DRIVEABILITY OF RECYCLED PLASTIC PIN (RPP) IN NORTH TEXAS

By

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ABSTRACT

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The Recycled Plastic Pins (RPP) have been extensively used as an alternative to the conventional slope stabilization methods in stabilizing shallows slope failure in Texas, Missouri, Kansas, and Idaho in the United States. The RPP provides additional resistance along the potential slip surface when it is driven into the slope and increases the factor of safety. RPPs are fabricated from recycled plastics and waste materials (i.e. polymers, sawdust, and fly ash), and is non-degradable in nature. It is found to be very useful and beneficial engineering materials for civil engineering infrastructure projects. The cost-effectiveness of using any engineering material largely depends on the efficacy of the project planning where proper scheduling based on the estimation of time is a prime aspect. Although several studies have been conducted to estimate the driving rate of piles using wave equation analysis, static resistance to driving, and dynamic soil properties, no study has been performed to estimate the driving rate of RPPs based on different soil properties. Also the comprehensive understanding of the interaction between RPP and soil properties while driving would assist in bringing about an optimized design method for slope stabilization. Hence, a better understanding of the interconnection between the RPP driving rate and soil properties is an essence. Therefore, the objective of this study is to evaluate the driving rate of RPP based on different soil parameters.

The influences of soil properties on RPP driving rate were studied in detailed manner in the present research. The considered soil properties were natural moisture content, dry density, plasticity index, and cohesion. Also the influence of standard penetration test (SPT) value of soil on driving rate of RPP was examined. The driving time and rate along with soil properties utilized in this study were assembled from the studies conducted by Khan (2013), Tamrakar (2015), Zaman (2019), and Sapkota (2019). The data were also collected directly from different sites in Irving and Arlington where RPPs were used for different engineering applications. Based on the analyses, it was found that the driving rate increased with an increase in natural moisture content and plasticity index while it decreased with the increase in dry density, cohesion, and SPT value of soil.

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Chapter 1

INTRODUCTION

1.1 BACKGROUND

The utilization of recycled plastic pin (RPP) for highway slope stabilization and civil engineering infrastructure projects is considered as an excellent sustainable engineering solution. Being non-degradable, plastic is problematic if discarded into landfills. However, it can be very useful and beneficial for civil engineering infrastructure projects due to its non-degradable nature. Slopes repaired and reinforced with recycled plastic pin (RPP) preserve their engineering characteristics for a long time, thereby reducing the overall maintenance and repair cost over time.

As an alternative to the conventional methods of slope stabilization namely installation of drilled shaft, replacement of slope using retaining structures including MSE wall, installation of soil nail and reinforcing the slope using geogrids, the recycled plastic pins (RPP) have been effectively used as a sustainable and economic solution for stabilization of shallow slope failures in Missouri, Iowa, Idaho, and Texas (Hossain et al., 2017; Khan et al., 2016; Loehr and Bowder, 2007; Loehr et al., 2000; Sapkota et al., 2019). It is a lightweight material and is less susceptible to chemical and biological damage, resistant to moisture, required practically no maintenance, thereby making it an agreeable alternative compared to other structural materials (Krishnaswamy and Francini, 2005). The use of plastic pin reduces the volume of waste entering the landfill resulting in creation of additional demand for recycled plastic (Loehr et al. 2000). The study of Zaman (2019) also represented that RPP provides additional support to the embankment structure when driven into the weak foundation soil in addition to a layer of geogrid as well as increases shear resistance of the base of MSE wall against sliding failure by acting as a shear key.

Because of the cost-effectiveness and sustainability aspects as well as simplicity in the installation process, slope stabilization using recycled plastic pins (RPP) is getting enormous attention from the public and private sectors. Different state departments of transportation (DOTs) and private companies are now considering RPP as a superior alternative for slope stabilization. RPPs are installed in the slope, just driven through the ground, and they intercept potential sliding surfaces providing additional resistance with a view to maintaining the long term stability of the slope. A number of studies conducted over the past few years in an effort to stabilize shallow slope failures by using RPP were found (Khan et al., 2015; Khan et al., 2013; Loehr and Bowders, 2007). The long term performance of RPP in stabilizing slope as a geo-structural member depends on its material and structural properties, interaction with soil, and proper installation technique. Properties of RPP as a structural member were studied by Bowders et al. (2003), Lampo and Nosker (1997), Breslin et al. (1998), Krishnaswamy and Francini (2000), Malcom (1995), Van Ness et al. (1998), Ahmed (2012), etc.

Installation technique for any pile as well as RPP determines the project schedule time and the post-construction performance of the geo-structural member. The installation technique and the interaction of structural members with soil regulate the rate of driving. A better understanding of RPP-soil interaction and driving mechanism would contribute to optimized slope stabilization design methodology which will improve the functioning of RPP. On the other hand economic efficiency of RPP depends largely on the proper planning of the involving project where earlier estimation of installation time is an essence. Sommers et al. (2000), Khan (2013), Tamrakar (2015), Zaman (2019), and Sapkota (2019) examined the driving rate of RPP in different areas of North Texas without considering its interaction with soil properties. The current study represents a basis for analyzing the correlation of driving rate of RPP with soil properties as well as estimating the driving rate and the required time for driving recycled plastic pin (RPP).

1.2 PROBLEM STATEMENT

The previous studies on recycled plastic pin (RPP) discovers its properties (Bowders et al., 2003; Lampo and Nosker, 1997; Breslin et al., 1998; Krishnaswamy and Francini, 2000; Malcom, 1995; Van Ness et al., 1998; Ahmed, 2012 etc.) and methods for design in slope stabilization (Khan et al., 2015; Khan et al., 2013; Loehr and Bowders, 2007; Sapkota et. al., 2019) as well as its effectiveness in ground improvement and sliding resistance (Zaman, 2019). However, no study was found to evaluate the RPP's driving rate with respect to the site soil properties. The driving rate of RPP depends on a number of factors, e.g. structural properties of RPP, soil properties, site conditions, installation method, workmanship, weather, and machine used during installation.

Assessment of driving rate and installation time of RPP has become a concern since its popularized and effective use in large geotechnical projects. An important aspect of project planning is scheduling time with reasonable accuracy since when deadlines are failed to be met and the time limits get extended, project costs rise accordingly and thereby impacting the profitability of total project (Suri et al., 2009). Therefore it is necessary to estimate the installation time of RPP in the project planning stage. Also, a better understanding of the interaction between the RPP and soil properties would assist in bringing about a more optimized design methodology.

The ever increasing pattern for mass utilization of RPP in slope stabilization especially in North Texas demands the assessment of its installation time earlier at the project planning stage. However, site soil condition is examined increasingly by in-situ methods nowadays. Additionally, soil properties can be found from in-situ testing using established correlations. Therefore, this research attempted to study the influences of basic soil properties on the driving rate of RPP which can be helpful for prompt estimation of the RPPs driving time.

1.3 RESEARCH OBJECTIVES

The main objective of the current study was to evaluate the driving rate of recycled plastic pin (RPP) based on soil parameters. In order to fulfill the objective, the specific tasks performed:

- Evaluate the driving rate of RPPs based on soil parameters such as:
    a. Moisture content
    b. Dry density
    c. Cohesion
    d. Plasticity Index, and
    e. Standard Penetration Test (SPT)
- Predict possible trendline between the driving rate of RPPs and soil parameters.

Chapter 2

LITERATURE REVIEW

2.1 INTRODUCTION

Failure of highway slopes and structures constructed over incompetent soil is a common problem encountered by the civil engineers. In most of the cases, sites having incompetent soil is not suitable for the construction of structures over it. Sometimes removing and replacing of soil with proper fill material may be considered as the only option, even after being an expensive solution. However, as an alternative to the conventional methods of slope stabilization namely drilled shaft installation, slope replacement with retaining structures including MSE wall, utilization of soil nail, and geogrid reinforcement, the recycled plastic pins (RPP) have been effectively used as a sustainable and economic solution for stabilization of shallow slope failures. Installation mechanism of any driven pile system as well as RPP affects the project economy and post construction performance. This chapter presents comprehensive information collected from the related literature addressing the utilization of recycled plastic pin (RPP) as an innovative green solution to slope failure and weak foundation soil problems and the driving mechanism of different pile systems.

2.2 RECYCLED PLASTIC PIN (RPP)

Recycled plastic pin (RPP) is fabricated using recycled plastic and other waste materials like polymers, fly ash and saw dust (Chen et al., 2007). From the standpoint of environmental and life cycle cost analysis (LCCA), the recycled plastic pin (RPP) is under serious consideration as structural materials for marine and waterfront application (Khan, 2014). RPP is a sustainable material which require almost no maintenance and resistant to moisture, corrosion, rotting and insects. Typically, more than 50% of the feedstock used for plastic lumber composed of polyolefin in terms of high density polyethylene (HDPE), low density polyethylene (LDPE) and polypropylene (PP) (Khan, 2014). The polyolefin used in the combination acts as adhesive that helps combining high melt plastics and additives such as fiberglass, wood fibers within a rigid structure.

2.2.1 Green Engineering

According to EPA (United States Environmental Protection Agency), the design, commercialization, and use of products in a way that reduces pollution and waste, promotes sustainability, and minimizes risk to the environment without sacrificing viability and efficiency is termed as green engineering. An excellent example of green engineering can be Recycled Plastic Pins (RPP), which reduces the waste volume entering into the landfill, provides additional market for RPPs and can be an economical solution to numerous geotechnical projects.

The rapid growth of population resulted in an increased volume of waste generation. Annually, a total of 1.3 billion tons of municipal solid waste (MSW) is generated, which is expected to increase by almost double by the year 2025. 10 % of this waste is composed of plastic waste, which amounts to approximately 130 million tons. In USA, the amount of generated plastic waste is approximately 32.5 million tons, which is 13 % of the total waste volume. However, plastic waste occupies a large volume of landfill space, even though they are lightweight material. In addition, plastic waste, being a non-degradable part of the MSW stream, once buried in the landfill stays and occupies the space forever. Therefore, a huge landfill space can be saved if diversion and reuse of this non-degradable waste is ensured. At the same time, it ensures additional space availability for new waste and increases the operational life of a landfill.

The plastic and plastic products, being non-degradable, poses problem for landfill. However, this non-degradable nature becomes advantageous, when they are used in projects related to civil engineering infrastructure. RPP made out of recycled plastic bottles, when used in slope stabilization, ground improvement or other purpose, they can perform well for a long time by preserving their engineering characteristics. This minimizes the overall repair and maintenance cost of the project. Hence, the use of RPP demonstrates the perfect example of sustainable engineering solution (Hossain et al., 2017).

2.2.2 Manufacturing Process of RPP

The production process of plastic lumber begins with collection of raw materials followed by cleaning and pulverizing the raw materials. Approximately 600 mineral water/soda bottles are used for one 4 in. x 4 in. RPP (Figure 2.60). The resulting product is melted in an extrusion machine at a production site. Two methods of manufacturing process for recycled plastic lumber are presented by Malcolm (1995) such as the Injection molding process and the continuous extrusion process.

The injection process involves injection of molten plastic into a mold that defines the shape and length of the product followed by uniform cooling and then removal of the finished product. This process is relatively simple and inexpensive; however, the volume produced is limited (Malcolm, 1995). On the other hand, the continuous extrusion process allows production of varying length of the RPP. During this process, the molten plastic is continuously extruded through series of dies which shape the materials during its cooling. However, it becomes challenging to ensure uniform controlled cooling of the sample to prevent warpage and caving of the lumber. Also, a considerable investment is required in comparison to the injection molding process. However, the continuous extrusion process requires less labor and suitable for mass production.

Figure 2.1: Recycled Plastic Pins (Hossain et al., 2017).

Another widely used manufacturing process of the recycled plastic pin is the compression molding process (Lampo and Nosker, 1997) where other materials are mixed with batches consisting of 50-70% of thermoplastics by melting. An automatically adjusted scraper is used to remove the melted material from the plasticator followed by pressing it through a heated extruder die into premeasured, roll-shaped loaves. The loaves are then processed through a press-charging device that fills a sequence of compression molds alternately. The finished products are cooled to a temperature of 40 °C within the mold and ejected into a conveyor to be carried to a storage area.

2.2.3 Engineering Properties of RPP

Bowders et al., (2003) conducted a study on the different engineering properties of RPP to evaluate the engineering properties of wide varieties of production standard. As a part of the study uniaxial compression test and four point flexure test were performed, the results of which are presented in Table 2.1 (uniaxial compression test) and Table 2.2 (four point bending test).

Table 2. 1: Uniaxial compression test results (Bowders et al., 2003).

Specimen Batch	No. of Specimen tested	Nom. Strain Rate (%/min)	Uniaxial Compressive Strength (MPa)		Young's Modulus, $E_{1\%}$ (MPa)		Young's Modulus, $E_{5\%}$ (MPa)	
			Avg.	Std. Dev.	Avg.	Std. Dev.	Avg.	Std. Dev.
A1	10	-	19	0.9	922	53	390	27
A2	7	0.005	20	0.8	1285	69	378	15
A3	6	0.006	20	0.9	1220	108	363	27
A4	3	0.004	20	0.9	1377	165	363	25
A5	4	0.006	12	1	645	159	225	17
A6	4	0.006	13	0.9	786	106	238	34
B7	2	0.007	14	0.5	541	36	268	3
B8	2	0.006	16	0.4	643	1	308	0.5
C9	3	0.0085	17	1.1	533	84	387	40

A comparative experimental study on the compressive strength of Recycled Plastic Lumber on a total of 10 plastic samples, obtained from eight manufacturers, was conducted by Lampo and Nosker (1997). The product composition had great variations, such as, some were mixed plastics, some were pure resigns and others contained fillers such as wood pulp or fiberglass. The experimental study was conducted by following ASTM 695-85 with samples of 12 inch height. The study included an effective cross sectional area which was calculated based on a specific gravity measurement to calculate the mechanical properties of the material. It should be noted that the compressive strength test was performed at 0.1 in/min rate. Based on the experimental results, the modulus, ultimate strength at 10% strain and yield strength at 2% offset were calculated from the load-displacement data.

Table 2.2: Four point bending test results (Bowders et al., 2003).

Specimen Batch	No of Specimens Tested	Nom. Def. Rate (mm/min)	Flexural Strength (MPa)	Secant Flexural Modulus $E_{1\%}$ (MPa)	Secant Flexural Modulus $E_{5\%}$ (MPa)
A1	13	-	11	779	662
A4	3	4.27	18	1388	-
A5	3	5.74	11	711	504
A6	4	3.62	10	634	443
B7	1	4.05	9	544	425
B8	1	5.67	-	816	-
C9	2	3.21	12	691	553

To minimize effect of voids when comparing the material properties and effect from different extrusion method, the modulus and ultimate strength are normalized by dividing with specific gravity to determine specific modulus and specific strength. Based on the study, the compressive strength results are presented in Table 2.3. In addition, the comparisons of compressive strength between different samples are presented in Figure 2.2 and Figure 2.3.

According to the experimental study conducted by Lampo and Nosker (1997), the compressive strength for RPP lumber ranged between 1.74 to 3.5 ksi and the tensile strength varies between 1.25 to 3.5 ksi. However, it was also concluded that the RPP reaches its ultimate strength at different strain level compared to softwood.

