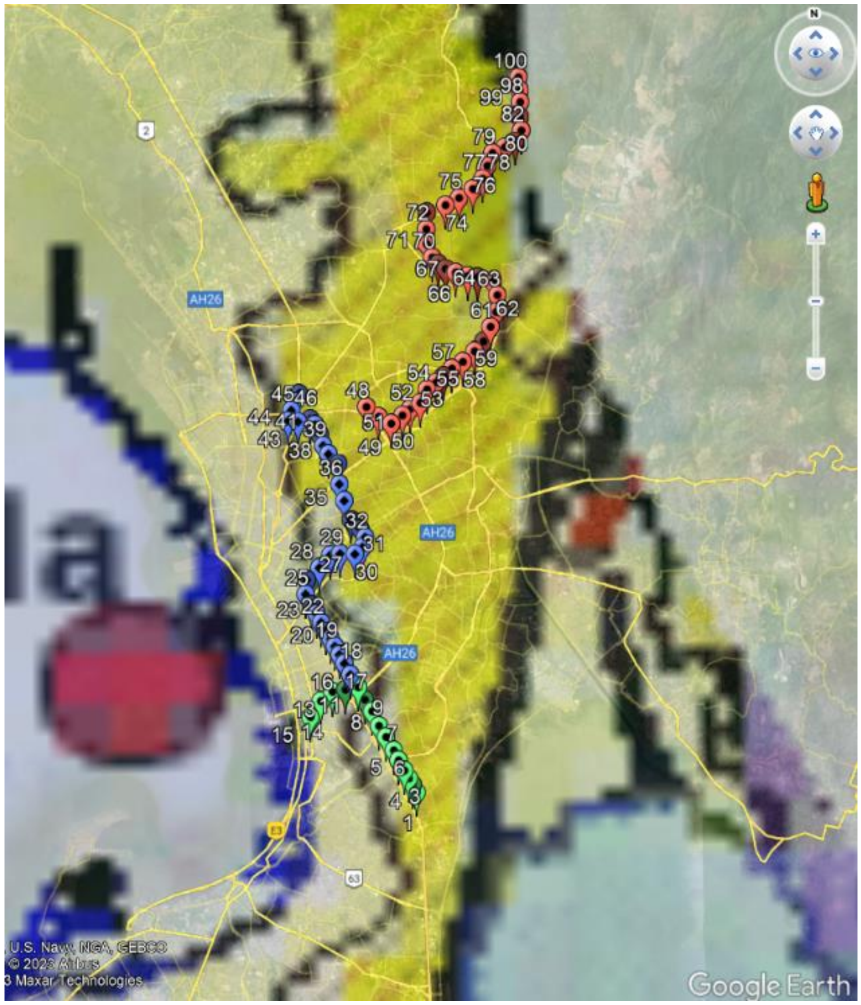

Figure 3. Borehole location plan superimposed on the GTF (Green = Project X, Blue = Project Y, and Red = Project Z)

II. ENGINEERING PROPERTIES OF GUADALUPE TUFF FORMATION (GTF)

2.1. Engineering Properties of Rocks

Based on the preceding discussions, to predict the shaft resistance of bored piles socketed in rock, the unconfined compressive strength (q_u), Rock Quality Designation (RQD), dry unit weight (γ), and elastic modulus of intact rock core (E_i) should be known.

Uniaxial compressive strength (q_u) refers to the maximum amount of compressive strength an intact rock core sample can sustain without failure when loaded along a single axis. The value of q_u provides an indication of the ability of the rock to withstand compressive forces (ASTM D2938)

Figure 4. Rock core sample failed in shear

RQD is a measure to assess the quality and integrity of a rock mass. It quantifies the percentage of intact and sound rock material within a core run (2000 mm). Higher RQD indicates that the rock mass is competent, whereas lower values indicate the possible presence of fractures, discontinuities, and planes of weakness. However, poor drilling techniques, core breakage upon handling, and improper drilling equipment can also yield to low RQD according to Pells et. al. [8]. Hence, prudence and judgment should be employed while reading RQD test results.

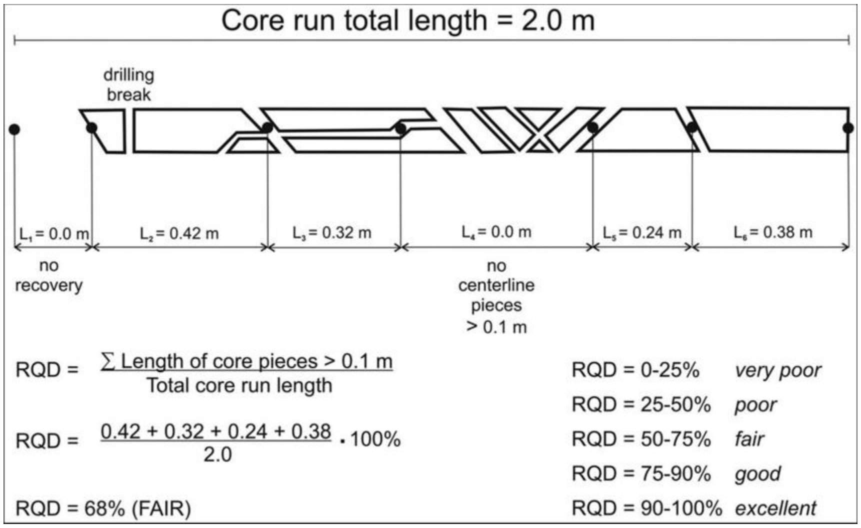

Figure 5. Calculation of RQD per ASTM D6032

The dry unit weight of rock, hereafter referred to simply as "unit weight" for the sake of simplicity, denotes the weight of a unit volume of rock material in a completely dry condition, devoid of any pore water. This parameter serves as a critical indicator of the rock's density and overall material composition. It provides valuable insights into the mineral composition, structural characteristics, and porosity of the rock, which in turn inform its mechanical behavior. The dry unit weight is essential in foundation design that is fundamentally based on the concept of effective stress.

Lastly, the elastic modulus is a measure of the stiffness and deformation characteristics of an intact rock core. It describes how much a rock will deform under applied stress and how will it recover once the stress is removed. The ratio of the elastic modulus of the rock mass (E_m) to the elastic modulus of the intact rock core (E_i) determines the reduction value α_E to be used in the analytical method of predicting shaft resistance as proposed by O'Neill and Resse [4].

Table 2. Estimation of α_E (O'Neill and Resse, 1999)

E_m/E_i	α_E
1.0	1.0
0.5	0.8
0.3	0.7
0.1	0.55
0.05	0.45

2.2. Uniaxial Compressive Strength (q_u)

The histogram of the uniaxial compressive strength (q_u) of all the intact rock core samples can be seen in Figure 6 below. The mean value (m) of the samples is 3,876 kPa and the data is positively skewed (Skewness = 0.52). Thus, the median is the most appropriate measure of central tendency to use when describing q_u since median is more resistant to higher outliers than the mean.

The boxplot of the uniaxial compressive strength for all the intact rock core samples can be seen in Figure 7. The minimum value is 1,539 kPa while the maximum value is 7,579 kPa. The Inter Quartile Range (IQR) is 2,615 kPa while the median value is 3,496 kPa. The mode of the sample is 4,533 kPa.

