INTERNATIONAL JOURNAL OF
COASTAL & OFFSHORE ENGINEERING

IJCOE Vol.6/No. 4/Autumn 2021 (33-43)

Available online at http://www.ijcoe.org

Pile Length Optimization in Fixed Template Offshore Platform Using Risk Reduction Approach

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ARTICLE INFO

Article History:
Received: 10 Oct. 2021
Accepted: 28 Nov. 2021

Keywords:
Jacket-Type Offshore Platform
Pile Drivability Analysis
Risk Reduction Approach
Pile Length
Bearing Capacity

ABSTRACT

The purpose of risk management is managing the uncertainties by considering activities for identifying, assessing, monitoring, and reducing the impact of risks. Three strategies may be used to deal with the kind of risks that exist in projects: risk acceptance, risk transfer, and risk reduction. Events that can affect the economical goals of a project must be identified and evaluated so that they can be appropriately managed. Fixed jacket-type offshore platform (JTOP) as an expensive and necessary structure in energy facilities. in this research, the effect of knowledge increasing on the risk reduction and cost optimization for JTOP is studying. This paper focuses on optimizing the pile length of the fixed jacket-type offshore platforms and reducing the conservative design by using the risk reduction approach. Fixed offshore platform in South Pars Gas Fields of Iran as a case study.Increasing the Geotechnical knowledge and reducing the pile lengths is performed as considering similar geotechnical study at this regions and pile dynamic driving test (PDA), updating the pile bearing capacity base on increased knowledge for geotechnical data, and finally assessing the result based on inplace analyzing Pile driving result shows increasing the longterm soil bearing capacity, So first of all the required strength and parameters extracted from the existing data with analyzing and comparing where to adjust and matches with the lower limit of the theoretical equations. Finally, this new assumption is used for optimizing the pile length design. This research shows that that the numerical analysis and assumptions that have been used in the design procedure are conservative and a proper risk management program with the knowledge increasing could have resulted in risk reduction. The analysis process that has been used in the present research leads to the pile cost reduction by 11% that is considerable for stakeholders in such an expensive structure. The most important innovation in this paper is the use of the results of pile driving operation for optimal pile design because, in pile driving operation, piles with design diameter are used.

1. Introduction

Jacket-type offshore platforms are fixed-base offshore structures that are used to produce oil and gas in relatively shallow water. Driven, open-ended, steel pipe piles are typically used to support this type of structure as a foundation. A significant finding from the performance of jacket platforms in major hurricanes, including Andrew (1992), Roxanne (1995), Lili (2002), Ivan (2004), Katrina (2005), and Rita (2005), is that the pile foundations hav performance better than expected (e.g., Aggarwal et al. 1996, Bea et al. 1999 and Energo 2006 and 2007) [1]. Assessment of jacket platforms subjected to greater environmental loads than their design loading has indicated that the pile foundation often governs the capacity of the structural system and the majority of damages and main failures have been observed in the structureal elements above the mud-line level. While the lack of observing foundation failures may be acceptable, but the concern is potential of conservatism from economical point of view. The conservative design pf foundation can lead to costly construction of new jacket platforms or unnecessary limitations on the manning and production levels of existing jacket platforms. In this research, the long-term bearing capacity of piles at the site of the South Pars Gas Field (Phase 22 jacket-type offshore platform) will be assessed and reviewed according to the technical documentation and the results have been compared with similar jacket platforms in the South Pars Gas Field of Iran. The geotechnical and pile drivability reports and dynamic pile driving analyzer (PDA) test results used as a technical reference for the JTOP of phase 22. According to this information, the pile length of the mentioned platform is 97 and 105 meters.

There are three strategies that can be used to deal with the types of risks that projects face: risk acceptance, risk transfer, and risk reduction. Risk reduction typically focuses on the management of vulnerabilities and the rational and appropriate use of operational and technical information. The challenge is that there is a great deal of uncertainty in estimating pile capacity under both axial and lateral loadings. To accounting this uncertainty, there are numerous assumptions used in design practice that tend to be conservative, so that uncertainty does not lead to an unexpected failure. These assumptions are clear in design practice and act in addition to the design factor of safety that is applied to the design capacity.

The following summary of potential sources of conservatism has been extracted from the studies by Tang and Gilbert (1992) [2], Murff et al. (1993) [3,4], Pelletier et al. (1993) [5], Aggarwal et al. (1996) [6], and Bea et al. (1999) [7]. In addition, Andrew (1992) and Roxanne (1995) hurricanes have provided the motivation for presenting this data.

1. Sampling and testing methods used for site investigations.
2. Time Effects: Long-term setup or aging for piles installed in clays and sands may increase the foundation capacity with time.
3. Rate-of-loading effects.
4. Strain-softening and cyclic degradation.
5- The greater safety margin for foundation versus Structure in Design standards: the properties of the most critically loaded pile are considered for other piles designs, and all supporting piles of jacket platforms often have the same diameter and thickness; in some cases, there may be up to two different pile designs for a structure. On the contrary, structural members are usually designed and sized separately based on the applied expected forces in each member considering the entire structural system. Therefore, structural members are generally more optimized than piles, and the potential conservatism due to the design practice for piles is higher than that for the structure.

In summary, there are different sources of uncertainty involved in estimating the capacity of offshore piles, and this uncertainty has understandably led to a conservative design method. Factors that could reduce the capacity are generally included in the design, while elements that could increase the capacity are usually neglected in design. Therefore, failure is not necessarily expected if a pile foundation is loaded to its design capacity, particularly for piles loaded axially in sand layers and laterally in clay layers. However, normal practice desing procedure cause the axial or lateral capacity of a pile foundation were much more than twice the design value and that is noticabble [1]. Therefore, the main goal of this studyis optimizing the design by using a post analysis of available information in order to reduce risk and increase the benefits of stakeholders.

with regard to this goal, comprehensive studies have been carried out based on the 10 geotechnical and pile driving reports of the South Pars Gas Field of Iran that are included phases 13, 17, 18, 22, 23, and 24. Additionally, in all implemented projects, the results of pile driving operation are never used, while pile driving are much closer to reality, because it is done with piles with design diameter and the soil properties. So, the optimal design in this study is based on the results of pile driving operations.