A study conducted by Breslin et al. (1998) showed the comparison between different test results from literature as presented in table 2.4. The authors reported that adding different additives like fibers, glass, polystyrene etc. into plastic lumber increases the stiffness of the final product.

Plastic is susceptible to temperature. At higher temperature it is weak and shows ductile behavior; however, at lower temperature plastic is much stronger and brittle in nature. Figure 2.4 presents the effect of temperature change on tensile strength of HDPE (Malcolm 1995).

Table 2.3: Average values of specific gravity, modulus, specific modulus, yield stress, ultimate stress and specific strength for each samples type (Lampo and Nosker, 1997).

Sample	Specific Gravity	Modulus (ksi)	Specific Modulus (ksi)	Yield Stress (ksi)	Ultimate Strength (ksi)	Specific Strength (ksi)
51A	0.2789	38.00	121.83	0.71	0.78	2.80
1B	0.7012	61.93	88.33	1.38	1.89	2.70
2D (BR)	0.8630	85.28	98.92	1.67	2.32	2.69
2D (G)	0.8098	116.03	143.30	2.10	2.86	3.53
1E	0.862	80.79	93.84	1.77	2.42	2.81
1F	0.7888	108.20	137.06	2.19	2.81	3.56
1J(B)	0.7534	93.26	123.86	1.90	2.36	3.13
1J(W)	0.9087	110.08	121.25	2.16	2.83	3.11
23L	0.7856	191.45	243.66	1.71	1.93	2.46
1M	0.5652	57.87	102.25	0.96	1.23	2.18
1S	0.9090	80.50	88.47	1.67	2.05	2.26
1T	0.8804	117.92	133.58	2.25	3.12	3.54
9U	0.774	86.73	111.53	1.83	2.41	3.11

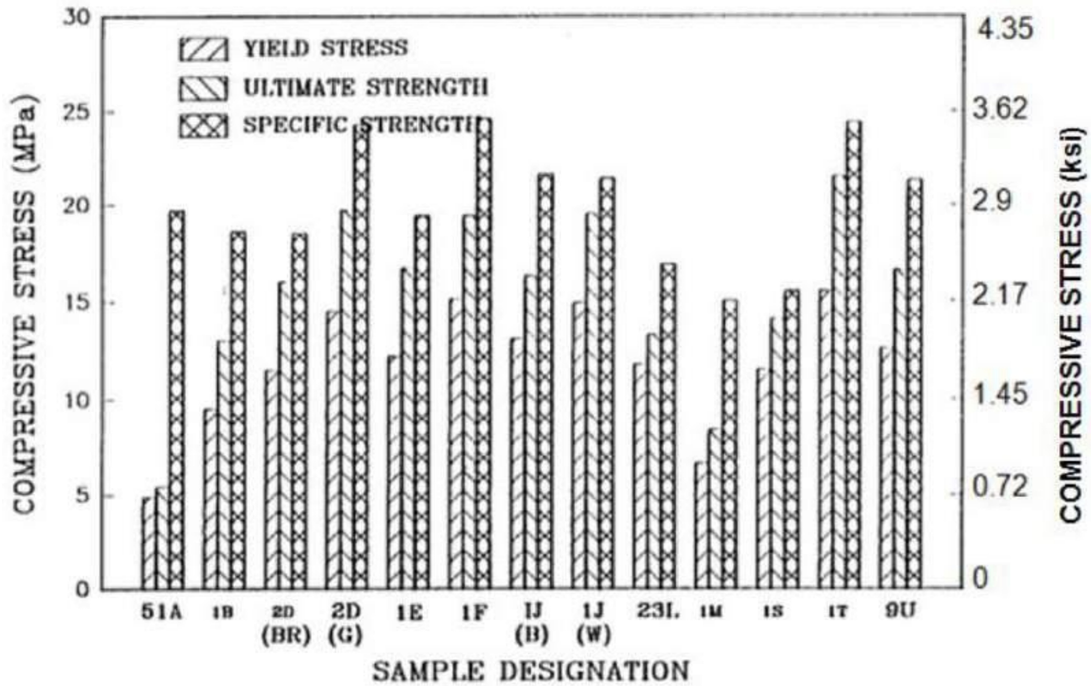


Figure 2.2: Comparison between compressive strength of RPP (Lampo and Nosker, 1997).

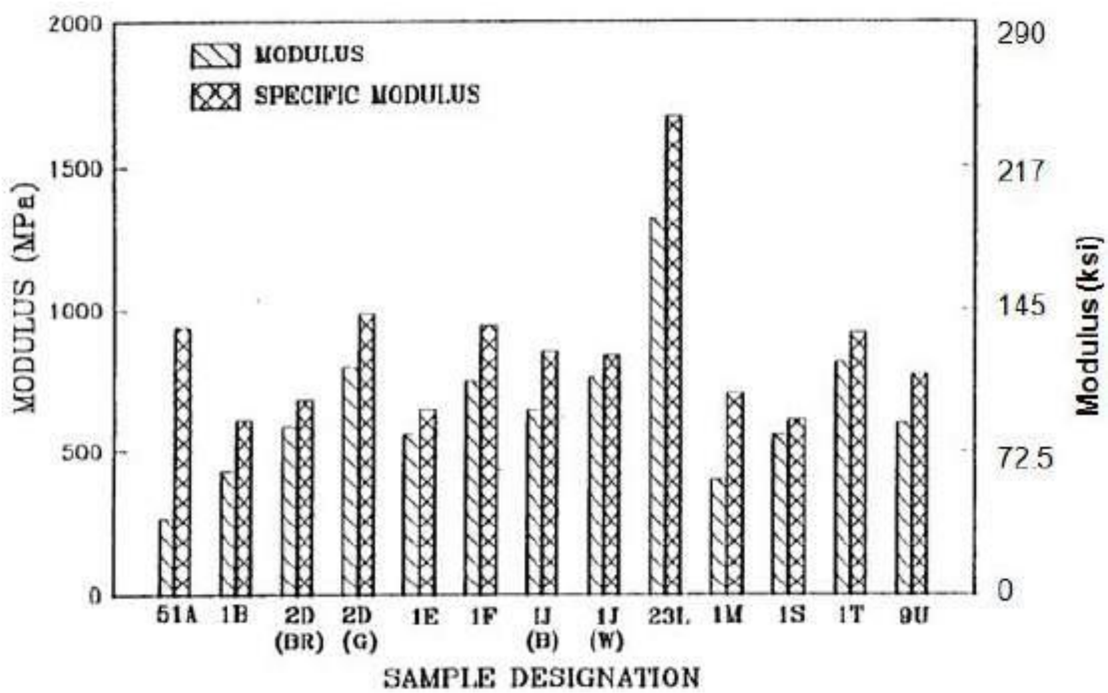


Figure 2.3: Comparison between Compressive modulus of RPP (Lampo and Nosker, 1997).

Table 2.4: Engineering properties of recycled plastic pins (Breslin et. al, 1998).

Product	Composition	Compressive Strength (psi)	Modulus of Elasticity (psi)	Tensile Strength (psi)	Source
TRIMAX	HDPE / Glass Fiber	1740	450 000	1250	TRIMAX literature SUNY at Stony Brook
TRIMAX	HDPE / Glass Fiber			1189	www.lumberlast.com
Lumber Last	Commingled recycled plastic	3755 (ultimate) (D198)	140 000 (D790)	1453 (ultimate) (D198)	www.ecpl.com
Earthcare recycled maid	Post-consumer milk jugs	0.79 (Density)	3205 (D695)	93 000–102 500 (D790)	Zarillo and Lockert (1993)
Hammer's plastic	80%HDPE/20%LDPE	2708	89 814		Zarillo and Lockert (1993)
	HDPE/LDPE (20PSGF)	4247	527 000		
	HDPE/LDPE (40PS20GF)	3514 (D695)	653 000 (D790)	1793 (D638)	
Superwood Selma, Alabama	33%HDPE/33%LDPE/33%PP	3468 (D695)	146 171 (D790)	1793 (D638)	Beck, R. (1993)

Table 2.4: Engineering properties of recycled plastic pins (Breslin et. al, 1998). (contd.)

Product	Composition	Compressive Strength (psi)	Modulus of Elasticity (psi)	Tensile Strength (psi)	Source
California recycling company	100% Commingled	81 717			Beck, R. (1993)
	10% Polypropylene	79 319			
	50% HDPE	92 636 (D790)			
RPL-A	HDPE/Glass fibers	2000			Smith and Kyanka (1994)
RPL-B	49% HDPE/51% wood fiber				Smith and Kyanka (1994)
Rutgers University	100% Curb tailings	3049	89 500		Renfee et al. (1989)
	60% Milk bottles, 15% Detergent bottles, 15% Curb tailings, 10% LDPE	3921	114 800		Renfee et al. (1989)
	50% Milk bottles, 50% Densified PS	4120 (D695)	164 000 (D790)		
Earth care products	HDPE		173 439 (D790)		www.ecpl.com
BTW recycled plastic lumber	Post-consumer	1840–2801	162 000		BTW/Hammers Brochure

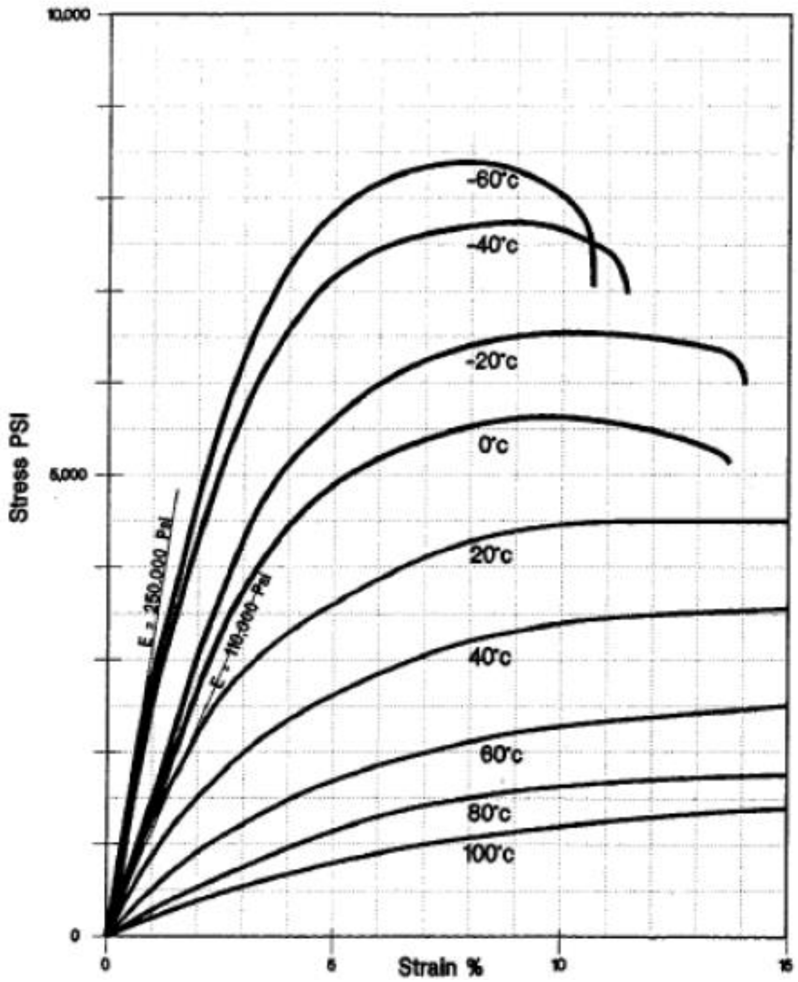


Figure 2. 4: Tensile strength of HDPE for different temperature (Malcolm, 1995).

Ahmed (2012) conducted a comparative study between RPP, wood and bamboo piles. Based on the study, wood showed to have highest compressive and flexural strength among these three alternatives; however, RPP has the advantage of facilitating greater soil movement which was found to be upto 19%. The most attractive part that led to the consideration of using RPP in place of other alternatives is its ability to perform well under different environmental and chemical conditions. The author reported that, for different environmental conditions the maximum decrease in RPP strength was only 8%, whereas for wood and bamboo the value was found to be about 50%

and 65% respectively. The durability and strength properties of RPP presents it to be a potential economic alternative over other materials.

2.2.4 Long Term Engineering Properties of RPP

Breslin et al. (1998) and Krishnaswamy and Francini (2000) conducted a study on the long term engineering properties of plastic lumber. During the study, the plastic lumber samples were removed from the deck over a two year period and returned to the laboratory for testing. At the beginning the authors investigated the initial engineering properties of recycled plastic pins that was manufactured by continuous extrusion process. For the long term properties test, the plastic lumbers were collected at 2 year intervals during monitoring periods. The authors reported that there was not any noticeable change such as warping, cracking and discoloration in the RPPs.

The effect of outdoor weathering and environmental effects including the degradation due to UV radiation, thermal expansion and combined effects of moisture and temperature on the mechanical behavior of RPP were determined. No significant variation of the flexural modulus and strength of RPP according to ASTM D6109 before and after the hydrothermal cycling, as presented in Table 2.5.

Table 2.5: Comparison of flexural properties of typical RPP materials with and without hygrothermal cycling (Krishnaswamy and Francini, 2000).

	Secant Modulus (psi)	Stress at 3% strain (psi)
Before cycling	97,800 ± 6,400	1,900 ± 120
After cycling	113,600 ± 14,400	2,400 ± 400

Lynch et al. (2001) conducted a study to investigate the effect of weathering on the mechanical behavior of recycled HDPE based plastic pins. A three point bending test was performed to obtain the flexural properties of weathered deck boards to compare against original flexural properties as per ASTM D796. Before the weathering action, the original flexural properties was determined to be 171 ksi for flexural modulus and 2.5 ksi for flexural strength. The three-point bending test results of the weathered samples obtained from the study are presented in table 2.6 and table 2.7.

Flexural properties of RPP when the exposed and unexposed side was tested in tension, are presented in table 2.13 and table 2.14 respectively. Comparison between the two results suggests that both modulus and strength increased after the outdoor exposure. Based on the results, it was found that the modulus increased by 28% and 25% when the exposed and unexposed sides are tested in tension respectively. In addition, for both cases, the strength at three percent strain increased by 4% from the original value.

Table 2.6: Results of three-point bending test of different RPP samples after weathering (exposed surface was subjected to tension) (Lynch et al., 2001).

Sample	Modulus (ksi)	Strength at 3% strain (ksi)	Ultimate strength (ksi)
1A	240.47	2.77	3.43
2A	213.79	2.48	3.12
3A	200.88	2.44	2.86
4A	214.22	2.55	3.32
5A	227.42	2.73	3.31
AVERAGE	219.30	2.59	3.21

Table 2.7: Results of three-point bending test of different RPP samples after weathering (unexposed surface was subjected to tension) (Lynch et al., 2001).