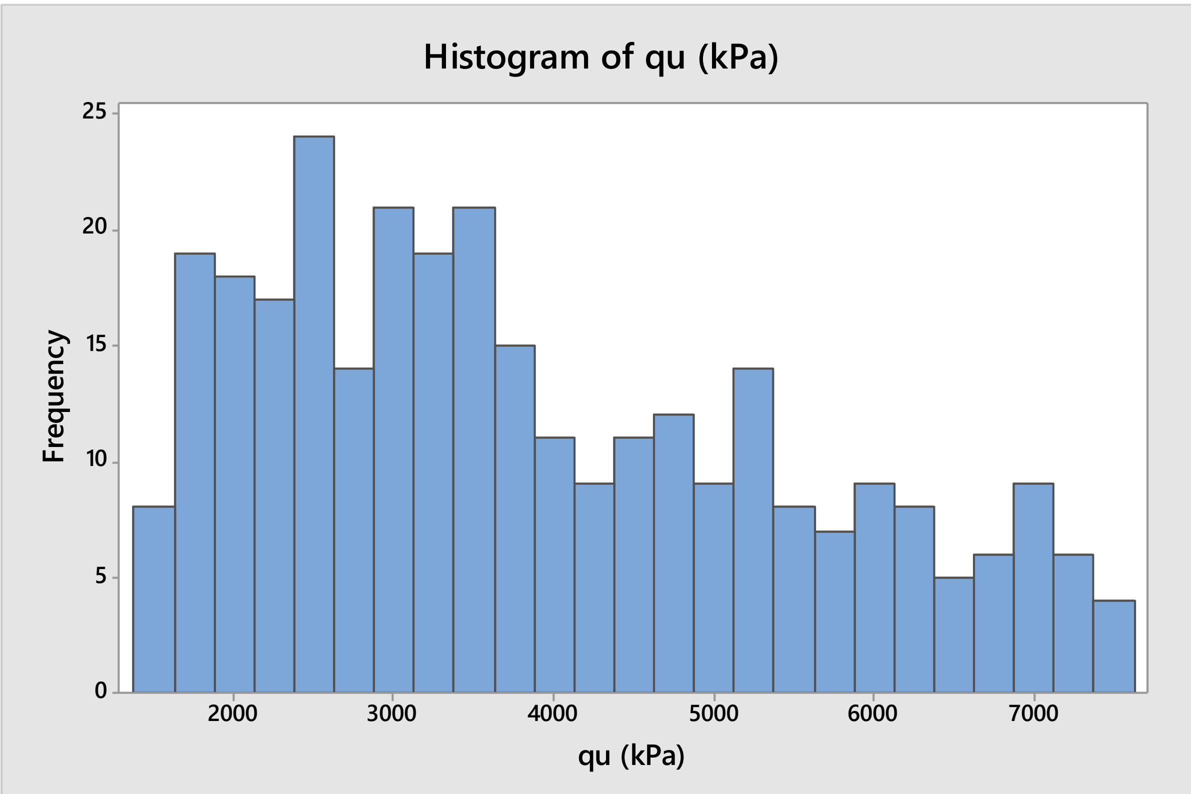

Figure 6. Histogram of the q_u (kPa) of all the samples

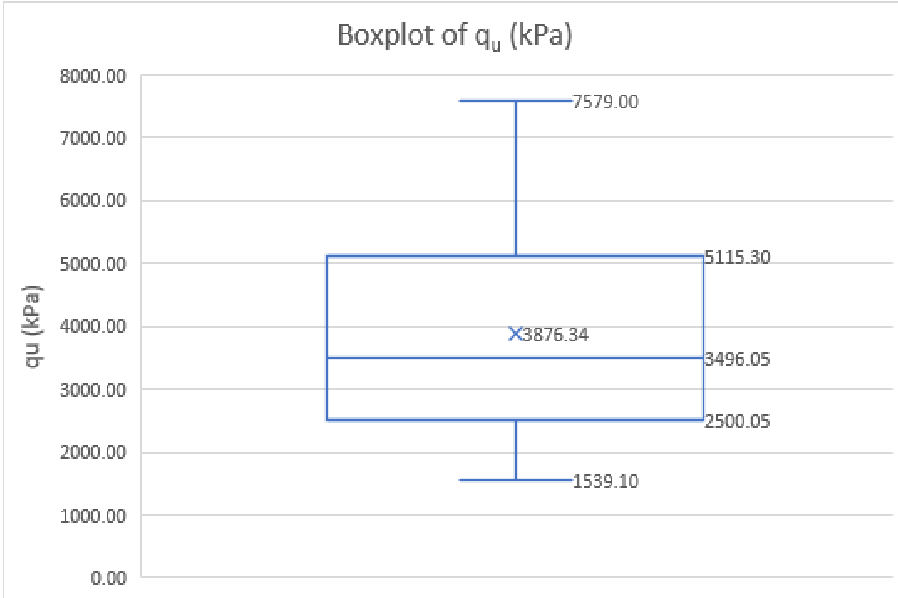

Figure 7. Boxplot of q_u (kPa) for all the samples

Analysis of the boxplots from the three projects indicates that the uniaxial compressive strength (q_u) exhibits a general increasing trend toward the northern region of the Guadalupe Tuff Formation (GTF). Most failures occur along specific planes aligned with the rock's internal structures, such as foliation, bedding planes, or visually observable cracks, leading to the development of shear fractures that propagate along a single plane. Axial and multiple splitting are infrequent.

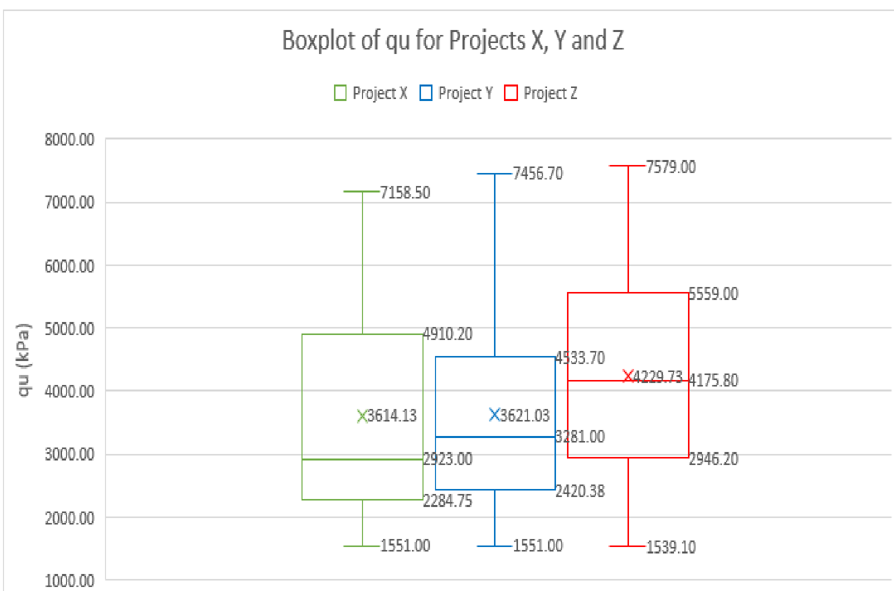

Figure 8. Boxplot of q_u (kPa) for the three projects

2.3. Rock Quality Designation (RQD)

The histogram of the Rock Quality Designation (RQD) of can be seen in Figure 9 below. The mean value (m) of the samples is 74% and the data is negatively skewed (Skewness = -0.71). Thus, it is more appropriate to use the median value to describe RQD. Since the median provides a central value that is not skewed by low outliers.

The boxplot of the Rock Quality Designation (RQD) for all the intact rock core samples can be seen in Figure 10. The minimum value is 15% while the maximum value is 100%. The Inter Quartile Range (IQR) is 30% while the median value is 75% which is very near to the mean of the sample. The mode of the sample is 100%.