2. Jacket-Type Offshore Platform Properties

In this study, SPD22 as JTOP where are located in the South Pars Gas Field of Persian Gulf-Phase-22 was investigated. The API RP 2A-WSD (2017) standard wich is most practical design guide is used[8], has been used. The height of the working point from the mud-line is 65.5 meters and the height of the upper level of the topside is 92.325 meters from the mud-line. The SPD22 jacket properties are as follows: Four-legged jacket with four main piles to support wellhead production facilities. Row 2 is a single batter at 1:7, and Row 1 is a double batter at 1:8 and 1:7. Legs Spacing is 13.716 m × 20 m at the working point and 29.237 m × 34.83 m at the mud-line level. Water depth is 65.60 m below LAT, and the upper deck is 26.65 m above the mean water level. The leg diameter is 1.655 m × 0.056 m, and through the leg, the pipe pile dimension is 1.524 m × 0.076 m. Pile penetration is 97 and 105 m. The weight of the topside is 3200 tons. The schematic view of the jacket platform and soil layers is depicted in Figure 1.

Inputs of the model have included the structural properties of all members, connections, piles, the soil surrounding the piles (i.e., t-z, p-y, and Q-z curves), and the environmental loading including the magnitude and directions of waves, wind, and currents, that are considered in the modeling. The three-dimensional finite element model of the structure and its foundation, was modeled by SACS software [9]. The structural model for jacket Platform SPD-22 that was developed in SACS™ is shown in Figure 2.

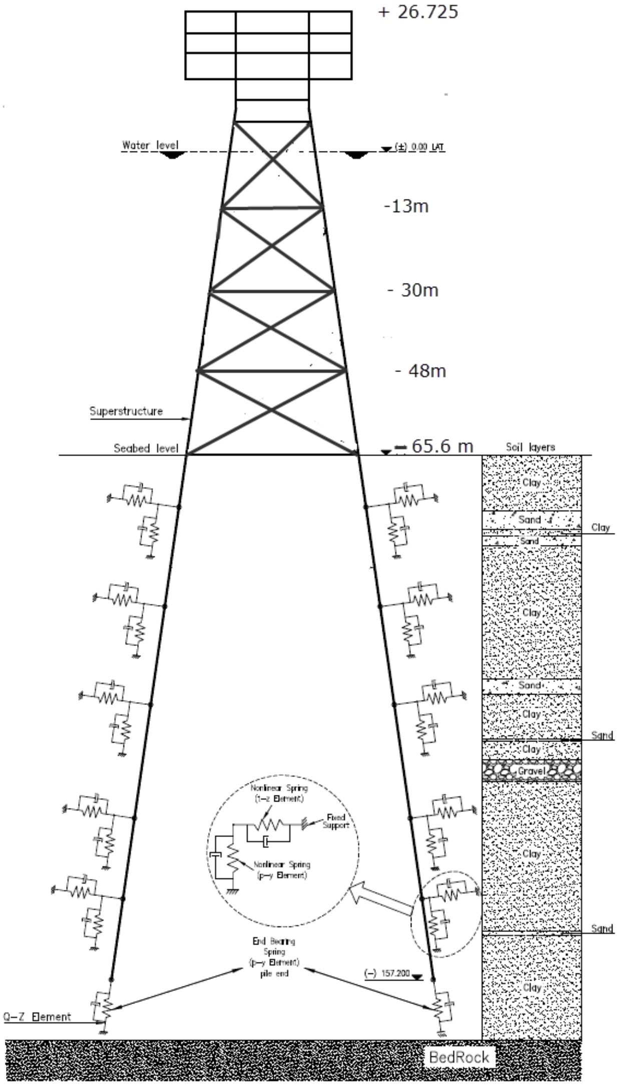


Figure 1. Schematic view of JTOP (SPD 22) and soil layers

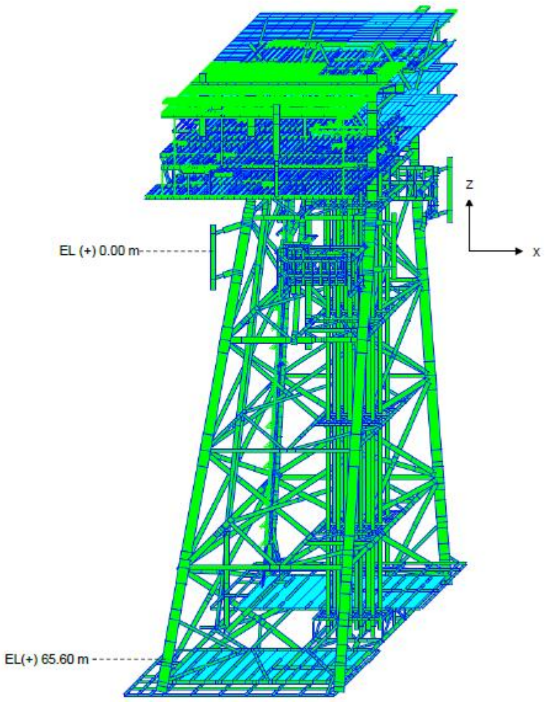


Figure 2. SPD 22 wellhead platform sacs model

According to the information presented in reports of the South Pars phases, the overall geotechnical stratum layer consists of soft clay sediments in depth throughout the area. As the depth increases, it rises from stiff clay deposits to very stiff ones and sandy layers with low to moderate cementation that has been reported in Table 1.

Table 1. Soil layer properties

Soil type (Figure 1)	Soil layer depth (m)
Clay	0-10.6
Sand	10.6-14.1
Clay	14.1-15.2
Sand	15.2-17.2
Clay	17.2-42.2
Sand	42.2-45.1
Clay	45.1-53.5
Sand	53.5-54
Clay	54-57.5
Gravel	57.5-61.6
Clay	61.6-89.8
Sand	89.8-90.65
Clay	90.65-110.4

3. Methgof of Risk Reduction Approach

One way to meet risk reduction is to incearse the knowledge of the problems. In this study, we are going to reduce the risk by increasing soil condition data in this area. The chosen area is south pars gas field and includes several jacket platforms. The jacket information of the table 2 has been used in this research.To confidence maintain, the names of geotechnical companies are not mentioned. A satellite image of the South Pars Gas Fields location is shown in Figure 3.