Sample	Modulus (ksi)	Strength at 3% strain (ksi)	Ultimate strength (ksi)
1B	217.56	2.77	3.49
2B	204.50	2.47	3.05
3B	190.29	2.45	3.05
4B	219.30	2.43	3.11
5B	234.67	2.76	3.25
AVERAGE	213.26	2.58	3.19

2.2.5 Creep of RPP

The recycled plastic pin is a nearly isotropic material having considerable strength, durability and workability which can be reinforced to increase the strength by forming a composite material. It is strong as wood; however, being visco-elastic material, it is susceptible to creep and deflection under sustained load. A study conducted by Malcolm (1995) showed the creep behavior of a 3.5 in. x 3.5 in. RPP under sustained mid span bending stress of 516.70 psi. Figure 2.5 presents the generated creep curve in his study.

According to Chen et al. (2007), variety in manufacturing process is responsible for variation in engineering properties of commercially available materials. The polymeric materials show higher creep compared to timber, concrete or steel, while they are more resilient against environmental degradation. Van Ness et al. (1998) tested on RPPs collected from various commercial sources and concluded that, RPP containing oriented glass fiber is more resistant against creep.

Lampo and Nosker (1997) reported that, for any load bearing application, creep is a serious concern while using RPP. Inheriting the viscoelastic properties of plastic, a plastic lumber will sag under static sustained loading which increases with increasing temperature. Civil engineers generally study this time dependent variable to develop load-duration factors for design purpose. To develop the design guideline for plastic lumber, this effect is extremely crucial which should be taken into account.

2.2.6 Effect of Environmental Conditions

Ahmed (2012) conducted a study at UTA to determine the axial compressive strength of RPP under different extreme environmental conditions. The uniaxial compressive strength test was performed on RPP in accordance with ASTM D6108. Both the normal samples and samples submerged in environmental chambers for two months were tested. Recycled plastic lumber reinforced with fiber glass was utilized, with a cross section size of 3.5 in x 3.5 in. The test samples were prepared as the specimen height = 2 x minimum width, in accordance with the ASTM standards. As the 3.5 inch samples were utilized for the all tests, the height of each sample was 7 inches.

Ahmed (2012) evaluated the environment effects, considering Acidic (pH = 5.5), Neutral (pH = 7.0) and Basic (pH = 8.5) conditions, which represent the pH of different clayey soils in Texas.

The samples were submerged in three large tubs filled with water, and basic and acidic solutions for two months to study the degradation behavior at an accelerated rate. For acidic and basic solutions, the samples were kept in sealed, covered containers inside the laboratory at room temperature (70° F). The pH of the solution was measured each week to monitor the variations of solution concentration. In addition, the pH was adjusted, to keep it in the target range, by adding an acidic and/or basic solution if any variation was observed. The samples submerged in neutral solutions (pH = 7.0) were kept open in a hot room, with no covers, to simulate the effect of heat and moisture on the samples. The temperature of the hot room was kept at 98° F, resembling the average temperature during summers in Texas.

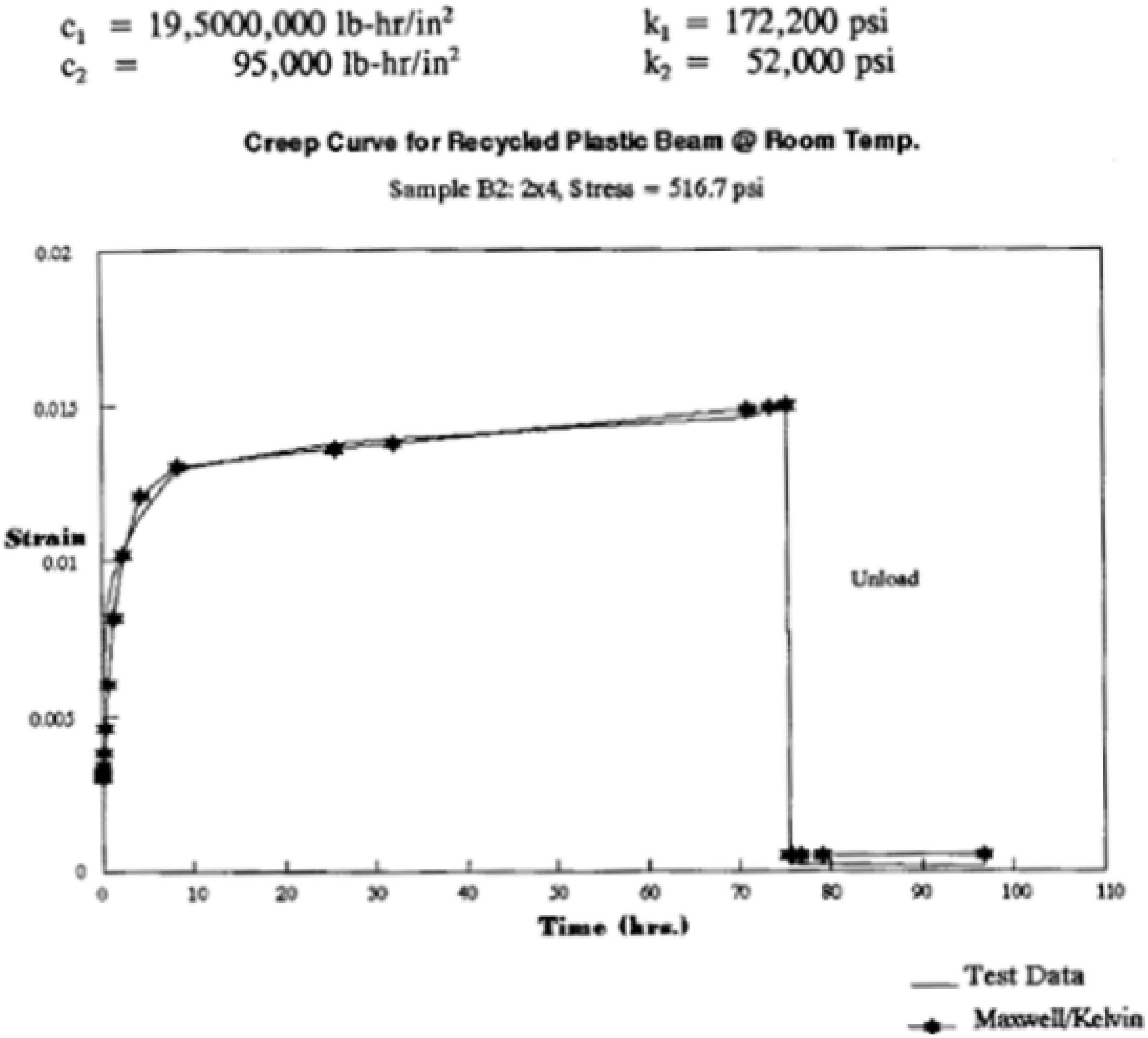


Figure 2.5: Creep behavior of RPP beam at room temperature (Malcolm, 1995).

The ASTM standards suggest maintaining a controlled strain rate during the test. Therefore, the applied loading rates, corresponding to the strain rates for the plastic lumbers as described in

ASTM D6108-09, were used as the upper limit of the test. The strain rate recommended for testing plastic pins for slope stability applications was utilized for the lower strain rate. The third strain rate was chosen in between two strain rate limits. Three samples were tested for each loading rate. The experimental program developed for this study is presented in Table 2.8.

Table 2.8: Compression Test on RPP at different Environmental Conditions (Ahmed, 2012).

Uniaxial Compression Test	Loading Rate	No of Tests
At Normal Condition	2.5 kips/min	3
Acidic Solution, pH 5.5	3.1 kips/min	3
Neutral Solution, pH 7.0		3
Alkaline Solution, pH 8.5	3.75 kips/min	3

Based on the results of Ahmed (2012), the highest axial strength was observed at the lowest loading rate of 2.5 kips/min under all environmental conditions for all of the samples tested. In addition, the peak strength decreased with increased loading rate, resulting in lower strength regardless of any environmental effect. The elastic modulus of RPP is considered as the initial slope of the stress strain curve. The stress-strain curve shifted to the left with the application of both acidic and basic conditions, signifying the increment of elastic modulus during axial compression. The stress strain curve of the sample in neutral condition also shifted to the left and followed a trend similar to both the acidic and basic conditions, except for the samples submerged in the neutral solution of pH 7.0 at loading rate of 2.5 kip/min. However, regardless of the environmental condition, no significant change in the peak strength of RPP was observed during the experimental period. The compressive strength of the RPP were observed almost constantly, regardless of the environmental condition, for all loading rates. It is important to note that RPP are composite material that contains mainly high density polyethylene, HDPE (55% – 70%), which is a non-degradable material. As a result, RPP experience almost no degradation due to environmental exposure.

The non-degradation behavior of RPP offer potential benefits for slope stabilization. When installed in the ground, RPP are exposed to different levels of pH, which over time usually tends to weather/degrade other construction materials, such as steel, concrete, or timber pile. However, as RPP are resistant to the differential pH variations, almost no degradation of the strength is expected over time, which will ensure a longer design life than other alternative construction materials.

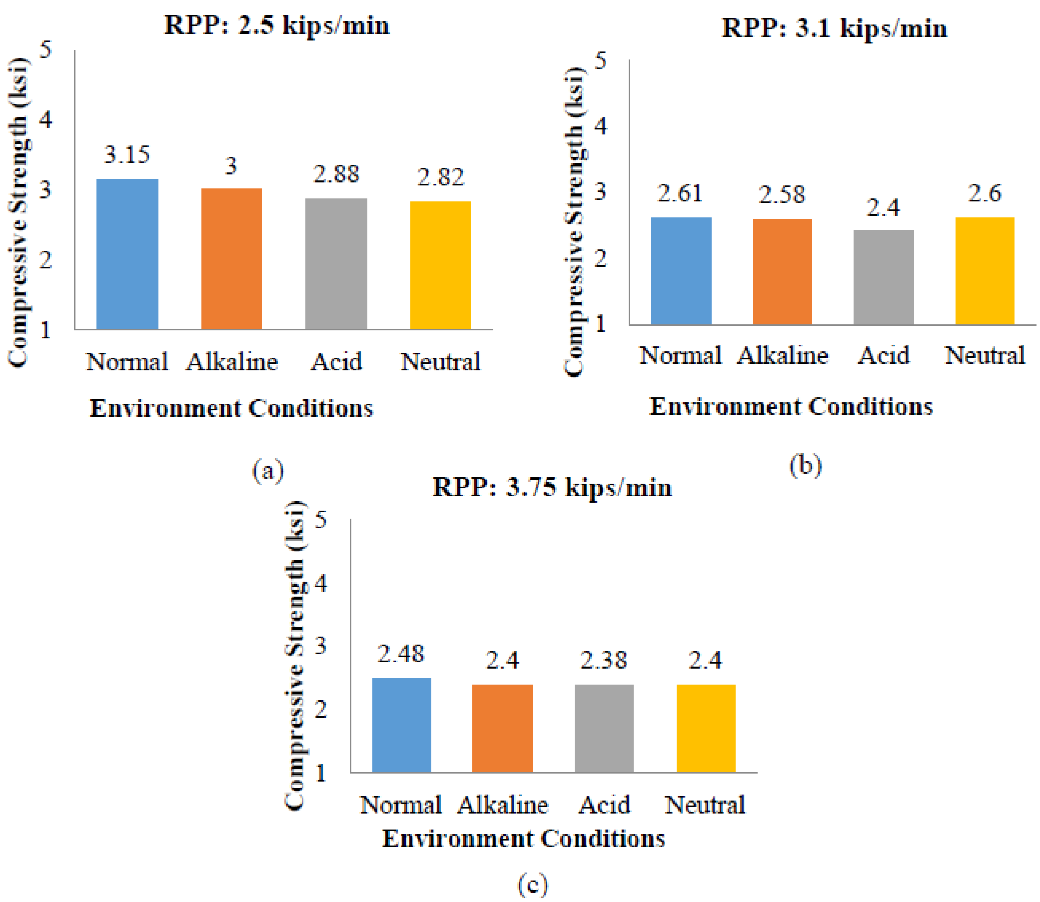


Figure 2.6: Compressive Strength of RPP at different Loading Rates and Environmental Conditions (Ahmed, 2012).

2.2.7 Design Consideration for Structural Application

Malcolm M. G. (1995) and Mclaren and Pensiero (1999) presented a simplified approach to design the recycled plastic as a structural material that is predominantly applicable to HDPE materials.

The design method included the load duration factor (LDF) similar to timber design to accommodate the effect of creep over the design period. The design method proposed the LDF ranged between 1.0 and 7.0, based on the time of application. On the other hand, the method included a temperature factor Ct, based on the effect of temperature over the tensile strength. The value of Ct ranged between 0.87 for 60 C (140 F) and 1.8 for 0 C (32 F) with Ct = 1 at 50 C (122 F). According to Malcolm, M. G. (1995), the allowable design stress Fa for Recycled Plastic Lumber is,

$$F_a=F_a*LDF*F' \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \text{(2.1)}$$

It should be noted that allowable stress is applicable for the structural member subjected to Dead and Live loads. However, it cannot explain the behavior of the RPP that could be installed in Slope and subjected to sustained lateral loads.

Lampo and Nosker (1997) addressed three main limitations of the designers planning for the structural application of RPP which are the lower modulus of RPP, creep and its co-efficient of thermal expansion. The author suggested accounting the first two issues by specifying a stiffened product or change in the design of support and spans. The thermal expansion can be taken care of by providing additional space for the expansion and contraction which is not a major issue for the RPP in slope stabilization application.

Nosker and Renfree (2000) presented the evaluation of the recycled plastic lumber and its applications on different civil engineering applications. For the successful utilization of the recycled plastic lumbers, the major concern for the structural application is the elastic modulus and the time dependent mechanical behavior (creep). To improve the mechanical properties and stiffness of the recycled plastics, the composites were produced. This initiative was first undertaken during 1990's where around 20-30% fiberglass were mixed in continuous extrusion process to produce stiffer product. The product had been successfully utilized for sheet piling, as structural plastic lumbers and for marine pilling.

Researches from Rutgers University developed a polymer-polymer composite with high stiffness and high strength during 1988-89 (Nosker et al., 1989) and later found that short glass fibers were capable of being oriented in a curbside tailing matrix, require about 10%-12% fiber glass to obtain high strength and stiffness value (Nosker et al., 1999).

Another innovation in the recycled plastic lumber had been conducted utilizing continuous glass fiber reinforcement with thermosetting plastics (in the shape of rebar) molded with HDPE (Lampo et al, 1998). The fiberglass members act as rebar supporting the less rigid thermoplastic material. The fiberglass rods are placed strategically and symmetrically about central axis. This technology has been used to produce marine piles which also performed well as fender pile.

2.3 Utilization and Superiority of Recycled Plastic Pin for Geotechnical Projects

Recycled Plastic Pins (RPP) are becoming more and more popular as a cost effective and sustainable solution for slope stabilization compared to conventional techniques (Loehr and Bowders, 2007, Khan, 2014). Compared to other piles, e.g. concrete or steel piles or other structural materials, RPP weighs much less and is more resistant to chemical and biological degradation. The compressive strength of each RPP is sufficient enough to carry vertical load from the structure above. In addition, previous studies showed the use of RPPs in the failed area of the slope to provide additional resistance along the sliding plane to increase factor of safety. The theoretical calculation as well as practical application proved that RPP is suitable to resist lateral load and increase the factor of safety of highway slopes. Therefore, it might also be used effectively as reinforcement to act as a shear key and provide additional resistance against sliding or lateral loading for any retaining type structure for example MSE retaining structures.