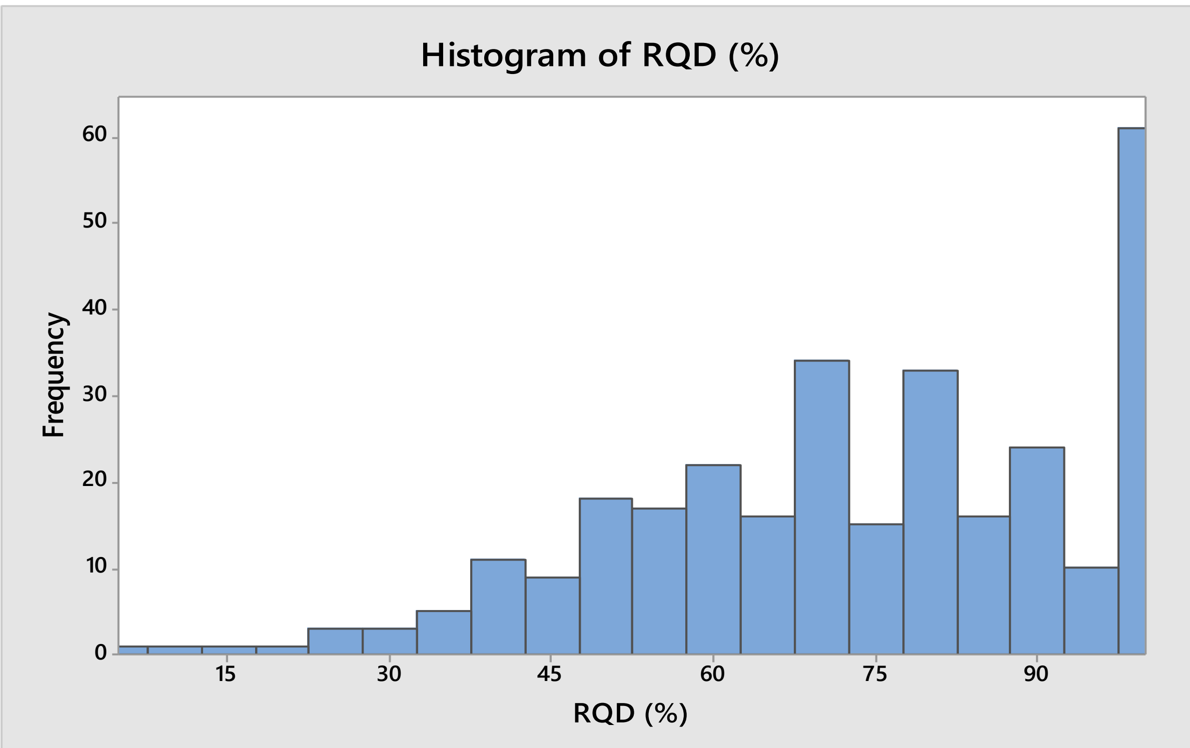

Figure 9. Histogram of the RQD (%) of all the samples

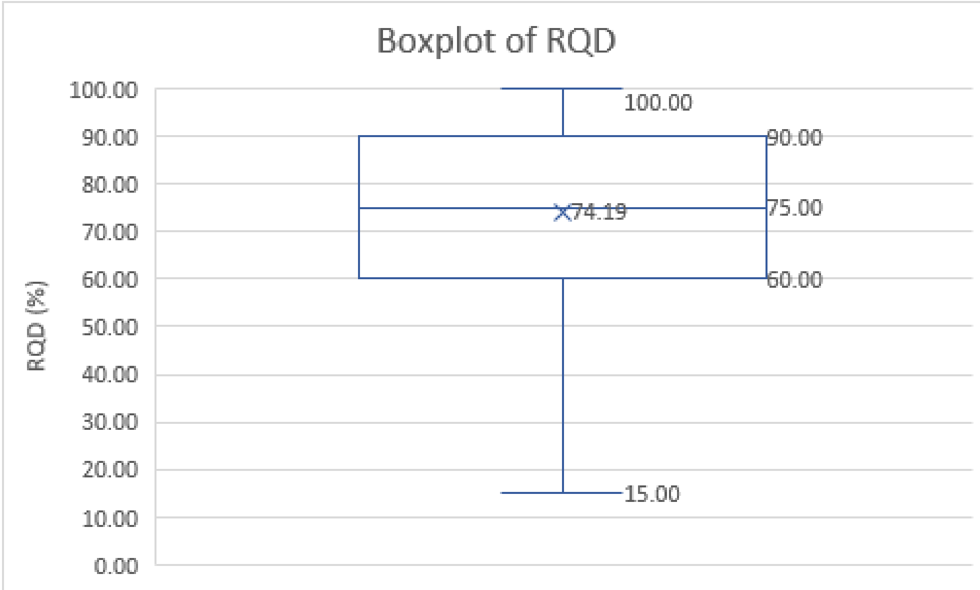

Figure 10. Boxplot of RQD (%) for all the samples

Analysis of the boxplots from the three projects indicates that the Rock Quality Designation (RQD) remains relatively constant, regardless of the borehole's position relative to the north. However, an increasing variation in RQD values is observed as one moves northward. It is important to note that these boreholes were drilled by different contractors and drilling crews, which may significantly influence the results due to variations in workmanship and drilling methodology

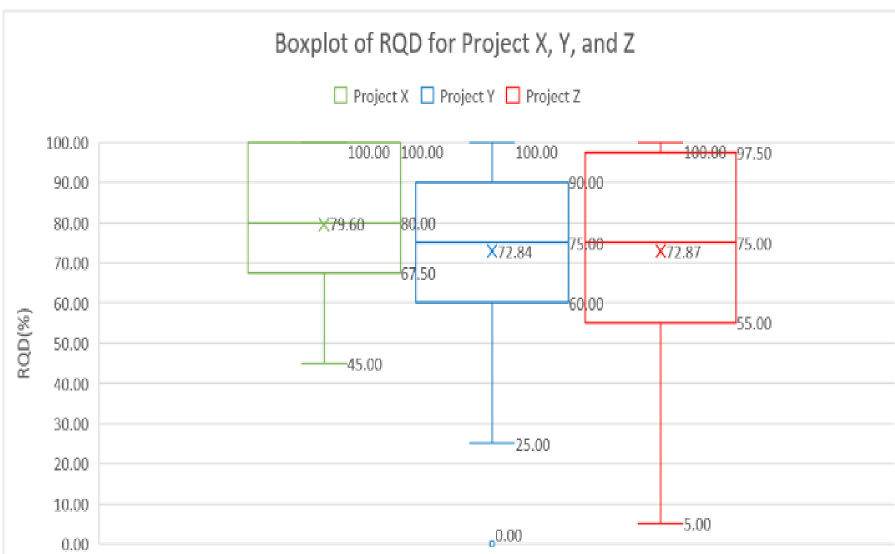

Figure 11. Boxplot of RQD for the three projects

2.4. Unit Weight (γ)

The histogram of the unit weight (γ) of the intact rock core sample of can be seen in Figure 12 below. The mean value (m) of the samples is 16.77 kN/m^3 and the data is practically symmetric or normal (Skewness = -0.002). Thus, it is more appropriate to use the mean value of the samples to describe the unit weight.

The boxplot of the unit weight (γ) for all the intact rock core samples can be seen in Figure 13. The minimum value is 14.21 kN/m^3 while the maximum value is 19.32 kN/m^3. The Inter Quartile Range (IQR) is 1.37 kN/m^3 while the median value is 16.74 kN/m^3 which is very near to the mean of the sample. The mode of the sample is 16.22 kN/m^3.