Table 2. The name of jacket platforms

Jacket Platform Name	Geotechnical Company
SPD-13A	A
SPD-13B	A
SPD-13C	A
SPD-13D	A
SPD-22	A
SPD-23	A
SPD-24A	A
SPD-24B	A
SPD-17 (25)	B
SPD-18 (26)	B

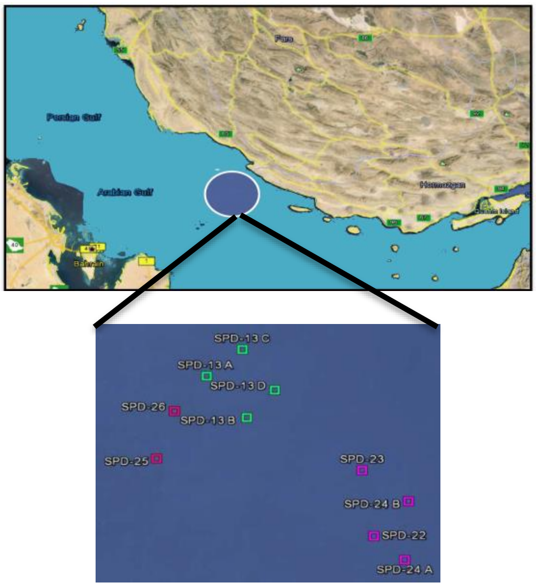


Figure 3. The location of the South Pars Gas Field in the satellite image

The risk reduction procedure that has been used has been shown in the following flowchart (Figure 4). The structure of jacket platform was modeled, risk reduction alternativese was chaecke and proofed, system and related component was evaluated, at the end, the performance goals in light of risks of jacket platform system was clarified and revised.

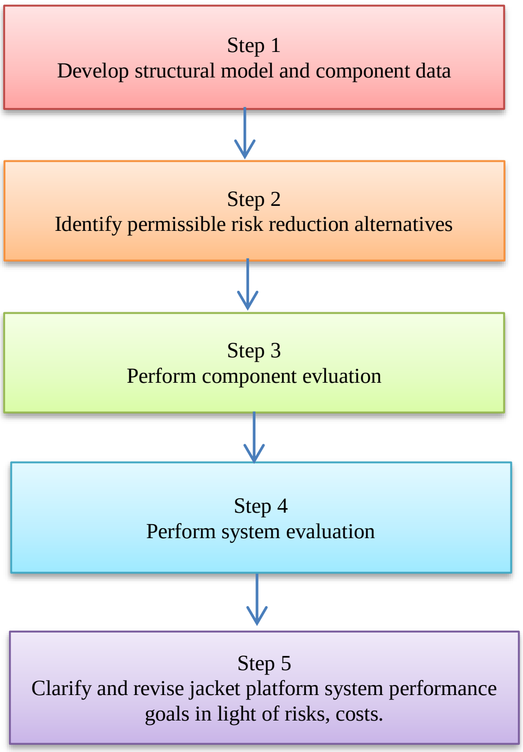


Figure 4. Risk evaluation procedure [10]

In the this research, the process of study has been considered as follows in three steps:
1) Statistical Analysis the results of field experiments, laboratory tests, and comparing the soil strength profile of the SPD22 site with the general trend of the observed changes in the other South Pars Gas Fields that a modified strength profile has been proposed based on.
2) Back Analysis of Pile Drivability: back analysis of Pile driving has been carried out by GRLWEAP software and the new soil strength profile has been extracted for achieving the long-term properties of the soil parameters.
3) Inplace Analysis: the long-term static bearing capacity, according to the proposed soil strength profile (steps 1 and 2) has been calculated.

4.Results and Discussion

In this section, the three previously mentioned steps was reviewed in order and the results was presented and discussed.

4.1. Statistical Analysis

A comparison of the trend of changes in soil properties of field and laboratory experiments provides a good view of the quality of the soil parameters used in engineering analysis. After gathering the geotechnical data, the investigations were carried out on the available evidence. The statistical analyses of all undrained shear strength data for Phases 13, 17, 18, and 22 to 24 of South Pars are presented In Figures 5 to 7.