2.3.1 Slope Stabilization Using Recycled Plastic Pin

Parra et al. (2003) conducted a field performance study on slope sites that had been stabilized with RPP. The authors reported that the sites experienced recurring surficial sliding, ranging from depth of 3 ft. (0.9 m) to 5 ft. (1.5 m). It has to be noted that the soil in the sites were composed of mainly clayey soil. Khan (2014), had presented field performance and numerical modeling of RPP for shallow slope stabilization. Field performance of RPP based on their analysis are discussed in the following sub-sections.

2.3.1.1 Case Study 1: Interstate-70 (I-70) Emma Field Test Site

The test site is located on I-70, about 65 mi (105 km) west of Columbia, Missouri, having a slope height of 22 ft. (6.7 m) with 2.5H: 1V side slope that forms eastbound entrance ramp to I-70 in Saline County. The slope soil is composed of mixed lean clays with scattered cobbles and construction rubble (concrete & asphalt). The slope experienced recurring slides over the past few decades in four areas of the embankment, denoted as S1, S2, S3 and S4 as shown in the Figure 2.7a. Slide areas of S1 and S2 were considered and stabilized with RPP while area S3 and S4 served as control section. A 3 ft. (0.9m) staggered grid covered the failed area for stabilization based on the laboratory test results on soil samples and back analyzed failure condition. The installation of the RPP took place during November and December of 1999 and the installation was done approximately perpendicular to the slope. For the S1 area a total of 199 pins were installed and for S2 area total installed pins were 163; the layout is shown in Figure 2.7b.

Inclinometers were installed to monitor lateral displacement of the sections. Figure 2.8 presents the depth vs. cumulative lateral displacement and cumulative lateral displacement vs. time plot developed from field monitoring data. Based on the data, it was reported that, for the first year the movement was minimum followed by an increased maximum movement of about 0.8 inches (20 mm) during next 6 months. After that, the lateral movement became minimum. According to Parra et al. (2003), the control sections (S3 and S4) failed during late spring while in the reinforced sections, very small movement was observed.

2.3.1.2 Case Study 2: US 287 Slope Site

The slope site is located over Highway US 287, near the St. Paul overpass in Midlothian, Texas. The location is presented in Figure 2.67. The slope was constructed during 2003 – 2004 with a maximum slope height of about 30 to 35 ft. and a side slope of 3H: 1V. Cracks were observed near the shoulder during September 2010, which eventually resulted in the need for the slope to be stabilized to restrict further movement.

Three 50 ft. sections were selected and two of them were reinforced with RPP while the third one served as a control section. The layout of RPP installed in site US 287 is presented in Figure 2.9. Slope movement was monitored using three inclinometers for three sections.

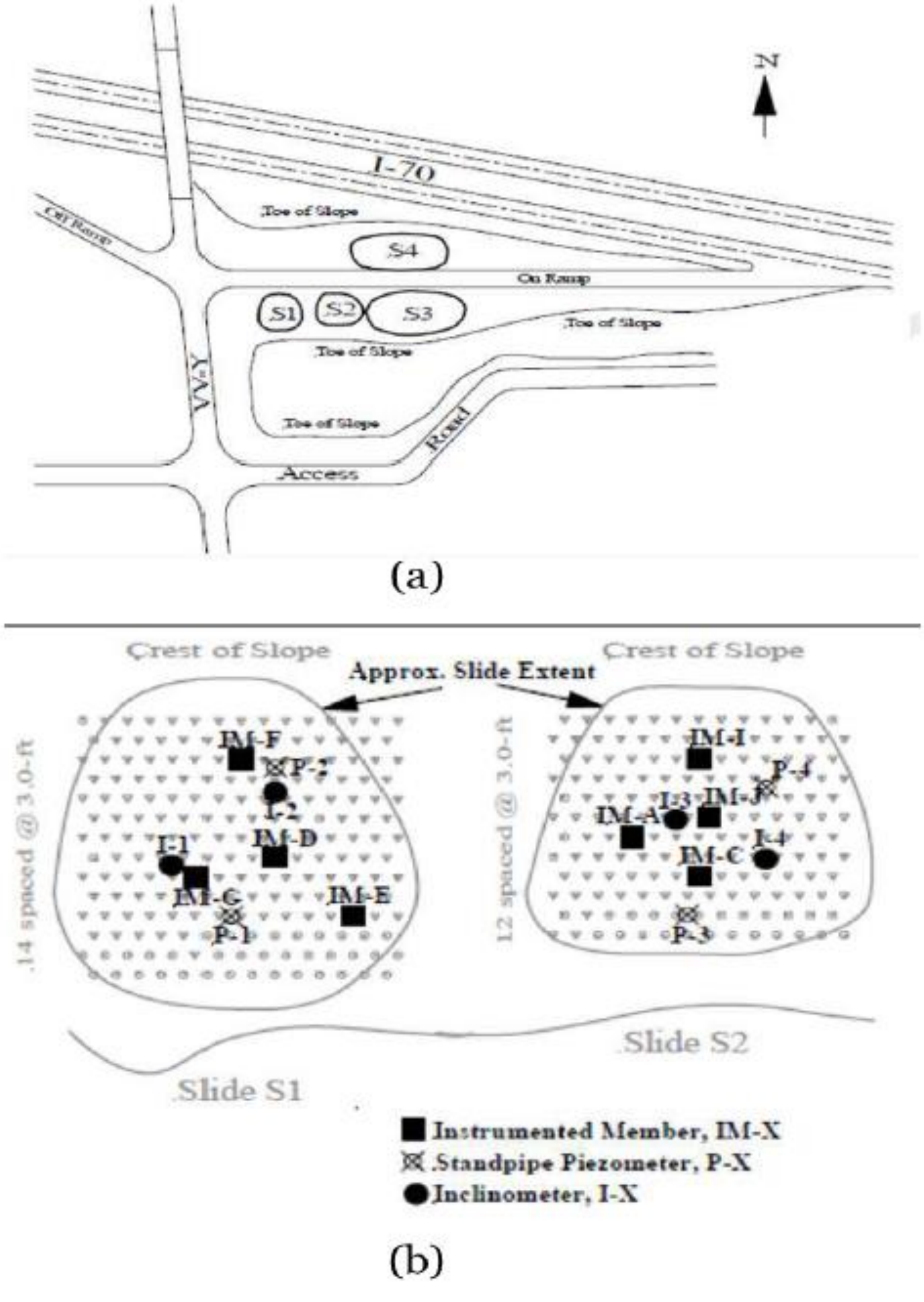


Figure 2.7: (a) I-70 site slide areas Location; (b) RPP layout plan for the slide area S1 & S2 (Parra et al., 2003).

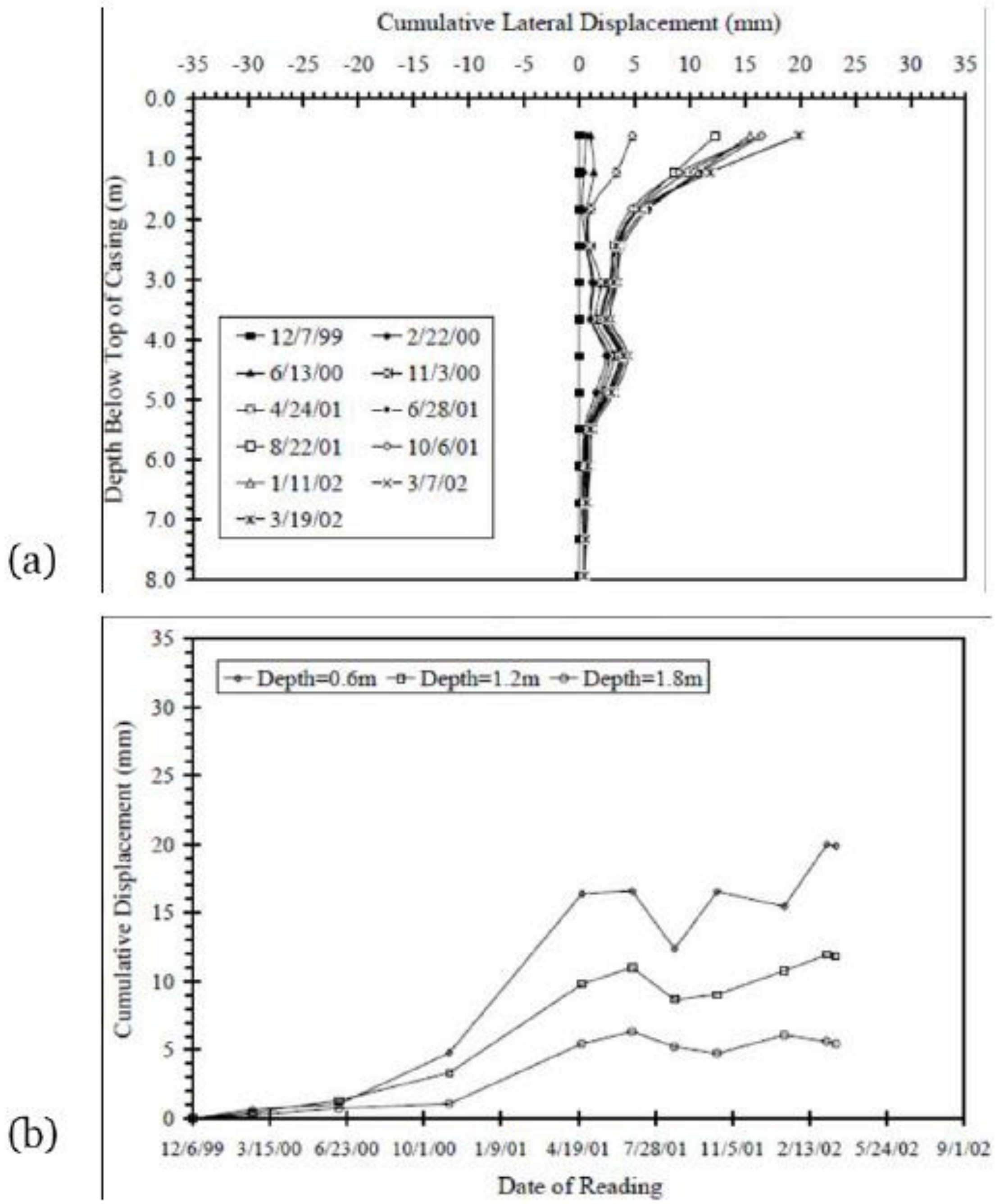


Figure 2.8: Performance monitoring from Inclinometer I-2 at I-70 Site (Parra et al., 2003).

The performance monitoring results of the US 287 slope indicated that the unreinforced control sections had significant settlement (as much as 15 inches) at the crest of the slope. In addition, a total of 3 inch increments in settlement had taken place during the year.

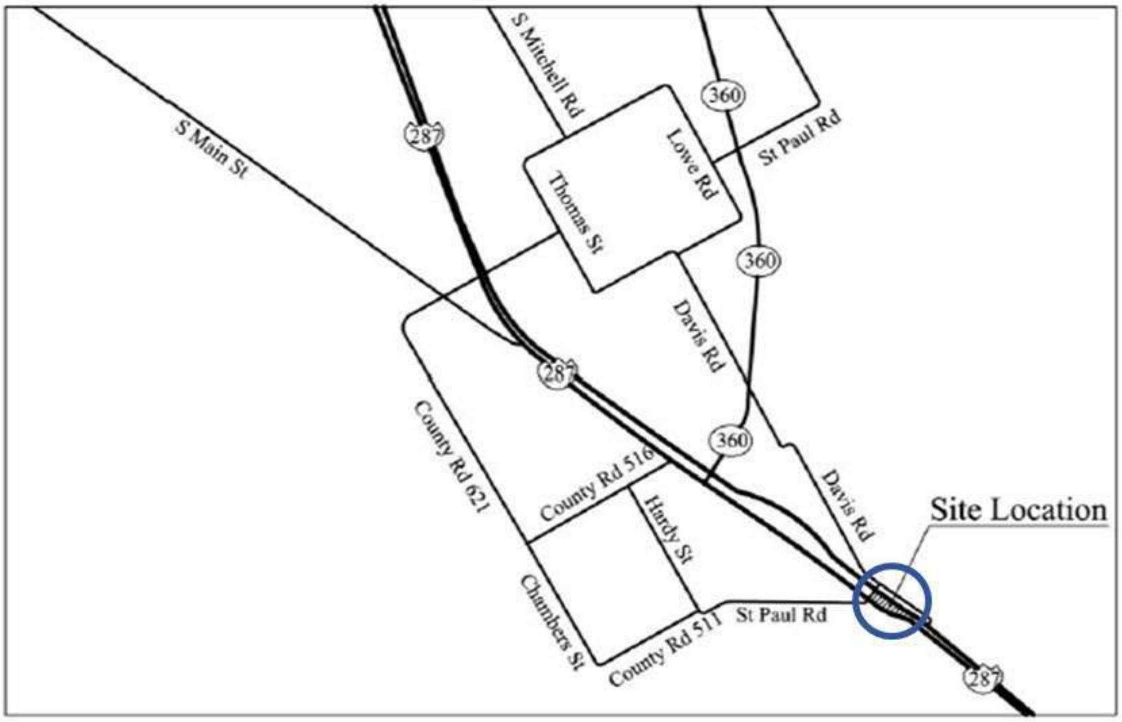


Figure 2.9: Site location map for the slope at US 287 (Khan, 2014).

However, almost no increment in settlement was observed at the reinforced section. A total settlement of the reinforced section was found to be 2 to 4 inch, which was significantly less compared to the unreinforced section. The lateral displacement of the test section had taken place after 1 year of construction which was about 1.5 inch. After 1 year the horizontal displacement became less than 0.1 inches in the reinforced section. A total of three inclinometers were installed to monitor the horizontal displacement; inclinometer 1 and 3 was installed in the reinforced section 1 and 2, while inclinometer 2 was installed in control section. From the inclinometer results, it was concluded that after the initial movement during the load mobilization period (little more than a year) the movement became almost constant. For the inclinometer 1, maximum horizontal movement was observed to be 1.3 inches while for inclinometer 3, it was found to be 1.8 inches.

2.3.2 Ground Improvement and Improving Sliding Resistance of MSE Wall Base

Bearing and shearing capacity failure is a common phenomenon for the structures constructed on unsuitable foundation soil, which leads to spending a significant portion of the budget for repair and maintenance for the related agencies. Structures (e.g. embankments, roadways, highways etc.) constructed on weak/soft foundation soil tend to experience excessive total and differential settlement due to not having sufficient support; on the other hand, MSE retaining structure constructed on stiff foundation soil, subjected to excessive lateral load is prone to sliding failure due to lack of shear resistance of the base.

The conventional techniques involved in improving the bearing capacity of the subgrade soil and shear resistance of the base of MSE wall might be either expensive in some instances or challenging to incorporate. Therefore, new, innovative, cost effective, and sustainable solution to the improvement of bearing and shearing capacity of the unsuitable soil are being tested increasingly. One such method could be the use of recycled plastic pins (RPP).