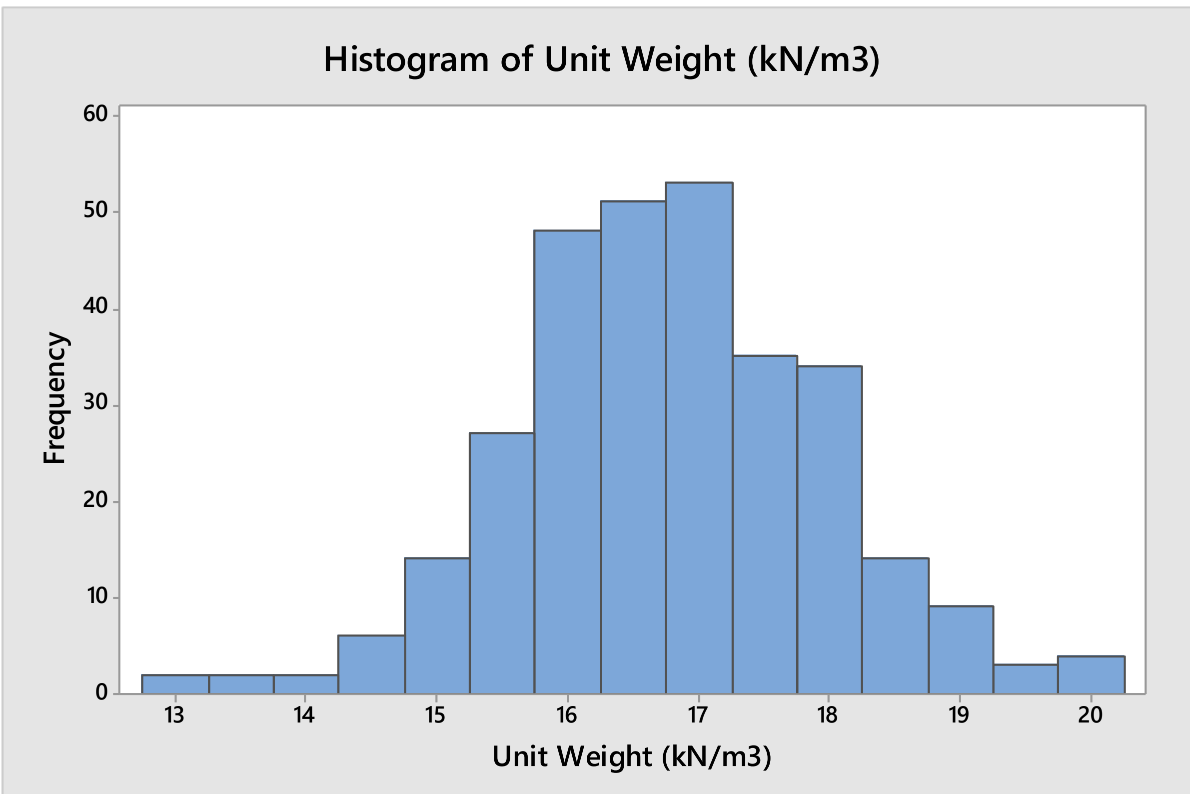

Figure 12. Histogram of the γ (kN/m^3) of all the samples

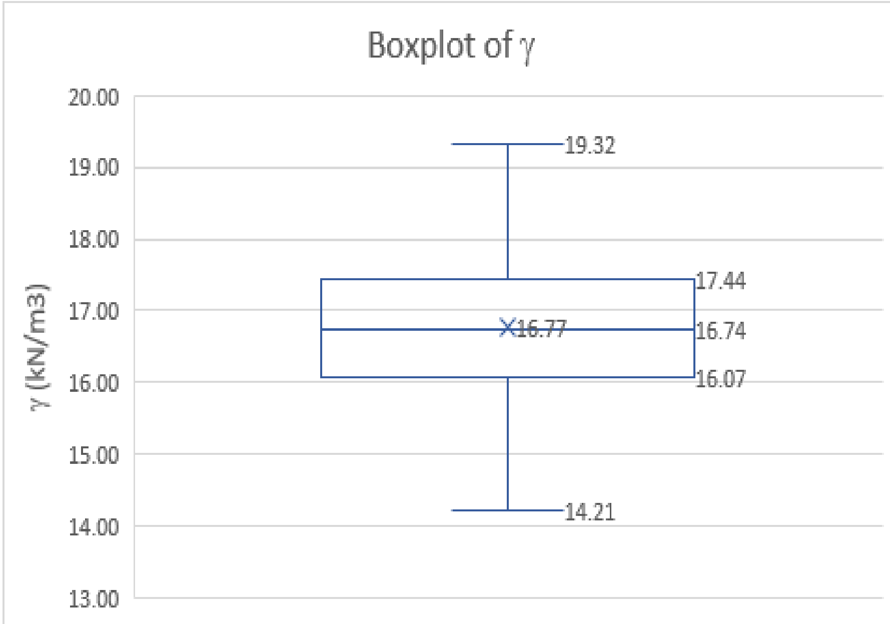

Figure 13. Boxplot of γ (kN/m^3) for all the samples

Analysis of the boxplots from the three projects reveals that the unit weight (γ) remains relatively constant regardless of the borehole's position relative to the north. However, there is a noticeable trend of increased variation in (γ) values as one moves northward.

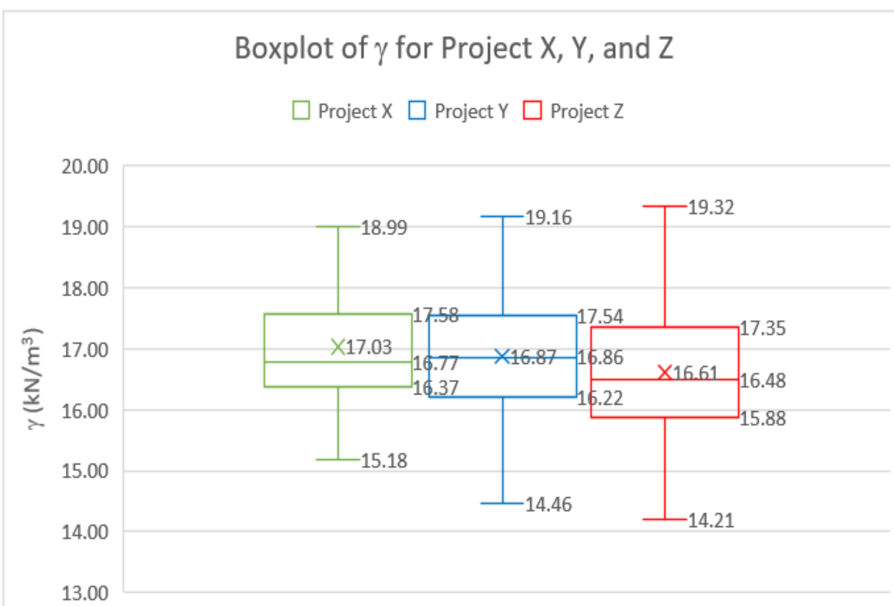

Figure 14. Boxplot of RQD for the three projects

2.4. Elastic Modulus of Intact Rock Core Sample (Ei)

The histogram of the elastic modulus (E_i) of the intact rock core sample can be seen in Figure 15. The mean value (m) of the samples is 7,497 kPa and the data is positively skewed (Skewness = 1.15). Therefore, using the median value to describe E_i is more appropriate, as it is less influenced by high outliers, providing a more representative measure of the sample's central tendency.