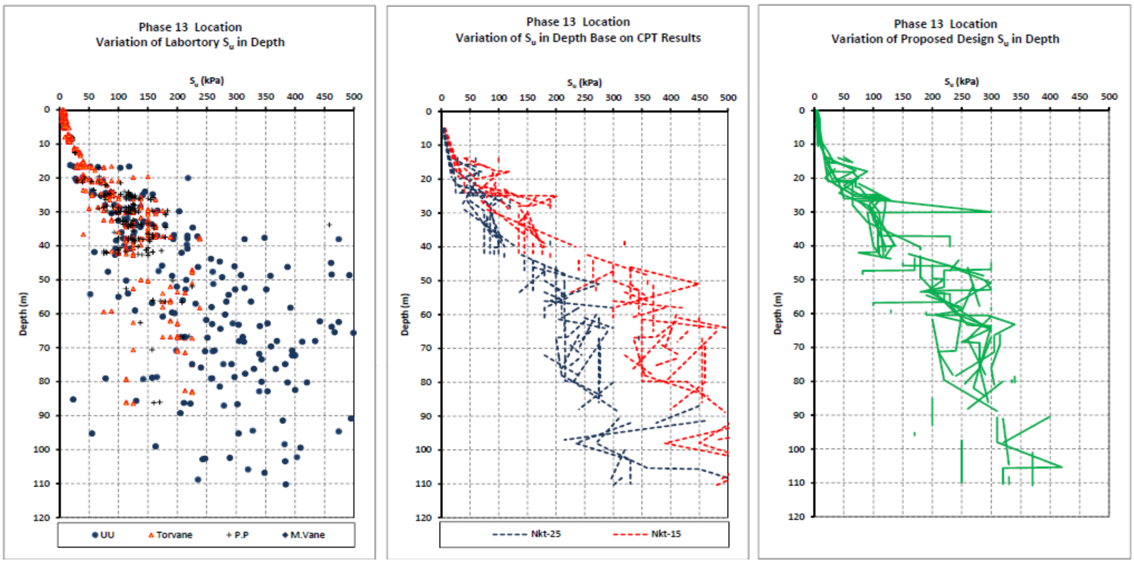


Figure 5. Undrained shear strength in phase 13 boreholes

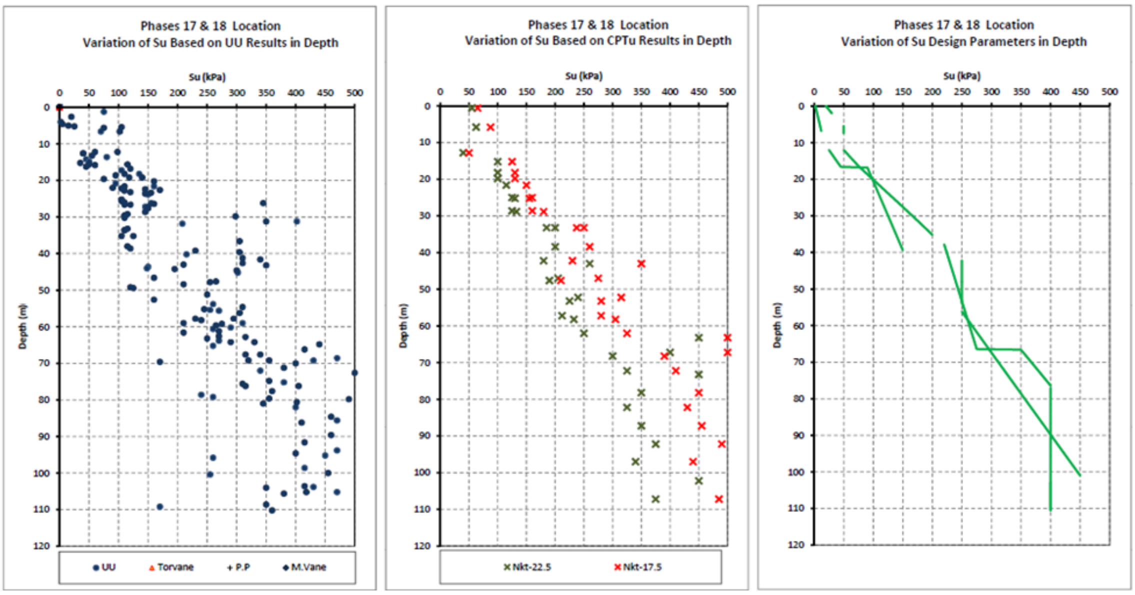


Figure 6. Undrained shear strength in phases 17 & 18 boreholes

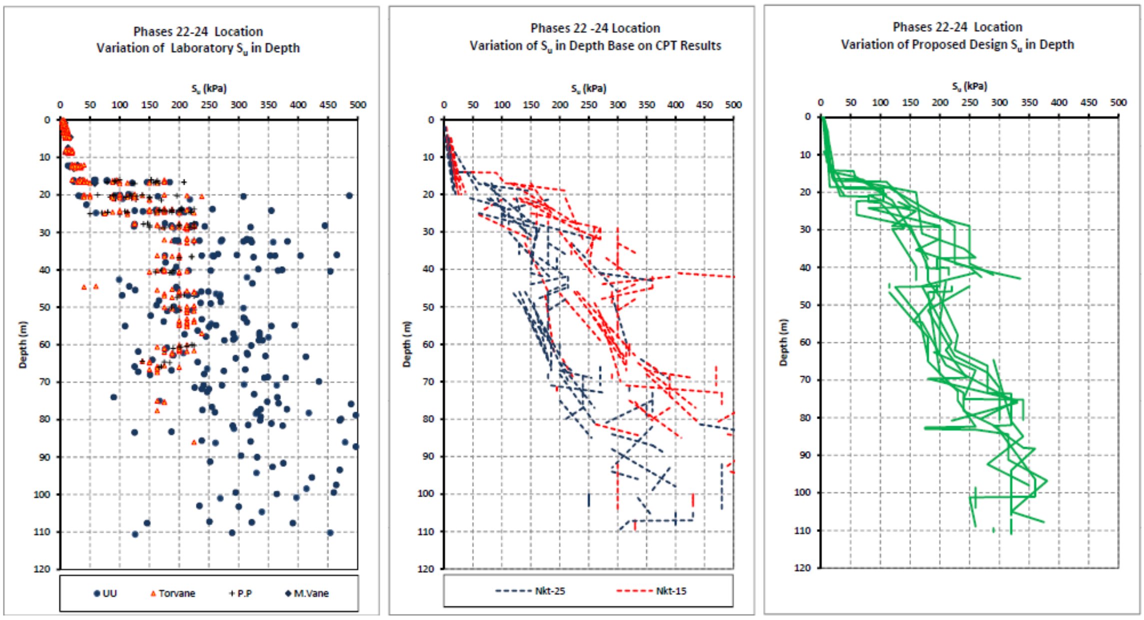


Figure 7. Undrained shear strength in phases 22-24 boreholes

There is three category data in the each figure. The first graphs was obtained by performing statistical analyses of all available experiments and field tests such as Miniature vane cutting, Unconsolidated-Undrained (UU) Triaxial Compression, torvane test, and Pocket Penetrometer results. UU triaxial tests were presented due to the high quality of the results as a suitable measure for comparing the undrained shear strength values (s_u) of clay layers. The second graphs were obtained by the CPT (cone penetration test) results. The undrained shear strength can be extracted from the results of the CPT by applying the experimental N_{kt} coefficient (a dimensionless cone factor) in the pure resistance of pile tip (q_{net}) from the following equation:

$$S_u = q_{net}/N_{kt} \qquad (1)$$

The N_{kt} coefficient is dependent on the site sedimentation conditions and applies to increase the confidence of both upper and lower limits in calculations. By examining the reports, it is clear that the two companies have used different N_{kt} coefficient to determine the adhesion factor. According to Table 3, the limits by company A are more conservative.

Table 3. Proposed N_{kt} coefficient values by companies

Company	The upper limit of N_{kt}	Lower limit of N_{kt}
A	25	15
B	17.5	17.5

The third graphs are the proposed measures that were gained by the mean, median, and standard deviation of the data.
The UU and CPT parameters in Figure 8, represent the degree of matching design parameters with the results of the triaxial tests of UU and CPT and the N_{kt} accuracy. The following points can be deduced:
1. The matching of selected parameters by company A with the results of both UU and CPT decreases with the depth increasing. Even at depths of more than 80 meters, matching decreased to less than 30 percent accuracy which indicates the conservative choice of parameters by company A, and can have a significant effect on the pile bearing capacity reduction because of the high sensitivity to adhesion variations.
2. Correlation between the CPT result and the values obtained from the triaxial UU test in the range of depths of 20 to 68 meters shows a significant decrease. However, the review of existing reports suggests that the choice of proposed adhesion values in these layers was based on the results of the CPT and did not pay attention to the difference between the values obtained with the triaxial UU test results.

Finally, it can be concluded that by the statistical analysis, comparisons, reviewing the design parameters, considering all technical aspects in interpreting the results of laboratory and field experiments, the bearing capacity of the pile can be increased.