The study of Zaman (2019), conducted in two phases, presented an innovative and sustainable solution for minimizing both foundation settlement due to the application of embankment loading and lateral movement of the base of the MSE retaining structures using Recycled Plastic Pin (RPP). RPP is a light weight material, which is less susceptible to both chemical and biological degradation. It is moisture resistant and requires almost zero maintenance; these characteristics can present it as an attractive alternative compared to other available structural solutions (Krishnaswamy and Francini, 2005). Apart from the structural benefits, the use of RPP reduces the volume of non-degradable wastes entering the waste stream and provides additional market for the recycled materials (Loehr et al. 2000).

Zaman (2019) found that performance monitoring results of bearing capacity improvement from Phase–I showed a total settlement of 2.01 inches took place for the foundation of the control section; while for the reinforced section the settlement reduced considerably. Use of 4 in. x 4 in. RPP as foundation reinforcement reduced the settlement by 60%, while for 6 in. x 6 in. RPP, the reduction in settlement was found to be about 70%.Also the performance monitoring results from Phase–II presented similar conclusion. A reduction in settlement of about 56% was observed due to the use of 4 in. x 4 in. RPP compared to the control section. The settlement reduction depends on the strength of existing foundation soil.

In case of sliding resistance of MSE wall base, Zaman (2019) found that Phase–I represented the base of the control section experienced significant lateral movement (as much as 3.8 inches) during the monitoring period, while almost no movement was observed for the reinforced section. The performance monitoring results observed in the Phase–II presented similar outcome as observed in Phase–I. The control section experienced a lateral movement of 1.76 inches while horizontal movement of the reinforced section was much less (as low as 0.29 inches) compared to the control section. From the inclinometer data it was observed that the maximum horizontal movement was observed at the base level of the wall which reduces with depth under the ground level.

2.4 INSTALLATION METHOD OF RPP

2.4.1 Early Development of Installation Techniques

A series of laboratory and field tests were performed, using RPP to evaluate different installation methods. Both impact methods and vibratory methods were considered. Sommers et al., (2000) evaluated the impact driving technique in the laboratory, using a simple drop weight driving mechanism to drive a small-scale 4 cm x 4 cm RPP into a soil-filled drum. The laboratory drive test indicated that RPP are extremely resilient to driving stresses; however, the installation process was determined to be unsuitable for RPP.

A further evaluation of the installation technique was conducted, using the vibratory driving method. During this method, a slightly modified 27 kg (60 lb) pavement breaker was used to drive reduced-scale pins at a field test site near Columbia, Missouri (Sommers et al., 2000). The field test results indicated the resilience of RPP and also demonstrated that the penetration rates for the pseudo vibratory method far exceeded those of the drop hammer method.

Sommers et al. (2000) recommended the pseudo vibratory method for subsequent full-scale RPP installations and conducted further field scale tests, using the pseudo vibratory mechanism at a site in St. Joseph, Missouri. The site was located in the flood plain, and had two soil layers. The upper layer was approximately 1.5 m (5 ft) of compacted low plasticity clay (CL). The next layer of soil was natural alluvial deposit (CH) of highly plastic clay. The in situ dry densities of the site soils ranged from 86 lbs /ft3 to107 lbs /ft3. A total of seven 10 cm by 10 cm square pins of 1.2 m and 2.4 m length were driven at the site. A modified Indeco MES 351 hydraulic breaker mounted on a rubber-tired 835 Bobcat® skid loader was utilized as trial equipment. The penetration rates varied from a maximum of 12 ft/min to a minimum of 0.8 ft/min at the field trials due to varying soil conditions. The penetration rate was the highest in the soft, highly plastic alluvial deposits, with an average dry density of 86 lbs/ft3. The lowest penetration rates were observed for highly compacted low plasticity clays, with dry densities up to 106 lbs/ft3. The penetration rate was considered effective for large-scale field installation; however, the method had some limitations. The short wheel base caused equipment stability problems during installation, and could prove unsafe for installation in the slopes. In addition, the minimal headroom restricted the capacity to install RPP of more than 2.4 m (7.9 ft).

Based on the laboratory and field scale installation trials, it was evident that the pseudo vibratory method provides some advantages. However, due to the limitation of the tire-mounted equipment, the crawler-mounted systems were selected for further trials and proved to be the appropriate method for installation. The descriptions of the equipment that has been successfully utilized for field installations are presented in the following section.

2.4.2 Equipment and Tools for RPP Installation

Several field installations were conducted, using both the pseudo vibratory method and the impact driving technique. The field installations at the test sites indicated that RPP can be installed using either the driving or vibratory method, at a reasonable driving rate. This section describes the most appropriate equipment and its use in slope stabilization.

2.4.2.1 Davey-Kent DK 100B Drilling Rig

Installation of RPP was performed, using this equipment at a field demonstration site in Kansas City, Kansas, during November 1999. The rig was mounted on the crawler and equipped with a mast capable of a 50-degree tilt from vertical forward, 105-degree tilt backward, and side-to-side tilt of 32 degrees from vertical. The installation technique indicated that the mast system was very effective at maintaining the alignment of the hammer and the crawler and keeping them along the same line. That significantly reduced the chance that eccentric force would develop, thereby reducing the buckling of the RPP. The crawler-mounted rig also had a great benefit, which provided stability during the driving and was much easier to maneuver on the inclines (Sommers et al, 2000).

As a result, the required time to move the equipment from one point to another point was significantly less. The rig was equipped with a Krupp HB28A hydraulic hammer drill attached to the mast. The hydraulic hammer was capable of producing a maximum of 400 N-m (295 ft/lbs) of energy at a maximum frequency of 1,800 blows/min. The hammer energy was further enhanced by a push/pull of 8165 kg (18000 lbs) supplied by the drill mast. The field installation technique indicated that RPP installed in a vertical alignment were driven with the rig being backed up the slope. As a result, this feature lowered the installation rates for the pins driven vertically, as compared to pins driven perpendicular to the face of the slope.

2.4.2.2 Klemm 802 Drill Rig along with KD 1011 Percussion Head Drifter

Installation of RPP was performed using this rig at several field demonstration site locations around Dallas, Texas during March 2011. This specific type of the rig was selected based on the successful outcome of the DK 100B drilling rig. The Klemm 802 is a compact and multi-purpose drilling rig. The standard boom with its 6 x 90° swivel head allows the highest possible flexibility. It is equipped with an 18 ft long mast, which allows installation of RPP up to a length of 15 ft. The KD 1011drifter is a hydraulic hammer, which is equipped with two motors and can produce a blow frequency of 2800 blows/min. The single-blow energy for each of the KD 1011 drifters is 295 ft/lb.

The crawler-type rig is suitable for the installation over the slopes, as no additional anchorage is required to maintain the stability of the equipment. This reduces the amount of labor, cost, and time required for the installation process.

The installation of RPP, using the Klemm 802 drill rig, typically started from the crest of the slope. During the field demonstration process, it was observed that the crawler made it easy to maneuver the equipment on the slope and the setup time was significantly less when the installation started from the crest and the rig was gradually backed down toward the toe of the slope. In addition, during this process, the depression of the ground is much smaller due to the crawler movement.

2.4.2.3 Deer 200D with FRD, F22 Hydraulic Hammer

The crawler-mounted rig with mast equipped with a pseudo vibratory hammer works well for RPP installation. However, one major limitation is that this rig is not widely available and requires a special operator to install the RPP. A crawler-mounted rig with a pseudo vibratory hammer (model: Casagrande M9-1) was utilized to install the RPP in a slope stabilization project in Texas. However, the rig was not suitable due to the steepness of the slope at the crest. At the highest steepness (2.5H: 1V) near the crest of the slope, the crawler of the rig tended to tilt and lose ground contact during the RPP driving process. As a result, this rig was replaced with a crawler-mounted excavator which has greater stability on steep ground.

The crawler-mounted excavators, with extendible boom, offered several benefits. They had additional reach, which allowed the equipment to remain off the slope during installation, further limiting damage to the slope and reducing the setup time. An excavator with extendible boom also had a greater swing range than the track-mounted system, which allowed a larger number of pin installations, without movement of equipment and with reduction of the setup time.

The Deer 200D is a medium-size excavator with net power of 159 hp. The excavator was equipped with a hydraulic system, which could facilitate a hydraulic flow of 112 gal/min. and a FRD F22 hydraulic hammer capable of producing an impact energy of 4000 lbs/ft, with between 360 to 700 blows per minute. The hammer required a minimum hydraulic flow of 37 gal/min. The F22 hydraulic hammer produces impact energy 13.5 times higher than the KD 1011 percussion drifter, and requires a minimum hydraulic flow of 37 gal/min. The hammer needs a minimum pressure of 2320 psi to operate, which requires pushing the RPP into the ground to trigger the hammer impact for installation. During the field installations, stiff soil was encountered at some locations, and pushing the RPP into the ground using the powerful hammer caused buckling and cracking of the RPP.

This method was useful on soft ground, as the excavator was capable of pushing most of the RPP. Another limitation of the equipment is that the excavator does not have mast system. As a result, it required the expertise of the rig operator to keep the vertical alignment. At the beginning of installation, the wastage of RPP was higher; however, with time, as the operator get used to the installation technique, the wastage decreased significantly. Finally, a total of 130 RPP were installed with a wastage of less than 5%.

2.4.2.4 Caterpillar Rig CAT 32D LLR with CAT H130S Hydraulic Hammer

More than 500 RPP were installed in two different slopes in Dallas, Texas, using the CAT 320D LLR excavation, which is a medium-size excavator with net power of 148 h and features similar to the Deer 200D. The excavator was equipped with a hydraulic system, which can produce a hydraulic flow of 54 gal/min. and can generate a maximum lifting pressure up to 5221 psi. For the RPP installation, the excavator was equipped with a CAT H130S hydraulic hammer capable of producing an impact energy of 4500 lbs-ft, with between 320 and 600 blows per minute.

Similar to the F22 hydraulic hammer, this hammer requires a minimum pressure of 2030 psi to operate, which requires pushing the RPP into the ground to trigger the hammer to begin the installation. The impact driving method, using this system, was useful in the soft ground, as the excavator was capable of pushing most of the RPP. However, similar to the Deer 200D, this equipment doesn't have a mast system, which makes it difficult to drive the pin at the beginning. As the rig operator got used to keeping the alignment, the installation rate increased significantly.

A slope stabilization project, using a CAT 32D, was undertaken during the dry summer, and it was very difficult to drive the RPP into the hard soil layer. To overcome this problem, a full-size steel pin was manufactured and driven into the predefined locations. The steel pin was then withdrawn, leaving an empty hole, making it easy to push the RPP into the hole. This technique proved to be very effective in the dry clay soil and is discussed further at the end of this chapter.

2.4.3 Field Installation Rate

During the installation of RPP, the installation time was recorded to investigate the penetration or driving rate and installation rate. The penetration or driving rate of RPP considered the time required to drive the RPP into the ground, and the installation rate was considered the total time, which combined the time required to drive the RPP and to set up the equipment before installation. The crawler-mounted rig was easy to operate in the inclined ground and required no additional anchorage, thereby significantly reducing the setup time and increasing the installation rate of the RPP.

Sommers et al. (2000) indicated that the maximum penetration rate for driving RPP perpendicular to the slope was 10 ft/min., with an average rate of 5.2 ft/min. On the other hand, penetration rates for RPP driven vertically were only slightly lower, reaching 9.6 ft/min. and averaging 4.1 ft/min. It was also observed that it was easier to install the RPP perpendicular to the ground surface, as it provided more stability for the rig. Installation rates were also faster, with a maximum of 2.1 ft/min. at peak production. The average installation rate for installation of all pins was 1.33 ft/min.

The expertise of the rig operator plays a major role in controlling the installation rates, which generally increased with time as the construction team became familiar with the installation process. The installation rates for the field demonstration study performed in Missouri are summarized in Table 2.9.

Table 2.9: Summary of Penetration and Installation Rates from a RPP Installation Project in Missouri (Hossain et al., 2017).

	Penetration Rate (ft/min)	Installation Rate (ft/min)
Average Rate	4.64	1.37
Maximum Rate	10	2.1
Minimum Rate	0.12	0.55

Several RPP installation projects from 2012-2016 in the Dallas-Fort Worth area in Texas utilized both the vibratory method and the impact method (Khan 2013). A crawler-mounted rig, equipped with a mast-mounted pseudo vibratory hammer (Klemm 802 drill rig along with KD 1011 percussion head drifter) was used to install more than 600 RPP in a highway slope stabilization project. The installation rate of RPP was 2.85 ft/min., with 3 ft c/c spacing for 10 ft long RPP. The driving rate reduced to 2 ft/min. with an increase in RPP spacing of 6 ft c/c, due to the longer time required to maneuver the equipment between the higher spacing of the RPP. Conversely, the highest driving rate of 3 ft/min. was observed with 8 ft long RPP at 5 ft c/c spacing, which was located near the toe of the slope. The soil near the toe of the slope was very soft, and as a result, the time it took to drive RPP into the slope reduced drastically, resulting in the highest driving rate. The overall average driving rate for Reinforced Section 1 was observed as 2.72 ft/min. The installation rate of 10 ft long RPP with 4 ft c/c spacing was conducted at a different section, near the end of the 3-day installation process. The construction team became experts at installing the

RPP, which significantly reduced the time required for installation, as depicted in Table 2.10. Based on the pilot study in Texas, the average driving rate is 2.66 ft/min., which signifies that a 10 ft long RPP can be installed within 4 min, including the maneuvering of the crawler-mounted equipment. With this installation rate, on average, a total of 100 to 120 RPP could be installed per day.

Table 2.10: Average RPP Installation Rate in a Slope Stabilization Project in DFW Area in Texas (Hossain et al., 2017).

Length of RPP	RPP Spacing	Average RPP Driving Time	Average RPP Driving Rate
(ft)	(ft)	(min)	(ft/min)
10	3	3.55	2.9
10	4	2.76	3.6
10	6	4.76	2.1
8	4	3.08	2.6
8	5	2.63	3.1
8	6	3.65	2.2

Tamrakar (2015) conducted a study on the slope of SH-183. He found that driving time for the pins without hammering the iron nail was 22.33 minutes per pin. However, final pin installation time using the iron nail was between 1.81 and 5.64 minutes per pin. In the beginning, operators were having tough time to install the pins at the slope hence the longer time to hammer the iron nail and then drive the pins. Average time to hammer the iron nail into the ground was between 0.49 and 2.92 minutes per hole. In addition, the time to drive the 10 feet RPPs into the ground ranged between 1.23 and 2.71 minutes. These time includes the installation time and to maneuver the rig to the next points. At a time about 20 to 25 points would be hammered which are in the close approximately and the mold would be changed to drive the RPPs in the ground. In a day, approximately 100 pins can be installed if worked without any disturbance like equipment breakage and weather permitted.