The boxplot of the elastic modulus (E_i) for all the intact rock core samples can be seen in Figure 16. The minimum value is 1,851 kPa while the maximum value is 17,500 kPa. The Inter Quartile Range (IQR) is 5,000 kPa while the median value is 7,500 kPa which is very near to the mean of the sample. The mode of the sample is 10,000 kPa.

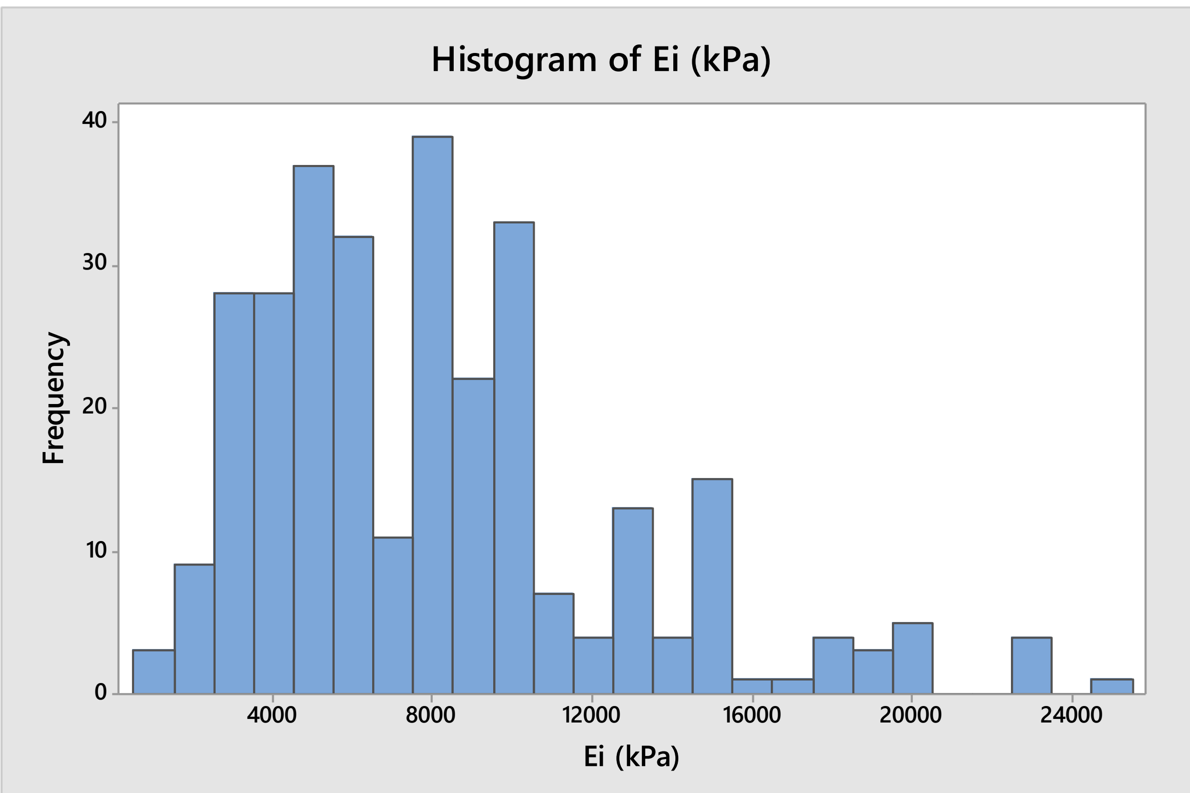

Figure 15. Histogram of Ei for the three projects

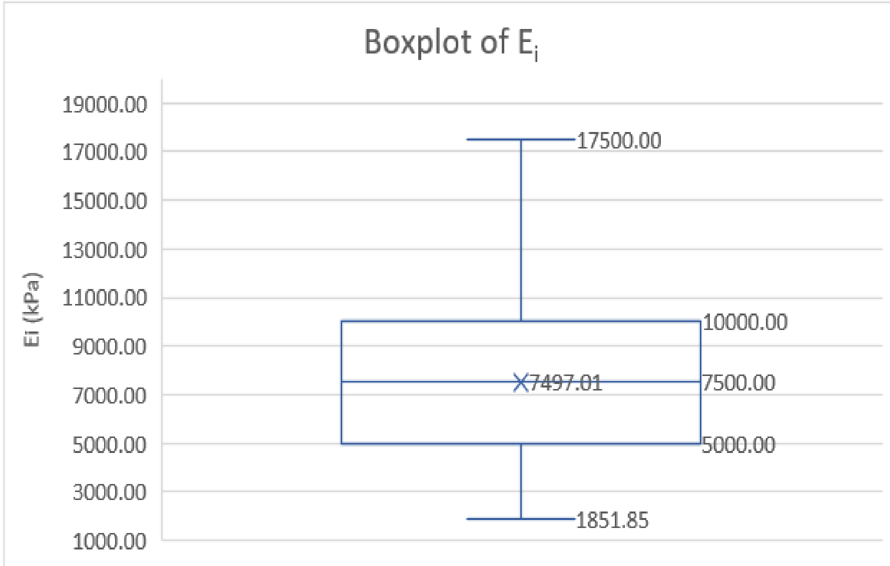

Figure 16. Boxplot of Ei for the three projects

Analysis of the boxplots of the three projects, indicates that in general, E_i increases going to the northern region of the GTF. Furthermore, we can see a trend of greater variation in E_i values as one moves northward.

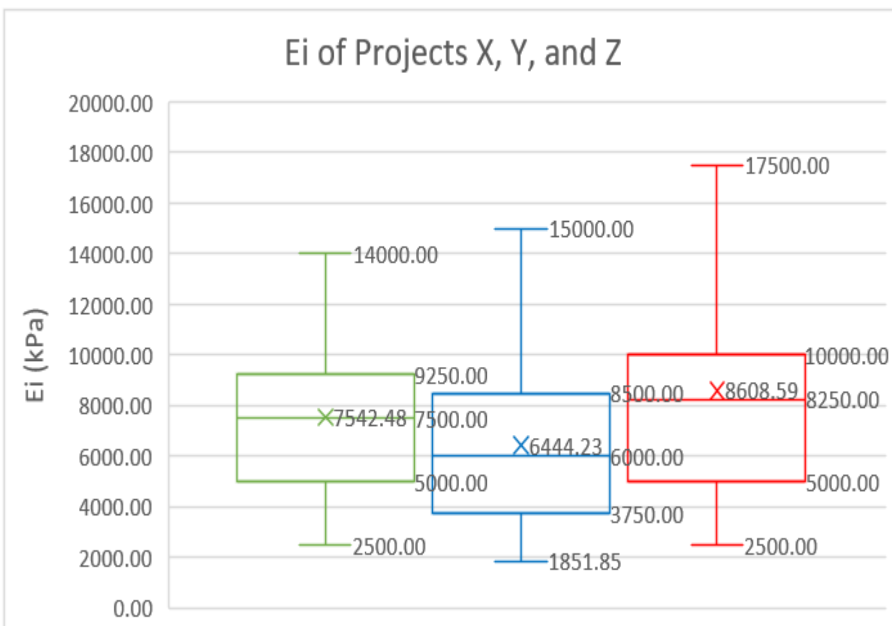

Figure 17. Boxplot of Ei for the three projects

Correlation analysis was performed between the depth of the intact rock core from the ground surface (Depth), uniaxial compressive strength (q_u), Rock Quality Designation (RQD), unit weight (γ), and elastic modulus of intact rock core (E_i). Apparently, there is no correlation that exists between the depth of the rock core and q_u, RQD, and E_i. But a very weak positive correlation exists between the depth of the rock core and γ ($r <+0.2$).