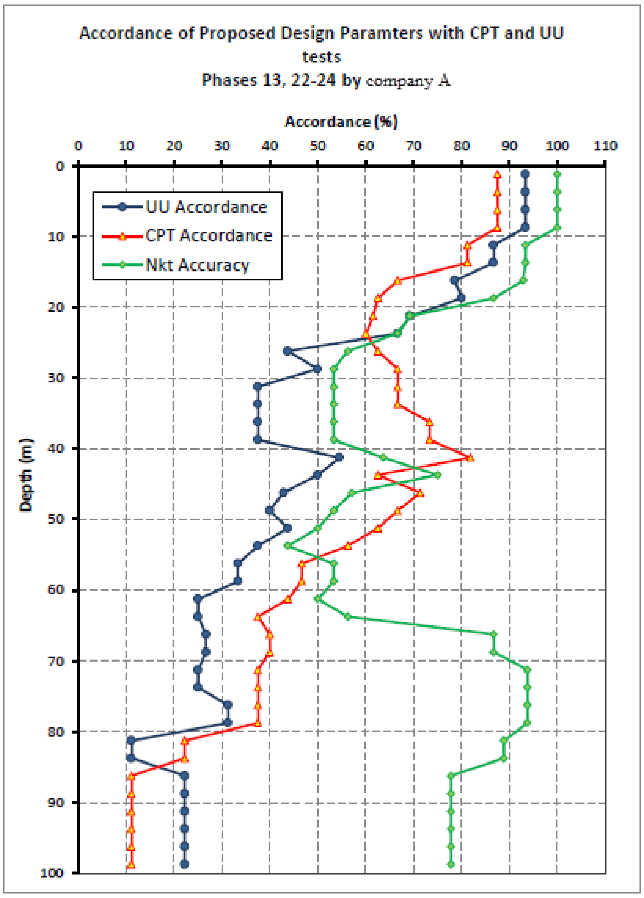


Figure 8. The adaptation variations trend of design parameters and results of laboratory and field experiments in phases 13 and 22- 24 based on the company A reports

The static bearing capacity of the phase 22 piles has been calculated based on the reports provided by Company A, and the length of piles was obtained 97 and 105 meters. Figure 9 provides a comparison between Company A's results and the proposed values derived from the present study. Figure 10 compares the graph of the static bearing capacity, which has shown a significant increase in the bearing capacity of the piles at the end depth of the boreholes.

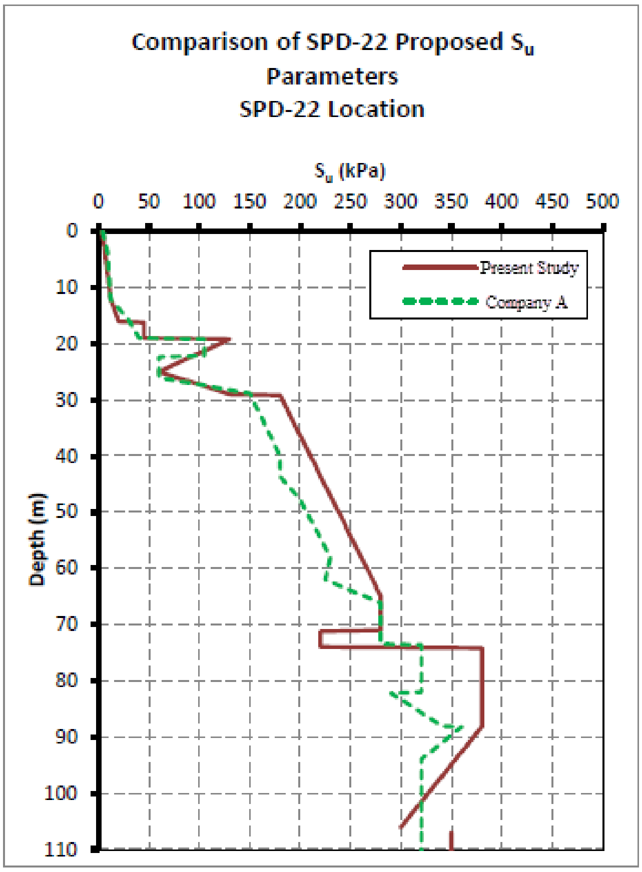


Figure 9. Comparison of S_u - company A and proposed values

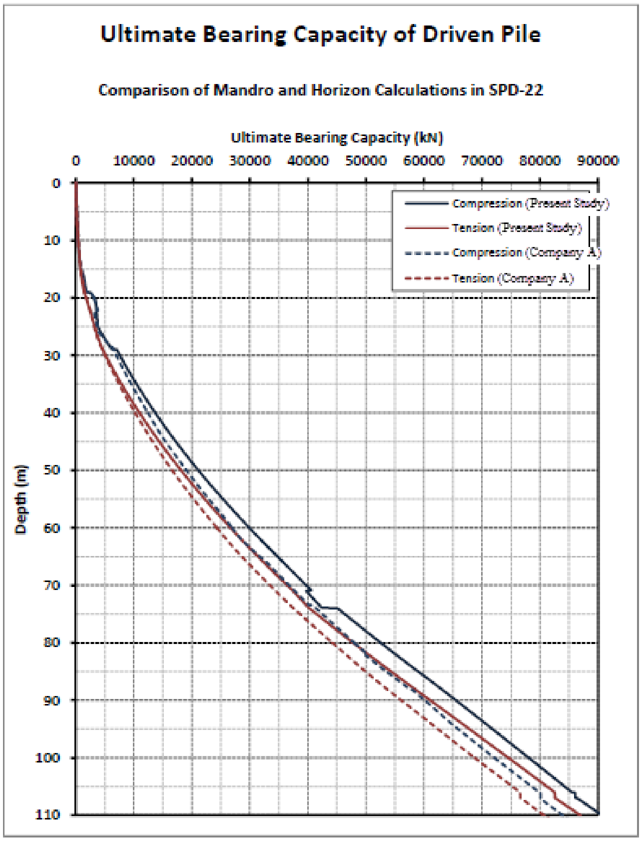


Figure 10. Comparison of the static capacity of the pile - Company A and proposed values

4.2. Back Analysis of Pile Drivability

The analysis on the available pile drivability data was performed in two steps as follows: Step 1 is extracting the soil layers properties using the pile driving simulating by GRLWEAP software [11] as back analysis and the Second step is to compare the results of the back analysis with the values obtained from theoretical relations. In this step, we have reduced the length of the piles by 12 meters and compare the results with the previous step. Each step is given as follows:

Step1: To more accurately estimate the soil layers' long-term strength, after the completion of the installation, it was an attempt to simulate the recorded driving data in the GRLWEAP software. Also, The PDA test, and the CAPWAP analysis [12] were used to extract the required software information to determine the dynamic parameters of the soil. The soil strength parameters that were obtained are the result of multiplying the fatigue factor and the undrained shear strength of soil layers. Figure 11 compares the simulated pile driving energy and the back analysis results.