Zaman (2019) studied the effect of RPP on improving bearing capacity and sliding resistance of MSE wall. It was observed that, average driving time for 4 in. x 4 in. section varied from 3.2 to 4.3 minutes whereas for 6 in. x 6 in. section it varied from 5.25 to 10.76 minutes. Larger size of RPP showed higher resistance as well as more energy and time required to install. Based on this study, the average driving rate for the whole test section with 4 in. x 4 in. was 2.87 ft. /min, which signifies that for vertical loaded sections a 10 ft. long 4 in. x 4 in. RPP could be installed within approximately 3.5 minutes. For locations like such, a total of approximately 115 to 135 RPPs of 4 in. x 4 in. can be installed each day. For the 6 in. x 6 in. RPP section, the average driving rate was 1.5 ft. /min, which signifies that for the test sections a 10 ft. long 6 in. x 6 in. RPP could be installed within 186 approximately 6.7 minutes. For similar site conditions, a total of approximately 50 to 70 RPPs of 6 in. x 6 in. can be installed each day.

2.4.4 Challenges to RPP Installation

Several field installations have already been conducted in Missouri, Iowa, and Texas. Some of the challenges observed during the RPP installation process are discussed in the following section.

2.4.4.1 Slope Steepness

The steepness of the slope has an influence on the installation process of RPP because it can cause instability of the rig. During the installations in North Texas, it was observed that the crawler-mounted rig with mast system was not suitable for installations on steep slopes because the vibration of the equipment caused it to lose contact with the ground. Moreover, the movement of the mast caused additional moment, during which the rig tended to tilt. This condition worsened during the bad weather. Due to rainfall, the slope, constructed using highly plastic clay soil, became saturated and soft, creating unfavorable conditions for operating equipment on top of it. Slopes on soft soil became slippery, and the crawler system was unable to grip the ground and became unstable. The crawler-mounted excavator with extendible boom was more stable on steep slopes.

2.4.4.2 Skilled Labor

Slope stabilization using RPP are sustainable technique. It is important to note that RPP are made of plastic and has a lower elasticity modulus than other alternative construction materials, such as steel. As a result, installation of the RPP, using powerful driving equipment, is not easy. Some trials have been run, driving RPP into the ground using a heavy jack hammer mounted with an excavator. It was observed that in this case, the installation of RPP required a gentle push and drive into the ground, rather than just hammering it at the top. Repeated hammering at the top of the RPP can often cause buckling and permanent deformation of RPP. A new construction team may lack experience in maneuvering the crawler-mounted excavator, resulting in lower installation rates (<0.75 ft/min) and high wastage (20%-25%). Coordination among members of the construction team is also crucial and influences the installation rate. As the construction team gets used to the installation process, however, the installation rate increases noticeably (>3 ft/min.), and wastage of RPP can be reduced to less than 3%. As the use of slope stabilization techniques using RPP increase, more contractors will become adept at implementing the installation process. Until then, it is recommended that the same contractor be employed for similar projects, if possible.

2.4.4.3 Connection between the Hammer and Pile Head

Several mechanical problems slowed the installation progress at the demonstration site in Texas. A drive head was fabricated, which served as a connection between the hammer and the RPP. It failed at various times during the installation, slowing down the installation.

In the early stages of installation with the mast-mounted hammers, a welded drive head was used as a connector and allowed the transfer of energy from the hammer to the RPP. Eventually, the repetitive impact from the hammer caused the welded connections to fail. Several spare connectors were kept on site; however, construction was slowed down due to repetitive failures of the welding joints. Later, a new hammer head was fabricated, using a solid stainless steel mold, without any joint or welded parts. The solid drive head worked well, and more than 300 RPP were installed, using the solid drive head without any interruption or sign of disturbance.

2.4.5 Special Installation Techniques

The highly plastic clay soil absorbs and retains water during wet periods, which results in the top soil becoming soft. The vibratory and impact methods work really well, when the installation is performed during the wet period and the top soil layer is relatively soft. However, during elongated summers, especially hot and dry ones like those in Texas, the clay soil loses moisture from the top, making the soil very stiff and causing an unfavorable condition for driving the RPP into the ground.

A highway slope stabilization program was undertaken in North Texas, during the month of September, immediately after an elongated summer where temperatures were greater than 90 F for more than two months. During the installation project, a caterpillar rig CAT 32D LLR with a CAT H130S hydraulic hammer was utilized to drive RPP into the ground. As the soil was stiff during the dry weather, the installation rate reduced significantly (<0.5 ft/min), while buckling, permanent deformation, and breakage of RPP increased significantly, as presented in Figure 2.10.

Figure 2.10: Bucking and Permanent Deformation of the RPP in Dry Soil (Hossain et al., 2017).

To overcome this problem, a full-size steel pin was manufactured to drive into the required/ predefined locations. After driving the steel pin, it should be withdrawn immediately, leaving an empty hole where the RPP could be pushed in easily. A 7 ft long steel pin was welded to the drive head and attached with the hammer, using a steel chain. The steel pin was driven into the same location prior to RPP installation, to make the hole in the stiff soil layer up to 7 ft. Later, the steel pin was pulled out from the ground, and the RPP were pushed in immediately. This procedure of installation worked well, and more than 500 RPP were installed, with an installation rate of more than 2 ft/min. Photos of the RPP installation are presented in Figure 2.11.

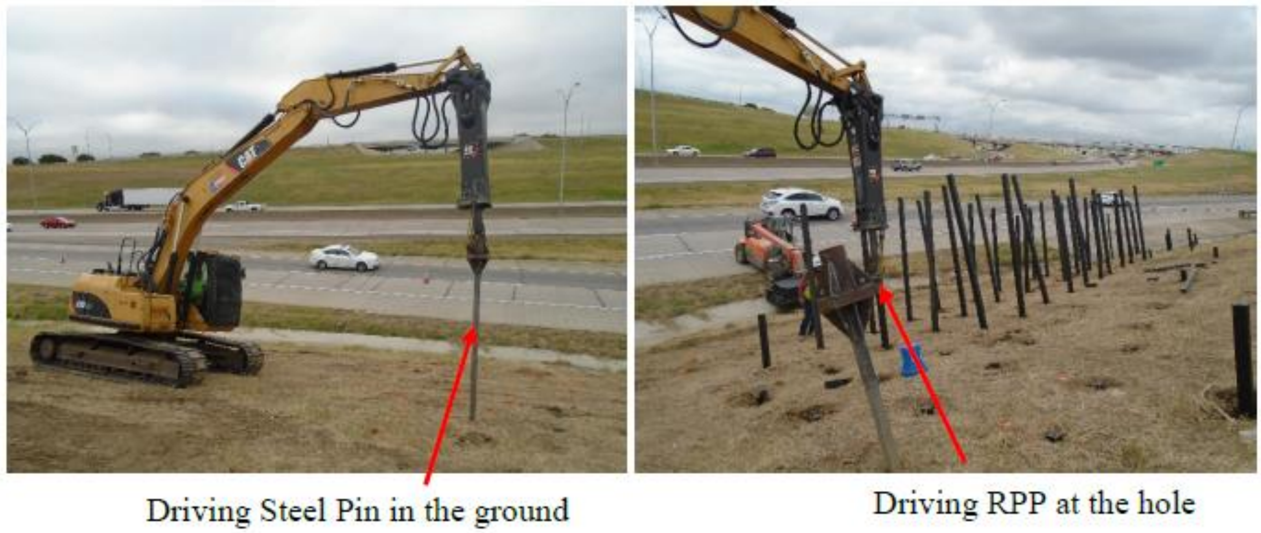


Figure 2.11: Plastic Pin Installation at SH 183 Slope (Hossain et al., 2017).

2.5 Drivability of Piles in Soil

Pile drivability represents the ability of a pile to be safely and economically driven to the required depth without causing excessive fatigue damage. The analysis for a particular set of driving equipment, pile material and dimensions, and a specific type of soil at the site involves a detailed static and dynamic soil resistance input parameters to reflect layers that pile penetrates. Predicting Soil Resistance to Driving (SRD) has been a challenging task and some of the methods used nowadays include procedures given by Toolan and Fox (1977), Stevens (1982), Alm and Hamre (2001).

The total resistance to driving may be divided in a static part, the static resistance to driving (SRD) and a velocity or displacement rate dependent part called the damping. Evaluation and development of correct input of static resistance is of high importance to obtain an accurate model. In order to determine SRD, common practice is to relate it to the Static Soil Resistance; American Petroleum Institute (API) proposes such methods. There are number of methods presented over the years and are still in use in North Sea pile design.

The earliest models like Toolan and Fox (1977) did not include friction fatigue concept, which was presented in 1978 by Heerema who made drivability prediction based on the assumption that skin friction in clay is gradually lost along the pile wall as driving proceeds (Heerema, 1978). Semple and Gemeinhardt's method from 1981 related unit skin friction to clay stress history (Semple and Gemeinhardt, 1981). In 1982, Stevens adopted model by Semple and Gemeinhardt. The methods mentioned above are referred to as traditional methods, while recently developed models are usually based on CPT data (Alm and Hamre, 1998).

According to Fenske and Hirsch (2003) the analysis of pile drivability consists of three phases or steps. The first step is to use an analysis based on the one-dimensional wave equation to estimate the resistance that can be overcome by the particular hammer pile-soil system. The second step is to evaluate the specific soil conditions at the location to estimate the resistance that the soil will offer to the forced penetration of the pile. The third step is to compare the resistance the hammer-pile-soil system can overcome with the resistance that the soil can offer in order to obtain an indication whether the pile can be driven to the desired penetration. Engineers should be aware that a drivability analysis does not necessarily produce a definite answer to the pile drivability question. Considerable engineering judgement is required for all three steps of a drivability analysis, and everyone making a drivability analysis may not arrive at exactly the same conclusions. A drivability analysis should be made for each specific combination of hammer, pile and soil conditions being considered for a project.

2.5.1 Methods for Assessing Pile Drivability

There are three methods used to evaluate pile drivability which include:

- Wave equation analysis
- Dynamic testing and analysis
- Static loads tests

2.5.2 Factors Influencing Pile Drivability

There are two major drivability features that a pile need to meet namely pile stiffness and pile strength. The pile need to have adequate stiffness to be able to transfer sufficient driving force that overcomes soil resistance. Regarding pile strength, it should be large enough so as the pile can withstand the driving force without suffering any damages.

Pile impedance (EA/C) is the major factor that controls pile drivability (Varma et al., 2013). The modulus of elasticity (E) of pile is specified based on the material used for the construction, (C) is the pile wave speed, and (A) is the cross-sectional area of the pile which seems to be the only parameter by which pile drivability can be improved. The inverse of pile impedance is defined as mobility (N). Change in impedance is related to change in pile cross-sectional area A, as well as pile material quality. Pile driven system features such as speed, stroke, ram weight, and real performance of pile driving system on the construction site would influence pile drivability to a certain extent.

The improvement of pile drivability though increasing cross sectional area can be clearly observed when steel pile is selected. For example, the increase of the thickness of steel pipe pile will enhance the pile drivability. However, when the area of reinforced concrete section is increased, the soil resistance would also increase.

2.6 IN-SITU TESTING

The interpretations of initial geostatic stress state and stress-strain-strength-flow characteristics can be obtained with laboratory test data on high quality samples (Mayne, 2004). However these are often done at high costs, and also the accuracy of geotechnical parameters measured from laboratory testing had been debated extensively over the last three decades. A growing awareness of this fact led to an increasing interest in all forms of in situ testing, where the disturbance of the soil structure is minimal. In situ testing is rapidly emerging as a viable alternative to the traditional approach of obtaining geotechnical parameters for design and analysis (Crawford and Campanella, 1990, Bergado et al., 1991). In recent years, some researchers have indicated the existence of a strong correlation between the predicted results from some of the insitu test methods and the observed results from the field. Bergado et al., (1991) investigated the usefulness of the screw plate and pressure meter tests to provide meaningful results for the prediction of embankment settlement on soft clays. The settlement predictions were generally in good agreement with the observed field settlement. LeClair et al. (1999) utilized flat dilatometer, piezocone, and screw plate tests to predict consolidation settlements of embankments at Vancouver International Airport. The authors concluded that settlement magnitudes can be predicted with reasonable confidence based on the parameters interpreted from in situ tests.

The following types of ground conditions are examples of those where in situ testing is either essential or desirable:

- Very soft or sensitive clay
- Stoney soils
- Sands and gravels
- Weak, fissile or fractured rock

In situ tests may be classified in a number of ways - cost, ease of use, method of interpretation, soil types in which they may be used, parameters which can be determined, etc. A classification can be established on the basis of the degree to which tests can be analyzed in a fundamental way to obtain real soil parameters, which is a function not only of how the test is applied to the soil, but also of the type of data collected. On this basis, the tests can be classified as explained below:

1. **Wholly empirical interpretation**: No fundamental analysis is possible. Stress paths, strain levels, drainage conditions, and rate of loading are either uncontrolled or inappropriate (Examples: SPT, CPT).
2. **Semi-analytical interpretation**: Some relationships between parameters and measurements may be developed, but in reality, interpretation is semi-empirical either, because both stress paths and strain levels vary widely within the mass of ground under test, or drainage is uncontrolled, or inappropriate shearing rates are used. (Examples: plate test, vane test.)
3. **Analytical interpretation**: Stress paths are controlled and similar although strain levels and drainage are not (Example: self-boring pressure meter).

Many forms of in situ penetration test are in use worldwide. Penetrometers can be divided into two broad groups:

1. Dynamic penetrometers: simplest form consisting of tubes or solid points driven by repeated blows of a drop weight.
2. Static penetrometers: more complex being pushed hydraulically into the soil.

The most common penetration tests are:

- Dynamic penetration test: Standard Penetration Test (SPT)
- Static penetration test: Dutch Cone Penetration Test (DCP)
- Quasi-static penetration test: Cone Penetration Test (CPT)

In Texas the widely used in-situ penetration is the Texas Cone Penetration Test (TCP) which is a modified version of Standard Penetration Test (SPT).

2.6.1 Standard Penetration Test (SPT)

The standard penetration test, developed around 1927, is currently the most popular and economical means to obtain subsurface information (both on land and offshore). It is estimated that 85 to 90 percent of conventional foundation design in North and South America is made using the SPT. This test is also widely used in other geographic regions. The method has been standardized as ASTM D 1586 since 1958 with periodic revisions to date. The test consists of the following:

1. Driving the standard split-barrel sampler a distance of 460 mm into the soil at the bottom of the boring.
2. Counting the number of blows to drive the sampler the last two 150 mm distances (total = 300 mm) to obtain the *N* number.4
3. Using a 63.5-kg driving mass (or hammer) falling "free" from a height of 760 mm.