A very weak to weak positive correlation was found between γ and E_i ($r <+0.2$) and γ and q_u ($+0.2 < r < + 0.4$). On the other hand, there was a moderately strong correlation between q_u and E_i ($+0.4 < r < + 0.6$). The corresponding correlation coefficients for each variable can be seen in Table 3.

Table 3. Correlation Among the Depth of Rock Core Samples from the Surface of the Rock Mass (m), q_u (kPa), RQD (%), γ (kN/m^3), and E_i (kPa)

	Depth (m)	*qu (kPa)*	*RQD (%)*	*γ (kN/m3)*	*Ei (kPa)*
Depth (m)	1				
qu (kPa)	0.030295	1			
RQD (%)	-0.05347	-0.00492	1		
γ (kN/m3)	0.122221	0.252772	0.030227	1	
Ei (kPa)	-0.02306	0.586803	-0.06343	0.14733	1

III. BORED PILE SHAFT RESISTANCE

3.1. High-Strain Dynamic Test (ASTM D4945)

According to Robinson et. al., [9], since the early 1970s, dynamic load tests have been routinely conducted on hundreds of construction sites around the world using a Pile Driving Analyzer (PDA) as a quality assurance measure of driven piles. But since the end of the 1970s, these tests have also been more and more frequently employed for the bearing capacity assessment of cast-in-place piles and bored piles and these results have been correlated to static load tests. A very extensive correlation test series was conducted in 1982 in Melbourne, Australia by Seidel and Rausche [10] on 12 shafts of 1.5 m diameter and 60 m length. After good correlations had been established, approximately 100 additional dynamic pile tests were performed at this site. Subsequent studies conducted by numerous researchers have established further correlations between static and dynamic load test results. These findings made the dynamic testing a more economical, faster, and safer alternative to the traditional static load test setups- such as kentledge or reaction frames- which are both difficult and costly to construct. A typical set-up of a High-Strain Dynamic Test/ PDA Test can be seen in Figure 18 below, and this is based on ASTM D 4945 [11].

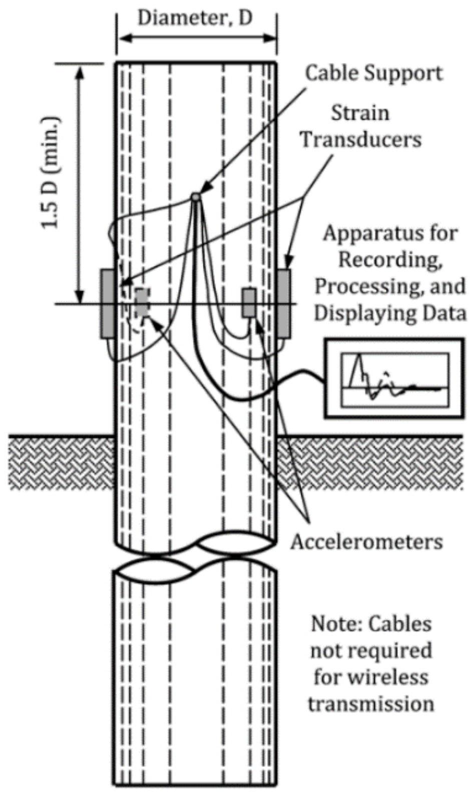

Figure 18. Test set up of High Strain Dynamic Test (ASTM D 4945)

The PDA Test requires an impact load generated by an impact device striking the top of the pile during dynamic testing. During the impact loading, measurements of pile top force and velocity will be taken using the strain transducers and accelerometers connected to the Pile Driving Analyzer (PDA) device. An analysis will then be conducted off-site to reduce the dynamic force and motion measurements to a static load and settlement curve. The most employed software for this purpose is CAPWAP®, a proprietary signal matching software developed by Pile Dynamics, Inc. CAPWAP utilizes an elastic pile model alongside static, elastoplastic, and viscous dynamic soil models to correlate computed and measured signals through a trial-and-error matching procedure [6]. Match quality should be kept to less than 5 [12].

Several factors can significantly influence the results of high-strain dynamic tests on bored piles. According to Pells [13] these factors include the integrity of the pile, the condition of the surrounding soil, the adequacy of the test setup, the weight of the impact device, the precision of instrumentation and data acquisition, and the subsequent interpretation of test data, including signal processing techniques. In this study, the 38 bored piles selected were free from any significant defects, tested by a single contractor, and utilized the same impact device—a 50-ton drop hammer specially fabricated by the contractor of the three projects under study. The actual assembly and fabrication drawings are illustrated in Figure 19. The assembly consists of a structural steel frame equipped with an automatic winch capable of lifting the stacked and welded plates, measuring 1.5 m x 1.5 m x 2.85 m, to a height of up to 2 m.

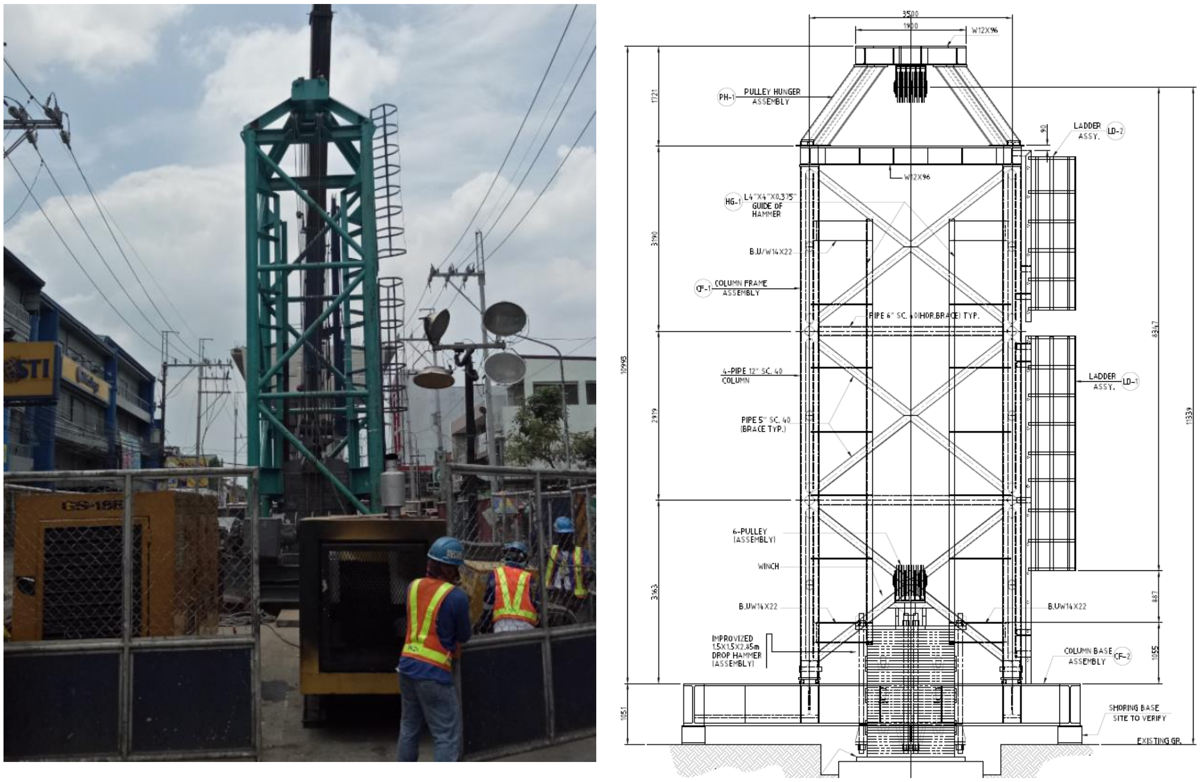

Figure 19. 50-T fabricated impact device used in the 38 High-Strain Dynamic Test performed