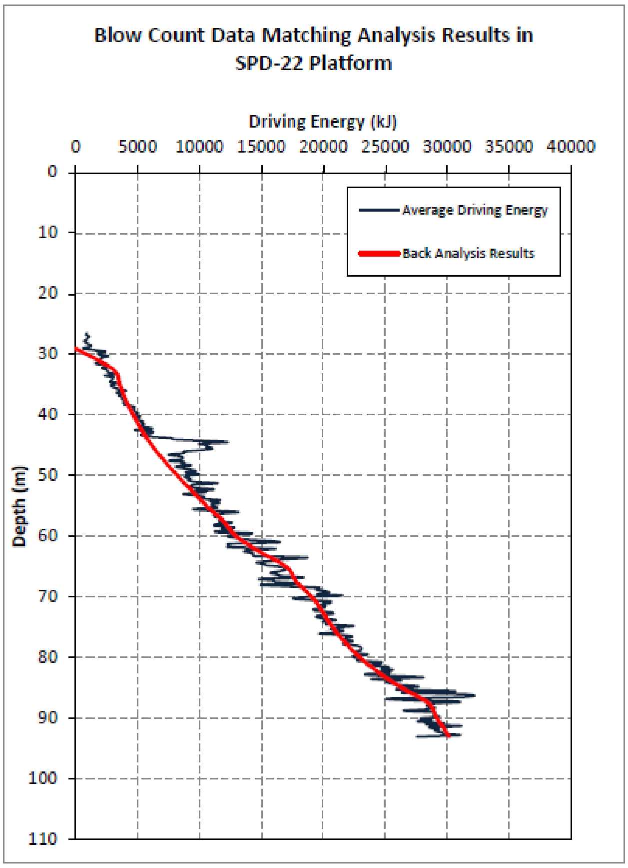


Figure 11. Comparative analysis of pile drivability information on the site of installation

The parameters extracted from the analysis represent the strength of the soil layers during the operation of the pile driving. The clay layers are faced with a considerable reduction in the stresses due to the significant increase of water pressure in the soil texture. Consequently, the adhesion to the pile shaft was significantly reduced, which this phenomenon is referred to as the fatigue factor. The differences between the short-term strength parameters during pile driving and the proposed values by company A and extracted quantities in this study are shown in Figure 12.

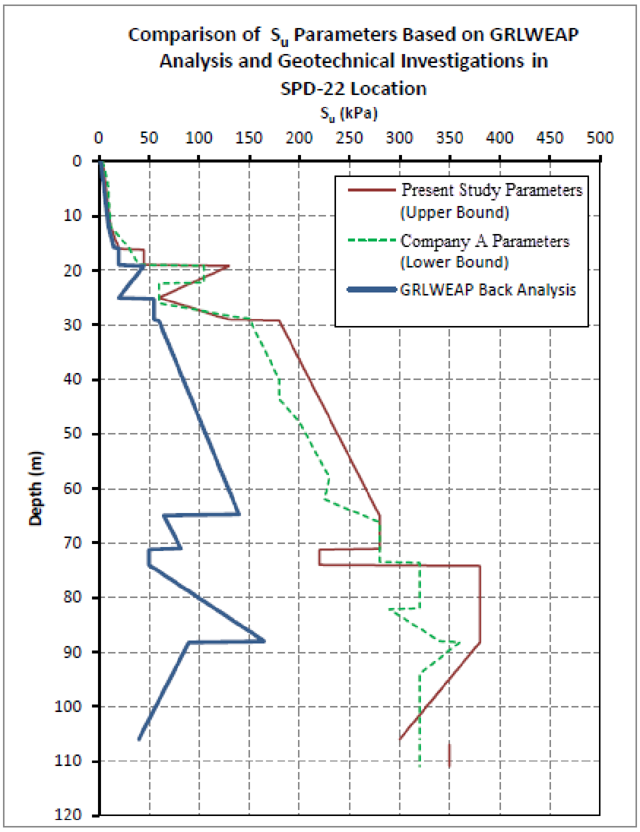


Figure 12. Comparison of short-term parameters during pile driving and suggested values by present study and company A

In this diagram, the proposed strength profile of company A is the lower limit, and the intended profile of the present study is obtained as the upper limit. knowing that the short-term properties have resulted from fatigue factor measure and soil adhesion in each layer, the expected upper and lower limit of the fatigue factor can be estimated by equation 2:

$$setup = \frac{1}{soil\ fatigue} \qquad (2)$$

In the technical references (Randolph, Gourvenec, 2011) [13], the expected range of setup factor in clays is between 2 and 8, Also, according to the proposed relationship by Stevens et al., 1982 [14] and Colliat et al., 1993 [15], strength reduction is a function of the over-consolidation conditions (OCR) of soil layers. Company A has used a few corrections based on the pile drivability experiences in the Persian Gulf according to below equations.

$$F_p = 0.5\ OCR^{0.3} \qquad \text{Stevens et al. (1982)} \qquad (3)$$

$$F_p = \lambda\ OCR^{0.2} \qquad \text{Modified Colliat} \qquad (4)$$

Company A (5)

	λ Value	
	Lower Bound	*Upper Bound*
Upper Section of Pile (Seabed to 10 m above toe)	*0.1*	*0.2*
Lower Section of Pile (10 m above toe to final depth)	*0.25*	*0.4*

In Figure 13, the setup factor comparison was provided. The significant difference between the results of various theoretical relationships indicates that the accurate fatigue factor prediction of the clay layer is very complicated. The values were obtained from the numerical analysis show a good matching with theoretical values based on the general trend of changes. The average value of the setup factor is equal to 2.83 based on modified data. In other words, With passing a little time from the pile driving operation, the ultimate strength of the pile becomes about 2.8 times. The complete elimination of the excess water pressure in the clay layers and achieving the effective stresses of the static state can be the main reasons. In the following, the simulation of the PDA test was carried out and the results of this analysis are presented in two depths of 85 m and 93 m in figure 14 while applying the coefficient of setup equal 2.5, as shown in table 4.