The standard penetration test is done using a split- spoon sampler in a borehole / auger hole. This sampler consists of a driving shoe, a split- barrel of circular cross-section (longitudinally split into two parts) and a coupling. The procedure for carrying out the standard penetration test is discussed as follows:

- SPT uses a thick-walled sample tube, with an outside diameter of 51 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a weight of 63.5 kg (140 lb) falling through a distance of 760 mm. Figure 5.2 shows schematic representation of SPT setup and testing.
- The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm up to a depth of 450 mm is recorded. The sum of the number of blows required for the second and third 150 mm. of penetration is termed the "standard penetration resistance" or the "N-value".
- In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval, the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground.
- A borehole is dug to the required depth and the bottom of the hole is cleaned. The split-spoon sampler, attached to the drill-rods of required length is lowered into the borehole and is relaxed at the bottom.
- The sampler is then, driven to a distance of 450 mm in three intervals of 150 mm each. This is done by dropping a hammer of 63.5 kg from a height of 762 mm (BIS: 2131, 1981). The number of blows required to penetrate the soil is noted down for the last 300 mm, and

this is recorded as the N value. The number of blows required to penetrate the sampler through the first 150 mm is called the seating drive and is disregarded. This is because the soil for the first 150 mm is disturbed and is ineffective for the SPT- N value.

- The sampler is then pulled out and is detached from the drill rods. The soil sample, within the split barrel, is collected taking all precautions not to disturb the moisture content and is then transported to the laboratory, for tests. Sometimes, a thin liner is placed inside the split barrel. This makes it feasible for collecting the soil sample within the liner, by sealing off both the ends of the liner with molten wax and then taking it away for laboratory test of the contained soil.
- The standard penetration test is, performed at every 0.75 m intervals in a borehole. If the depth of the borehole is large, however, the interval can be, made 1.50 m. In case the soil under consideration consists of rocks or boulders, the SPT- N value can be recorded for the first 300 mm. The test is stopped if:
    1. 50 blows are required for any 150 mm penetration
    2. 100 blows are required for any 300 mm penetration
    3. 10 consecutive blows produce no advance
- However, it should be noted that the SPT- N value obtained from the above set of procedures has to be corrected before it can be used for any of the empirical relations. These corrections and their values for certain conditions are as follows:
    - The SPT data collected is field N‘values without applying any corrections. Usually for engineering use of site response studies and liquefaction analysis, the SPT ―N‖ values have to be corrected with various corrections and a seismic bore log has to be obtained.

- The seismic bore log contains information about depth, observed SPT N‘ values, density of soil, total stress, effective stress, fines content, correction factors for observed ―N‖ values, and corrected ―N‖ value.
- The ‗N‘ values measured in the field using Standard penetration test procedure have been, corrected for various corrections, such as:
    i. Overburden Pressure (C_N),
    ii. Hammer energy (C_E),
    iii. Borehole diameter (C_B),
    iv. Presence or absence of liner (C_S),
    v. Rod length (C_R) and
    vi. Fines content (C_{fines})

- Corrected N‘ value i.e., $(N_1)_{60}$ is obtained using the following equation:

$$(N_1)_{60} = N \times (C_N \times C_E \times C_B \times C_S \times C_R)$$

2.6.1.1 Advantages of SPT

- Many existing correlations
- Most contractors are capable of SPT testing
- Obtain sample by using the split spoon sampler of material and that can be tested to get soil properties
- Relatively cheap
- Robust
- Suitable for most soils
- Only investigation provides soil strength with soil sample; one can feel the soil

2.6.1.2 Disadvantages of SPT

- Ground at the base of borehole is disturbed by drilling process
- Prone to errors by drillers (e.g. water head, depth measurement errors)
- Device imposes very complex strain paths to the soil and no theory at present is capable of predicting what are the most influential factors affecting the N value.

2.6.2 Cone Penetration Test (CPT)

Cone Penetration Test (CPT) is an in-situ test done to determine the soil properties and to get the soil stratigraphy. This test was initially developed by the Dutch Laboratory for Soil Mechanics (in 1955) and hence it is sometimes known as the Dutch cone test. On a broad scale, the CPT test can be, divided into two:

1. Static Cone Penetration Test (BIS-4968, Part - 3, 1976): The cone with an apex angle of 60° and an end area of 10 cm^2 will be pushed through the ground at a controlled rate (2 cm/sec). The cone is pushed into the ground and not driven. During the penetration of cone penetrometer through the ground surface, the forces on the cone tip (qc) and sleeve friction (fs) are measured. The measurements are, carried out using electronic transfer and data logging with a measurement frequency that can secure the detailed data about soil contents and its characteristics. The Friction Ratio (FR = fs/qc) will vary with soil type and it is, also an important parameter.
2. Dynamic Cone Penetration Test: Dynamic test will be, conducted by driving the cone using hammer blows. The dynamic cone resistance will be, estimated by measuring the number of blows required for driving the cone through a specified distance. Usually, this test will

be, performed with a 50 mm cone without bentonite slurry or using a 65 mm cone with bentonite slurry. The hammer weighs 65 kg and the height of fall is 75 cm. The test will be, done in a cased borehole to eliminate the skin friction.

There are several variants of the basic cone penetrometer. Most popular three of them are:

1. Piezocone
2. Seismic cone and
3. Vision cone.

2.6.2.1 Advantages of CPT

- Many existing correlations
- Measurements allow soil classification but calibration boreholes preferred
- q_c values etc. , are computer logged and not drilling or driller dependent
- Capable of picking up the presence of thin sand/clay lenses
- Measurements may be related theoretically (at least qualitatively) with soil parameters such as OCR and D_r
- Allows in-situ determinations of the (reloading) horizontal coefficient of consolidation
- Relatively cheap and very quick.

2.6.2.2 Disadvantages of CPT

- Need to provide reaction for insertion of cone (typically ≈5t)
- Not ideally suited to Stoney ground
- De-saturation of the pore pressure sensor in dilatant clays
- Upkeep of instruments (+ their calibration): time consuming/expensive.

2.6.3 Texas Cone Penetration Test (TCP)

The Texas Cone Penetrometer is commonly used in site investigations by the Texas Department of Transportation. The TCP test involves driving a hardened conical point into soil and hard rock by dropping a 170 lb (77 kg) hammer a height of 2 feet (0.6 m) (Tex 132-E). From the soil test, a penetration resistance or blow count (N_{TCP}) is obtained which equals the number of blows of the hammer for the first 6 inches (150 mm) and the second 6 inches (150 mm) of penetration.

The relationship developed by Touma and Reese (1969) between SPT and TCP in cohesive and cohesion less soils is summarized in Tables 2.1 and 2.2. The N values of SPT and TCP at different soil density classifications are also summarized in the tables 2.11 and 2.12.

Table 2.11: Existing Correlations between SPT and TCP for Cohesion less Soils (Touma and Reese (1969))

Soil Classification	N_{SPT}	N_{TCP}	Relationship between SPT & TCP
Very Loose	0 to 4	0 to 8	$N_{SPT} = 0.5\ N_{TCP}$
Loose	4 to 10	8 to 20	$N_{SPT} = 0.5\ N_{TCP}$
Medium	10 to 30	20 to 60	$N_{SPT} = 0.5\ N_{TCP}$
Dense	30 to 50	60 to 100	$N_{SPT} = 0.5\ N_{TCP}$
Very Dense	> 50	> 100	$N_{SPT} = 0.5\ N_{TCP}$

Table 2.12: Existing Correlations between SPT and TCP for Cohesive Soils (Touma and Reese (1969)).

Soil Classification	N_{SPT}	N_{TCP}	Relationship between SPT & TCP
Very Soft	< 2	< 3	$N_{SPT} = 0.7\ N_{TCP}$
Soft to Medium	2 to 8	3 to 11	$N_{SPT} = 0.7\ N_{TCP}$
Stiff	8 to 15	11 to 21	$N_{SPT} = 0.7\ N_{TCP}$
Very Stiff	15 to 30	21 to 43	$N_{SPT} = 0.7\ N_{TCP}$
Hard	> 30	> 43	$N_{SPT} = 0.7\ N_{TCP}$

2.7 Essence of Time Estimation in Construction Management

Many variables have an impact upon construction cost overrun. Among those time is one of the most dominating factors (Kaming et al., 1995). Proper estimation of time in an early stage of planning enables it to foresee the management loop holes in construction work. The intentions of time estimation therefore are:

1. To identify time dependent variables influencing construction cost overrun.
2. To group these variables into factors.
3. To analyze the relationship of these factors and thereby enhance understanding of construction delays and cost overruns.

2.8 Limitations of Previous Studies and Room for Future Study

In recent time, the innovative use of recycled plastic pin (RPP) has become prominent in slope stabilization. Also, its potential uses in ground improvement and sliding resistance of MSE wall are about to come into practice. Therefore it is necessary to correlate the drivability of RPP with installation time.

The previous studies on RPP discovers its properties and methods for design in slope stabilization as well as its effectiveness in ground improvement and sliding resistance. However, no study is found to connect its driving mechanism with basic soil properties. Drivability of RPP depends on a number of factors, e.g. RPP properties, soil and site conditions, installation method, workmanship, weather, machine used in installation etc. Site condition is examined by in-situ method nowadays increasingly. Additionally, soil properties can be found from in-situ testing from correlations. Therefore, a study is necessary to correlate driving rate of RPP with soil properties and in-situ testing (SPT) result.

Chapter 3

METHODOLOGY

3.1 INTRODUCTION

The objective of the current study was to correlate the driving rate of recycled plastic pin (RPP) with soil properties and estimate driving rate from the SPT value of the site soil. To meet this objective, driving time data collected since the installation process in different test sections situated in North Texas region were used. Also, the properties of respective site soil were evaluated and used to make correlation with the driving rate of RPP. A rigorous statistical analysis was performed with a view to establishing a relationship so that driving rate of RPP can be estimated from SPT value of the site soil.

Figure 3.1: Locations for test sections.

Four areas of North Texas namely Dallas, Denton, Arlington, and Irving were selected as study area. There were four different locations in Dallas where test sections situated namely US 287, New US 287, Loop 12, and I-35. In Arlington three test sections were I-30 Fielder North, I-30 Fielder South, and I-20 Park Spring Boulevard. The test sections of Denton and Irving were in the City of Denton Landfill and the City of Irving Hunter Ferrell Landfill respectively.

3.2 DATA COLLECTION

The driving time and consequent driving rate of recycled plastic pin (RPP) as well as respective soil properties were collected in two ways. Data for the test sections of Dallas and Denton were collected from previous study while the data of Irving and Arlington test sections were collected directly at time of installation. This section of the chapter discussed the various sources and types of data collected from different test sections in different locations.

3.2.1 Test Sections in Dallas

3.2.1.1 US 287

The test section located in Highway US 287 near the St. Paul overpass in Midlothian, Texas was studied by Khan (2013). The filled slope was constructed during year 2003-2004. The maximum height was about 30 feet to 35 feet with slope geometry of 3 (H): 1(V).

The driving time and rate of RPP for the slope test sections and the properties of site soil were collected from the study of Khan (2013). The field installation of RPP was carried out during February, 2012. The size of RPP installed was 4 in. x 4 in and the spacing used were 2 feet, 3 feet, 4 feet, 5 feet, and 6 feet. These are summarized in Table 3.1.

Table 3.1: Driving rate and geometric properties of RPP installed in US 287, Dallas (Khan, 2013).

Test section ID	RPP Cross section (square inch)	Driven length (ft.)	Spacing (ft.)	Average driving rate (ft/min)
Reinforced section 1	4 in x 4 in	10	3	2.9
		10	6	2.1
		8	6	2.2
		8	5	3.1
Reinforced section 2	4 in x 4 in	10	4	3.6
		8	4	2.6
Reinforced section 3	4 in x 4 in	10	4	2.2
		12	3	2.1

Three boreholes namely BH_1, BH_2 and BH_3 represented the site soil properties (Khan, 2013). SPT values from BH_1 and BH_3 were interpolated to get values for RPPs in the middle of the section. As the maximum driven length of RPP is 10 feet, SPT value up to 10 feet was considered. Also the depth wise average SPT was taken for analysis. Moisture content, liquid limit, plastic limit, and plasticity index of the soil were collected which are summarized in Table 3.2.

Table 3.2: Soil properties of site in US 287, Dallas (Khan, 2013).

Bore hole ID	Depth (ft.)	Moisture content (%)	Plasticity index	SPT value	Dry density (pcf)	Cohesion (psf)
BH_1	5	22.2	25	5	101	485
	10	28.6	33	8	115	495
Avg.		25.4	29	7	108	490
BH_2	5	18.8	23	4	99	488
	10	34	38	5	107	452
Avg.		26.4	30.5	5	103	470
BH_3	5	20.2	27	5	107	521
	10	29.9	34	5	101	479
Avg.		20.2	27	5	104	500

Table 3.3: Driving rate and respective interpolated values of soil properties (US 287).

BH ID	Driving rate (ft/min)	Moisture content (%)	Plasticity index	SPT	Cohesion (psf)	Dry density (pcf)
BH_1	2.1	25.4	29	7	490	108
Interpolation of BH 1 & BH 3	2.2	23.32	29	7	494	107
	2.6	21.24	28	6	497	105
BH_3	3.6	20.2	27	5	500	104
Interpolation of BH 2 & BH 3	2.1	23.9	29.1	5	492	103.8
	2.2	24.7	29.6	5	485	103.5
	2.9	25.6	30	5	478	103.3
BH_2	3.1	26.4	30.5	5	470	103

3.2.1.2 New US 287

In 2017, a section of a highway slope along Texas highway US 287 near Midlothian was selected for the experiment (Sapkota 2019) due to the visible pavement distress signs and potential shallow slope failure. The shoulder of the selected highway section was closed to traffic due to failure potential at that time by TxDOT. The selected highway section was constructed over an embankment having a slope of 1V: 3.3 H. The existing report indicated the presence of Eagle Ford clay deposits.

The driving time of RPP for the slope test sections as well as the respective properties of the site soil were collected from the study of Sapkota (2019). The field installation of RPP was carried out during July, 2017. The size of RPP used was 4 in. x 4 in. whereas the driven length was kept constant as 10 feet. Three different spacing as 3 ft., 4 ft., and 5 ft. were used to install RPP along the slope. These data are summarized in Table 3.4.

Four boreholes namely BH_1, BH_2, BH_3, and BH_4 represented the properties of the site soil (Sapkota, 2019). Boreholes 1, 2, and 3 were on the crest of the slope and borehole 4 was at the toe of the slope. Soil properties of BH_1 and BH_3 were considered to be representative for the crest portion of the 'pin plus barrier' and 'pin only' sections respectively. Borehole 4 was considered to be representative for the toe portion of the 'pin only' section. For the middle portion of the slope sections soil properties were obtained using interpolation between crest boreholes and toe borehole. Depth wise average SPT value was used for analysis and SPT value up to 10 feet depth was considered. Soil properties collected from the study of Sapkota (2019) were moisture content, plasticity index, cohesion, and unit weight which are presented in Table 3.5.

Table 3.4: Driving rate and geometric properties of RPP installed in New US 287, Dallas (Sapkota, 2019).

Section	Location	RPP Cross section	Driven length (ft.)	Spacing (ft.)	Average driving rate (ft/min)
	Crest	4 in x 4 in		3	8
					7.63
Pin only	Middle	4 in x 4 in	10	4	6.58
					6.29
	Toe	4 in x 4 in		5	6.13
					5.91
	Crest	4 in x 4 in		3	7.75
					7.69
Pin+barrier	Middle	4 in x 4 in	10	4	6.76
					6.33
	Toe	4 in x 4 in		5	5.71
					5.78

Table 3.5: Properties of soil in New US 287, Dallas (Sapkota, 2019).