The dynamic test results were analyzed alongside supplementary data, including soil investigation reports, bored pile concrete pouring records, and other quality control documentations. This approach was used to ensure the reliability and accuracy of the results, minimizing the influence of external factors such as structural integrity issues within the piles. The values of the match quality (MQ) of the CAPWAP results of the 38 bored piles are all less than 3.0. Indicating that there was a good agreement between the measured pile response and the results of wave equation modeling. However, the recorded pile set for all 38 bored piles was approximately 2 mm or less, indicating that only a portion of the ultimate shaft capacity was mobilized during dynamic testing. Consequently, the measured shaft resistance can be considered a conservative estimate of the pile's ultimate shaft resistance. And in general, even when the pile set exceeds 3 mm, where the PDA Test results can already be considered equivalent to a Static Load Test (SLT) to failure, the unit shaft resistance measured is still lower than the measured unit shaft resistance using SLT to failure. Based on the 1996 database of dynamic testing and static load testing results collated by Likin and Rausche [14], as can be seen in Figure 20 below, the dynamic capacity of piles determined through CAPWAP typically does not exceed the ultimate pile capacity measured by Static Load Testing (SLT), often reaching around 80% of the SLT failure load only.

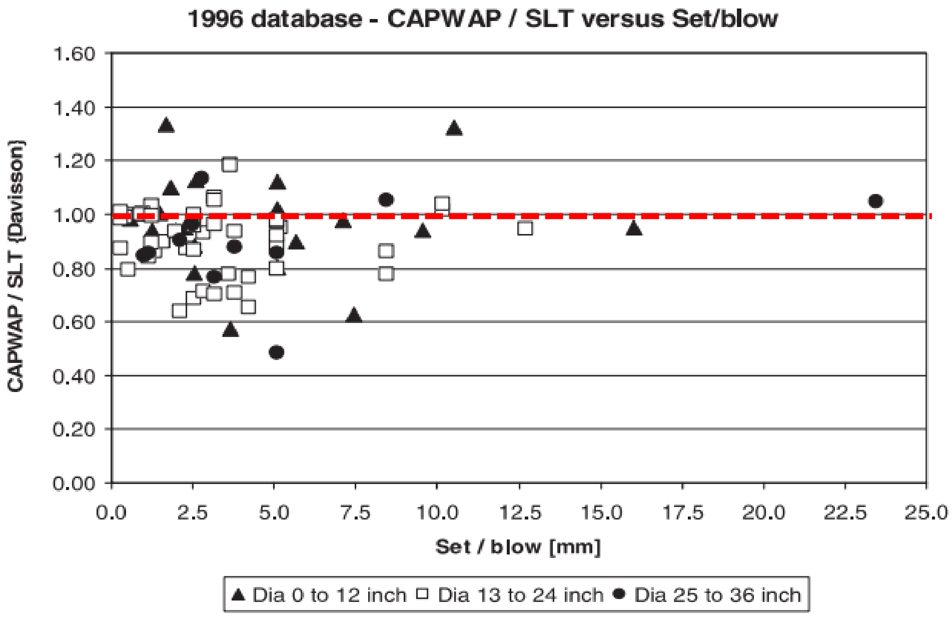

Figure 20. Ratio of CAPWAP to SLT prediction versus permanent set per blow

3.2. Prediction of Shaft Resistance

The process of predicting the shaft resistance of the 38 bored piles followed the flowchart in Figure 21. First, the field and laboratory test results of the rock core samples extracted in or near (< 5m) the location of the bored pile was collected. Second, these parameters were organized and analyzed to determine q_u, RQD, γ, and E_i which are required inputs in the empirical, analytical, and theoretical prediction of shaft resistance. Shaft resistance was then predicted using different methods, but parallel to this, the results of the high-strain dynamic testing and pile integrity testing of the bored pile were analyzed together with other peripheral data such as soil investigation reports and bored pile quality records. Only bored piles, whose CAPWAP Match Quality is less than 3 and with impeccable integrity test results were included in the study. Third, the results of the empirical, analytical, and theoretical methods were then compared to the mobilized shaft resistance determined through PDA Test. The results can be seen in Figure 22.

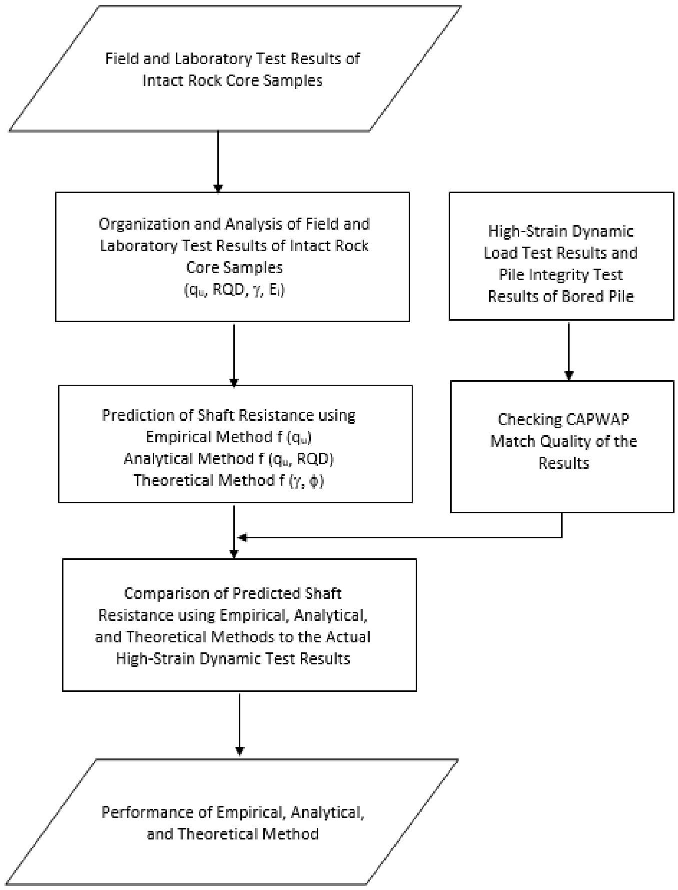

Figure 21. Flowchart of the prediction of shaft resistance using different methods and comparing the results with the results of High-Strain Dynamic Test

The unit shaft resistance obtained through empirical, analytical, and theoretical methods corresponds to the ultimate shaft resistance, representing the resistance of the shaft at failure. In contrast, the unit shaft resistance measured by the PDA Test—given that the pile set is less than 2 mm—reflects only the "mobilized" shaft resistance, which is just a fraction of the ultimate shaft resistance. Consequently, when the unit shaft resistance determined by empirical, analytical, and theoretical methods is lower than the values obtained from the PDA Test, it indicates a conservative estimate of the actual unit shaft resistance. Such methods, therefore, may provide a conservative estimate of the shaft capacity and utilization of which needs to be further evaluated if an economical pile design is desired.