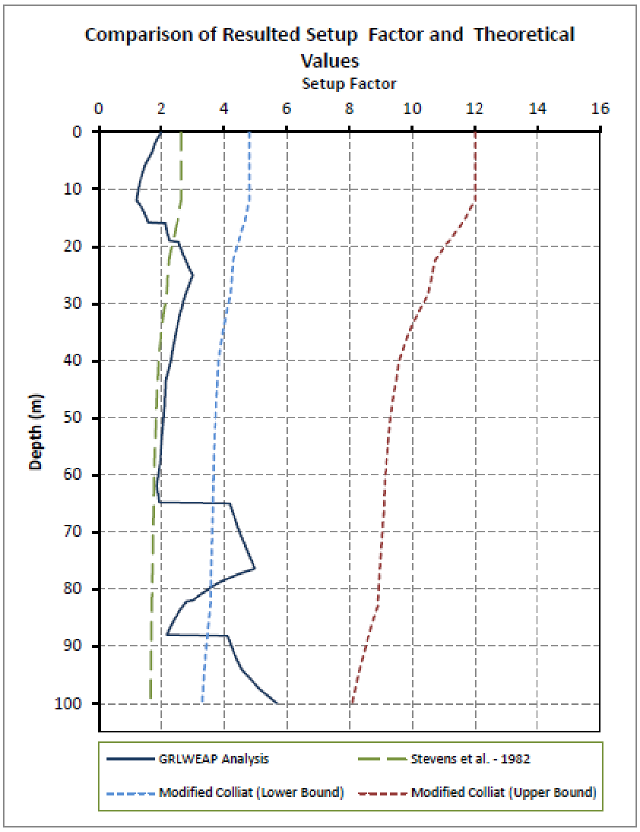


Figure 13. Comparison of set up values based on the results of back analysis and theoretical relations

Step 2: In this section, we will try to estimate the long-term bearing capacity after the completion of the operation, using various theoretical equations. In the absence of loading experiments, theoretical relationships can provide an estimate of the process of variation in the final bearing capacity of the piles over time from the end of the operation. Among the various researches, the relationships provided by both Skov and Denver, 1988 [16, 17] and Svinkin, 1996 [18] are more practical. The equations used to determine long-term bearing capacity are as follows:

Skov and Denver (1988)

$$Q_t = Q_0[A\log(t/t_0) + 1] \qquad (6)$$

Q_t = pile capacity as time t

Q_0 = Pile Capacity at $t=t_0$

For Clays, $A=0.6$ and $t_0=1.0$

Svinkin and Skov (2000)

$$Q_t/Q_{EOD} - 1 = B[\log(t) + 1], \quad \text{B is the same as A, in Skov and Denver} \qquad (7)$$

Svinkin (1996)

$$\text{Upper Bound:} \quad Q_t = 1.4\, Q_{EOD} t^{0.1} \qquad (8)$$

$$\text{Lower Bound:} \quad Q_t = 1.025\, Q_{EOD} t^{0.1}$$

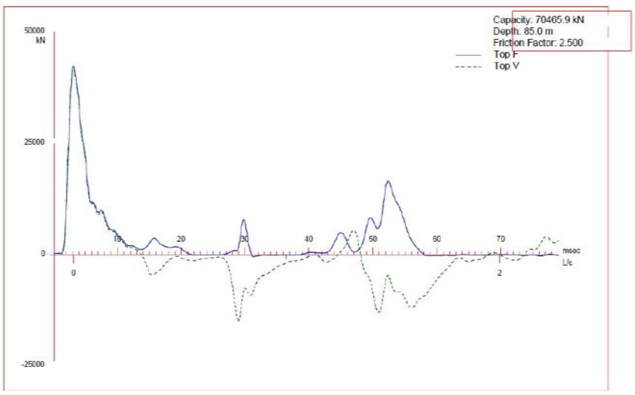
Table 4 presents a comparison between the results of long-term bearing capacity determination using GRLWEAP software and theoretical equations.

Table 4. Comparison of long-term bearing capacity using software and theoretical equations

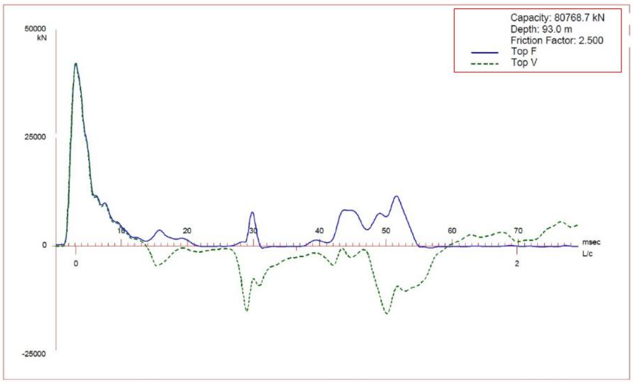
Pile ID	*Tip Penetration (m)*	*Anticipated Long Term Bearing Capacity (kN)*				
		GRLWEAP results	*Skov & Denver [1]*	*Svinkin[1](LB)*	*Svinkin [1](UB)*	*Svinkin & Skov [1]*
A1	*85*	*70,466*	*97,496*	*71,212*	*97,265*	*118,388*
A2	*93*	*80,769*	*114,044*	*83,299*	*113,774*	*138,482*

[1] Anticipation are presented for approximately1000 days after EOID

Also, the pile bearing capacity increase in time has shown based on theoretical relationships in Figure 15 both different pile lengths.



(a)

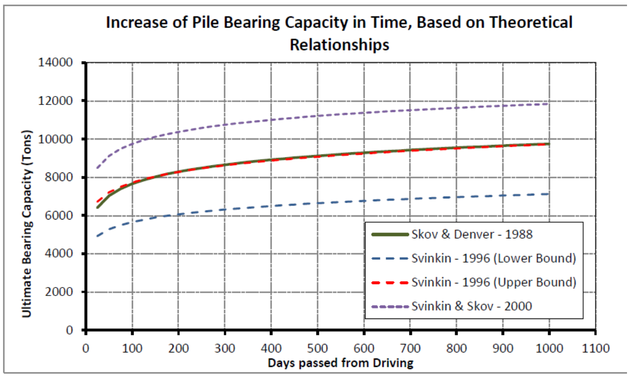


(b)

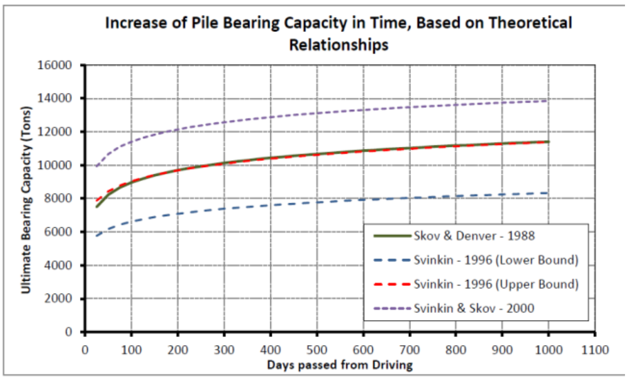
Figure 14. The result of long term loading capacity of piles at depths (a) 85 m & (b) 93 m by applying the setup factor of 2.5

Table 5. Expected long-term bearing capacity after the pile driving operation

Tip Penetration (m)	Applied Setup Factor	Anticipated Ultimate Long-term Bearing Capacity (kN)
85	2.5	70466
93	2.5	80769



a) Pile A1



b) Pile A2

Figure 15. Increase in pile bearing capacity in time based on theoretical relationships

It can be concluded that the process of numerical analysis is very conservative and even less than the minimum value obtained from the equation

4.3. Inplace analysis

In the third step, inplace Analysis of the SPD22 jacket platform was performed with the new pile length of 85 and 93 meters.Using the geotechnical report of company A, the length of piles is 97 & 105 m, but with the modified soil properties of the present study, 85 & 93 m is obtained with a safety factor of 2.5 for both analyses in SACS software (table 6).

Table 6. Maximum factor of safety for all piles

Pile	Minimum Factor of Safety
A1	3.08
A2	2.87
B1	3.24
B2	3.00

5. Conclusion

The results obtained can be summarized as follows:

a) By data gathering and statistic analysis, the bearing capacity of piles has been increased by 10 percent, which can be concluded that the exact research on field and laboratory tests results can reduce the over-conservatism and leads to risk reduction.

b) By performing the pile drivability analysis of the selected platform in the South Pars Gas Field and comparing the results with the other fields' data, it was found that the sensitivity of the clay layers was increased in depth when pile driving operation. In other words, soil resistance decreases.

c) From the pile drivability analysis, back analysis, and comparing these results, the estimated soil strength is about 2.8 times more than the pile driving operation time. However, if this value is considered to be 2.5, the pile length can be reduced by 12 m which reduces the use of the material by 11%.

d) The practical conclusion that can be drawn is that the Svonkin equation (lower bound) can be used for the estimation of the long-term bearing capacity to reduce the over-conservative.

e) Since pile driving operation is done with a pile with a design diameter, then the resulting soil properties are much closer to reality, so the results of pile driving in a field can be used for optimal pile design.

6. References

[1] Chen, J.Y., Matarak, B.A., carpenter, J.F., and Gilbert, R.B., (2009), Analysis of Potential Conservatism in Foundation Design for Offshore Platform Assessment, Final Project Report Prepared for the Minerals Management Service.

[2] Tang, W.H., and Gilbert, R.B., (1992), Offshore Pile System Reliability, Final Report to American Petroleum Institute (Project PRAC 89-29).

[3] Murff, J. D., and Hamilton, J. M., (1993), P-Ultimate for Undrained Analysis for Laterally Loaded Piles, Journal of Geotechnical Engineering, ASCE, Vol. 119, No. 1, pp. 91-107.

[4] Murff, J.D., Lacasse, S., and Young, A.G., (1993), Discussion and summary on foundation elements, system and analysis, Proceedings of an International Workshop Assessment and Requalification of Offshore Production Structures, New Orleans, Louisiana, pp. 151-165.

[5] Pelletier, J. H., Murff, J. D., and Young, A. C., (1993), Historical Development and Assessment of the Current API Design Methods for Axially Loaded Piles, Offshore Technology Conference, Houston, Texas, OTC 7157.

[6] Aggarwal, R.K., Litton, R.W., Cornell, C.A., Tang, W.H., Chen, J.H. and Murff, J.D., (1996), Development of pile foundation bias factors using observed behavior of platforms during Hurricane Andrew, Offshore Technology Conference, Houston,Texas, OTC 8078, pp. 445-455.

[7] Bea, R.G., Jin, Z., Valle, C. and Ramos, R., (1999), Evaluation of reliability of platform pile foundations, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 125, No. 8, pp. 695-704.

[8] American Petroleum Institute, (2000) Recommended practice for planning, designing and constructing fixed offshore platforms, 21st ed., Washington, D.C.

[9] SACS Suite program, (2011), version 5.3.

[10] Werner, S. D., Dickenson, S. E., and Taylor, C. E., (1997), Seismic Risk Reduction at Ports: Case Study and Acceptable Risk Evaluation, Journal of Waterway, Port, and Coastal Engineering, American Society of Civil Engineers, Vol. 123, November-December, pp. 337-346.

[11] PDI, (2010), GRLWEAP Manual for Windows, 30725 Aurora Road, Cleveland, OH 44139 USA: PDI.

[12] CAPWAP Signal Matching, Art of Foundation Engineering Practice (GSP 198), p. 534–553.

[13] Randolph, M.F., and Gourvenec, S., (2011), Offshore Geotechnical engineering, Spon Press, London.

[14] Stevens, R., Wiltsie, E. & Turton, and T., (1982), Evaluating Drivability for Hard Clay, Very Dense Sand, and Rock, Offshore Technology Conference.

[15] Colliat, J., Vergobbi, P. and Puech, A., (1993), Friction Degradation And Set-Up Effects In Hard Clays Offshore Congo And Angola, Offshore Technology Conference.

[16] Skov, R., Denver, H., (1988), Time-dependence of bearing capacity of piles, The 3rd International Conference on Application of Stress-wave Theory to Piles, Ottawa, Canada, pp. 879–888.

[17] Svinkin, M.R., and Skov, R., (2000), Set-up effect of cohesive soils in pile capacity, The 6th International Conference on Application of

Stress-wave Theory to Piles, Sao Paulo, Brazil, pp. 107–111.
[18] Svinkin, M.R., (1996), Setup and relaxation in glacial sand-discussion, Journal of Geotechnical and Geo-environmental Engineering – ASCE, 122 (4), pp. 319-321.