In this study, the average ratio between the unit shaft resistance obtained using theoretical method and the "mobilized" shaft resistance measured using dynamic testing is approximately 0.46, indicating that the theoretical method underestimates the unit shaft resistance of the piles, see Figure 22f for details. Conversely, the analytical method proposed by O'Neil and Reese (1999) yielded an average ratio of 1.05 as can be seen in Figure 22d and Figure 22e. This is irrespective of the consideration on the jointing condition of the GTF (whether open or closed joints). Therefore, while the analytical method offers improved accuracy over the theoretical approach, it still appears to underpredict the actual unit shaft resistance, suggesting that both methods will yield to an uneconomical pile design, as far as axial pile capacity is concerned. Empirical methods, on the other hand, yielded higher average ratios as can be seen in Figure 22a to Figure 22c, of 1.37 for Horvath and Kenney (1979), 1.32 for Carter and Kulhawy (1988), and 2.66 for Zhang and Einstein (1999). These empirical methods warrant further evaluation through static load testing to failure or dynamic load testing with larger pile sets (greater than 3 mm). While potentially more suitable for the GTF formation, the author advises caution in their application. It is essential to verify that the ultimate shaft resistance estimated by these methods can be confirmed through a proof load test, meaning a sufficient logistical and budgetary resources must be available to substantiate the existence of such unit shaft resistance.

The average unit shaft resistance determined by CAPWAP in this study is approximately 220 kPa on average, though the actual unit shaft resistance may exceed this value, with estimates by some pile testers suggesting it could reach 500 kPa or even higher.

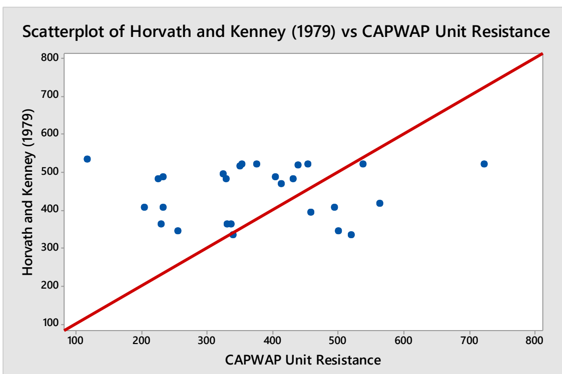

Figure 22a. Horvath and Kenney

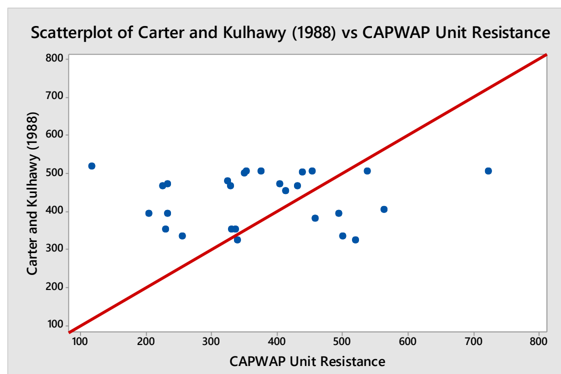

Figure 22b. Carter and Kulhawy

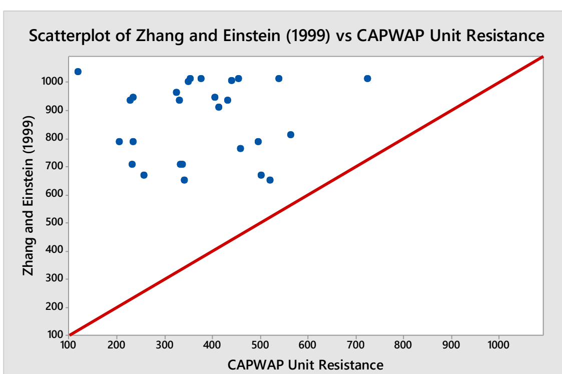

Figure 22c. Zhang and Einstein

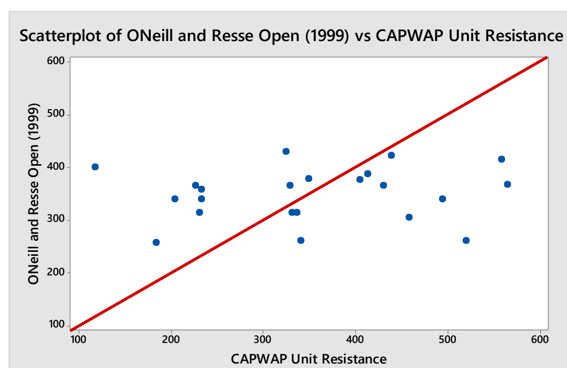

Figure 22d. O'Neill and Reese (Open)

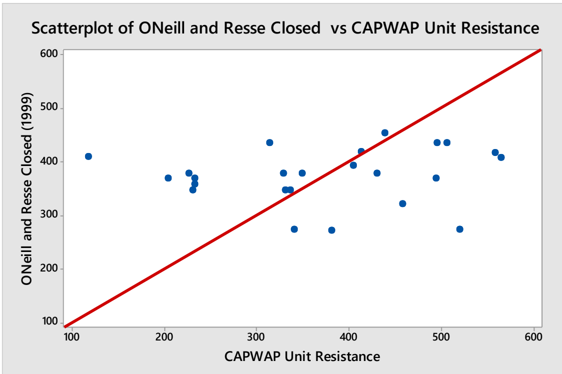

Figure 22e. O'Neill and Reese (Closed)

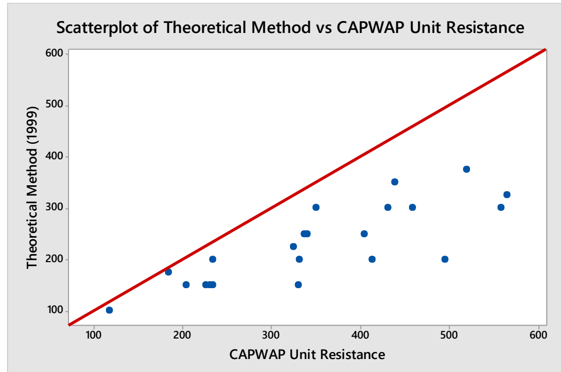

Figure 22f. Theoretical Method by Brom's

IV. CONCLUSION

From the 392 intact rock core samples analyzed, we were able to determine the geotechnical engineering properties of the upper 30m of the Guadalupe Tuff Formation (GTF). The descriptive statistics of the uniaxial compressive strength (q_u (kPa): m = 3,876, median = 3,496, Q1 = 2,500, Q2 = 5,115), Rock Quality Designation (RQD (%): m = 74, median = 75, Q1 = 60, Q2 = 90), unit weight (γ (kN/m^3): m = 16.8, median = 16.7, Q1 = 16.1, Q2 = 17.4), and elastic modulus of intact rock mass (Ei (kPa): m = 7,497, median = 7,500, Q1 = 5,000, Q2 = 10,000) were determined using data analytics. Additionally, findings indicate that there is no strong correlation between the depth at which the rock core was extracted and the geotechnical engineering properties q_u, RQD, γ, and E_i. And there is no strong correlation observed between the properties q_u, RQD, γ, and E_i confirming the earlier study made by Reyes [2]. Lastly, it was observed that in general, as you go northwards, the value of q_u increases noticeably.

This study examined empirical, analytical, and theoretical methods for predicting the shaft resistance of large-diameter bored piles socketed in the GTF. The findings indicate that the theoretical method tends to underpredict the unit shaft resistance of these piles. Similarly, while the analytical method offers a more accurate prediction of ultimate unit shaft resistance, it still underestimates the actual shaft resistance. To achieve a more robust and economical pile design, an empirical method specific to the GTF is recommended to be developed, like those proposed by Horvath and Kenney (1979), Carter and Kulhawy (1988), and Zhang and Einstein (1999). This development can be supported by building a comprehensive database of pile tests performed in large-diameter bored piles, incorporating both Static Load Test to failure and dynamic pile testing with larger pile sets (greater than 3 mm).

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