Bore hole ID	Depth (ft.)	Moisture content (%)	Plasticity index	SPT value	Cohesion (psf)	Dry density (pcf)
BH_1	3	32	22	2	272	99
	7	30	32	6	228	93
Avg.		31	27	4	250	96
BH_2	3	27	43	7	411	110
	7	32	37	7	439	104
Avg.		29.5	40	7	425	107
BH_3	3	42	36	2	197	91
	7	36	34	4	203	92.8
Avg.		39	35	3	200	92
BH_4	3	36	42	2	201	106
	7	30	37	6	269	96
Avg.		33	39.5	4	235	101

Table 3.6: Driving rate and respective interpolated soil properties (New US 287).

	BH ID	Driving rate (ft/min)	Moisture content (%)	Plasticity index	SPT	Cohesion (psf)	Dry density (pcf)
Crest	BH_3	8	39	35	3	200	92
	Interpolation between BH 3 & BH 2	7.63	34.25	37.5	5	312.5	100
	BH_1	7.75	31	27	4	250	96
	Interpolation between BH 1 & BH 2	7.69	30.25	33.5	6	337.5	102
Toe	BH_4	6.13	33	39.5	4	235	101
	BH_4	5.91	33	39.5	4	235	101
	BH_4	5.71	33	39.5	4	235	101
	BH_4	5.78	33	39.5	4	235	101
Middle	Interpolation (BH 3&4)	6.58	36	37.25	4	218	97
	Interpolation (BH 3&4)	6.29	36	37.25	4	218	97
	Interpolation (BH 1&4)	6.76	32	33.25	4	243	99
	Interpolation (BH 1&4)	6.33	32	33.25	4	243	99

3.2.2 Test Section in Denton

3.2.2.1 The City of Denton Landfill

Data for Denton area was collected from the study of Zaman (2019). The objectives of the study was to establish a new, efficient, cost effective and sustainable method for improving unsuitable foundation soil, i.e. to improve the bearing capacity of weak soil and to increase the sliding resistance or shear resistance of the MSE wall base, using Recycled Plastic Pins.

The driving time of RPP for the vertically loaded and laterally loaded test sections and the properties of the soil were collected from the study of Zaman (2019). The field installation of RPP for both vertical and lateral loaded test sections were carried out during July, 2017. The size of RPP installed included 4 in. x 4 in. and 6 in x 6 in. All the RPPs were installed with 3 feet spacing. These data are summarized in Table 3.7.

Three boreholes namely BH_1, BH_2 and BH_3 represented the condition of the selected sites for the study of Zaman (2019). Soil properties of these boreholes were considered to be representative for 6 in x 6 in (vertical), 4 in x 4 in (vertical), and 4 in x 4 in (lateral) sections respectively. SPT value up to 10 feet depth was considered and depth wise average SPT value was used in the analysis. Soil properties collected from this study were moisture content and unit weight as shown in Table 3.8.

Table 3.7: Driving rate and geometric properties of RPP installed in Denton (Zaman, 2019).

Section type	RPP Cross section	Driven length (ft)	Spacing (ft)	Average driving rate (ft/min)
Vertical loaded	4 in x 4 in	10	3	2.9
				2.3
				2.9
				3.1
				2.9
				3.1
Vertical loaded	6 in x 6 in	10	3	0.9
				1.2
				1.3
				1.8
				1.9
				1.9
Lateral loaded	4 in x 4 in	8	3	1.8
				1.7
				1.4
				1.5
				1.5
				1.6

Table 3.8: Soil properties of test sections in Denton (Zaman, 2019).

Bore hole ID	Depth (ft.)	Moisture content (%)	Dry density (pcf)	SPT value	Cohesion (psf)	Plasticity index
BH_1	5	12	98	5	350	11
BH_2	5	15	95	3	160	28
BH_3	5	12	98	12		

Table3.9: Driving rate and respective interpolated soil properties (Denton).

BH ID	RPP geometry	Driving rate (ft/min)	Moisture content (%)	Dry density (pcf)	SPT	Cohesion (psf)	Plasticity index
BH_2	4x4 Top 7 feet	7.89	15	95	3	160	28
Interpolation between BH 1 & BH 2		7.89	15	95	3	175	25
		7.38	15	95	3	190	22
		7.38	14	96	3	205	19
		7.38	14	96	4	220	17
		5.86	14	96	4	235	16
BH_1	6x6 Top 7 feet	2.29	12	98	5	350	11
Interpolation between BH 1 & BH 2		3.06	12	98	5	330	13
		3.31	12	98	5	310	15
		4.58	13	97	5	290	17
		4.84	13	97	4	270	19
		4.84	13	97	4	250	20

3.2.3 Test Section in Irving

3.2.3.1 The City of Irving Hunter Ferrell Landfill

A location inside the City of Irving Hunter Ferrell Landfill was chosen in December 2018 to conduct experimental study on using RPP to increase bearing capacity and shearing resistance of MSE wall and for improving bearing capacity of embankment soft foundation soil. For this purpose three types of test sections were built namely vertical loaded sections (to study bearing capacity of MSE wall), lateral loaded section (to study shearing capacity of MSE wall), and soft soil section (to study bearing capacity of soft foundation soil). The proposed site is located inside the landfill considerably far from the active zone of the landfill. The total available area in this zone is approximately 12,000 sq. ft. The location is accessible through the hauling roads inside the landfill, and also a road over the levee from the eastern side of the landfill. Driving time of RP data was collected directly during the installation process during February, 2019 to May, 2019.

Soft Soil Section

Two boreholes (BH_1 and BH_2) represented the properties of the site soil in case of Irving Soft sections. SPT values from BH_1 and BH_2 were interpolated to get values for RPPs in the middle of the section. Only 6 in. x 6 in. RPP were used and the spacing was 2 feet. As the length of RPP is 10 feet, SPT value up to 10 feet was considered. Depth wise average SPT value was used for analysis.

Undisturbed and disturbed soil samples were collected during the drilling. Soil properties evaluated and used were moisture content, plasticity index, and unit weight.

Table 3.10: Driving rate and geometric properties of RPP installed in Hunter Ferrell Landfill, Irving.

Section type	RPP Cross section	Driven length (ft)	Spacing (ft)	Average driving rate (ft/min)
Lateral loaded	4 in x 4 in	8	2	4.28
	4 in x 4 in	10	2	5.83
	4 in x 4 in	8	3	3.19
	4 in x 4 in	10	3	3.67
	6 in x 6 in	8	3	2.82
	6 in x 6 in	10	3	2.99
Vertical loaded	4 in x 4 in	10	2	1.28
	6 in x 6 in	10	3	1.86
	10 in x 10 in	10	3	2.9
Soft soil	6 in x 6 in	10	2	1.01
		10		1.12
		10		0.97
		10		0.96
		7		1.07
		7		0.92
		7		0.89

Vertical Loaded Section

Three boreholes namely BH_2, BH_3, and BH_4 represented the properties of the site soil of the Irving Vertical loaded sections. SPT values from BH_2 was representative of 10 in x 10 in section while BH_3 was considered representative for both 4 in x 4 in and 6 in x 6 in sections as BH_3 lied in between of these sections. In this case three sizes of RPP namely 4 in x 4 in, 6 in x 6 in, and 10 in x 10 in with spacing of 2 ft., 3 ft., and 3 ft. respectively were used. As the length of RPP is 10 feet, SPT value up to 10 feet was considered.

Figure 3.2: Borehole locations for Irving Vertical loaded sections.

Both undisturbed and disturbed soil samples were collected. Soil properties found and used in the present study were moisture content, cohesion, dry density, plasticity index, and unit weight.

Table 3.11: Soil properties of test sections in Irving.

Section type	Borehole ID	Depth (ft)	Moisture content (%)	Plasticity index	Cohesion (psf)	Dry density (pcf)	SPT value	Soil type
	BH_2	5	25	25	320	105	4	CL
Soft soil		10	20	32	341	108	7	Shale
	BH_3	5	24	23	341	102	5	CL
		10	21	35	341	110	41	Shale
	BH_1	5	11	12	395	109	11	CL
		10	12.7	12	328	106	15	CL
	Avg.		12	12	723	107.5	13	
Lateral	BH_2	5	20	31	432	112	8	CL
loaded		10	21	17	380	110	13	ML
	Avg.		21	24	406	111	11	
	BH_3	5	20	33	521	106	8	CL
		10	14	28	456	106	7	ML
	Avg.		17	30.3	489	106	8	
	BH_2	5	25	36	456	112	8	CH
Vertical		10	15	24	394	101	11	ML
loaded	Avg.		20	28	425	107	10	
	BH_3	5	20	38	433	109	5	CL
		10	14	26	367	101	5	CL
	Avg.		17	32	400	105	5	

Table 3.12: Driving rate and respective interpolated soil properties (Irving).

	BH ID	RPP	Driving rate (ft/min)	Moisture content (%)	Plasticity index	SPT	Dry density (pcf)	Cohesion (psf)
Soft soil Section	BH_2	6 in x 6 in Top 7 ft	2.69	25	25	4	105	320
	Interpolation between BH 2 & BH 3		3.04	24.8	25	4	105	324
			2.80	24.7	24	4	104	327
			3.68	24.6	24	4	104	330
			2.69	24.4	24	5	103	333
			3.04	24.2	23	5	103	336
			3.33	24.1	23	5	102	339
	BH_3		3.89	24	23	5	102	341
	BH_2	6 in x 6 in Bottom 3 ft	2.14	20	32	7	108	320
	Interpolation between BH 2 & BH 3		2.00	20.1	31	12	108	323
			1.67	20.3	32	17	109	327
			1.20	20.5	33	22	109	330
			1.43	20.6	33	27	109	333
			0.91	20.7	34	32	110	336
			1.11	20.9	34	37	110	339
	BH_3		0.40	21	35	41	110	341

	Interpolation	4 in x	5.33	12.64	13.09	13	107.8	365.5
	between BH	4 in	5.85	13.42	14.18	13	108.1	369.6
	1 & BH 2		5.45	14.21	15.27	12	108.5	373.6
		4 in x	4.62	19.98	23.27	11	110.8	403.3
		4 in	12.31	19.45	22.55	11	110.6	400.6
Lateral			9.23	18.93	21.82	11	110.4	397.9
loaded			12.00	18.40	21.09	11	110.2	395.2
section			9.60	17.88	20.36	11	109.9	392.5
	Interpolation		10.00	20.29	24.39	10	110.6	411.0
	between BH		15.00	20.08	24.79	10	110.3	416.0
	2 & BH 3		5.78	19.86	25.18	10	109.9	421.0
			11.16	19.65	25.58	10	109.5	426.0
			14.55	19.44	25.97	10	109.2	431.0
		6 in x	2.16	17.32	29.91	8	105.5	481.0
		6 in	5.39	17.64	29.32	8	106.1	473.5
			3.71	18.06	28.53	8	106.8	463.5
Vertical	BH_3	4x4	4.29	17	32	10	105	400.0
loaded	BH_3	6x6	1.86	17	32	10	105	400.0
section	BH_2	10x10	2.9	20	28	10	107	425.0

Figure 3.3: Collection of driving time data for 10 in x 10 in RPP section.

Figure 3.4: Collection of driving time data for 4 in x 4 in RPP sections.

Figure 3.5: Collection of driving time data for 6 in x 6 in RPP section.

Lateral Loaded Section

Three boreholes namely BH_1, BH_2, and BH_3 represented the properties of the site soil for the Irving Lateral loaded sections. SPT values from boreholes 1, 2, and 3 were considered representative for 4 in x 4 in @ 3 feet c/c, 4 in x 4 in @ 2 feet c/c, and 6 in x 6 in @ 3 feet c/c respectively. As the length of RPP is 10 feet, SPT value up to 10 feet was considered. Depth wise average SPT value was considered for analysis.

Disturbed and undisturbed both soil samples were collected. Soil properties tested and used in the present study were moisture content, cohesion, dry density, plasticity index, cohesion, and unit weight.

3.2.4 Test Sections in Arlington

The test sections were in the region of I-20 Park Spring Boulevard, I-30 Fielder (North), and I-30 Fielder (South) and named after accordingly. The field installation was carried out during August, 2019 to September, 2019. Two boreholes (BH_1 and BH_3) for each of the three test section areas represented the properties of the site soil. As the length of RPP is 10 feet, soil properties up to 10 feet was considered. Depth wise average value was considered and interpolation between bore holes was done for the analysis.

Disturbed and undisturbed both soil samples were collected. Soil properties tested and used in the present study were moisture content, plastic limit, plasticity index, cohesion, and unit weight.

Table 3.13: Soil properties of test sections in Arlington.

	BH ID	Depth (ft)	Moisture content (%)	Plasticity index	SPT	Dry density (pcf)	Cohesion (psf)
I30	BH_1	6	20	38.3	25	118	2000
Fielder		11	7	8.9			
North	BH_2	5	14.7	21.1	32	124.2	1550
		12	25.1	5.2			
I30	BH_1	7	20	40.9	6	100	450
Fielder		11	41.5	31.7	21		
South	BH_2	5	12.35		7	120	500
		10	15.8		17		
Park	BH_1	7	14.6	9	29	102	2150
Spring	BH_2	7	14.275	20	41	125	2050

Table 3.14: Driving rate and respective interpolated soil properties (Arlington).

	BH ID	RPP	Driving rate (ft/min)	Moisture content (%)	Plasticity index	SPT	Dry density (pcf)	Cohesion (psf)
I30 Fielder North	BH_2	4x4	1.17	14.7	21.1	32	124.2	2000
	interpolation (BH 1&2)		1.43	17.35	29.7	29	121.1	1775
	BH_1		1.52	20	38.3	25	118	1550
	*Avg driving depth was 5 feet							
Park Spring Boulevard	BH_1	4x4	2.5	14.6	9	29	102	2150
	interpolation between BH 1 & BH 2		1.26	14.44	14.5	35	113	2100
	BH_2		0.94	14.28	20	41	125	2050
	*Avg driving depth was 7 feet.							
I30 Fielder South	BH_1	4x4	2.26	20	40.9	6	450	100
	interpolation between BH 1 & BH 2		1.78	16.87		9	475	110
	BH_2		1.45	13.73		11	500	120
	*Avg driving depth was 7 feet.							

Chapter 4

RESULT ANALYSES & DISCUSSIONS

4.1 INTRODUCTION

This chapter represents the analyses of the driving rate data of RPP obtained from different test sections in four different areas (Dallas, Irving, Arlington, and Denton) of North Texas where recycled plastic pins (RPP) were used to stabilize the slope or improve the ground condition. The main focus of the analyses was to determine the influences of soil properties such as moisture content, cohesion, dry density, plasticity index, and standard penetration test (SPT) value on driving rate of RPP and evaluate the driving rate for different soil properties. For this case, driving rate data from the studies of Khan (2013), Tamrakar (2015), Zaman (2019), and Sapkota (2019) were considered. Additionally, data from new test sections in Irving and Arlington was considered.

4.2 EFFECTS OF SOIL PROPERTIES ON DRIVING RATE OF RPP

Properties of site soil have impactful influence on the rate of driving of any pile system and consequently they affect the installation rate of RPP. The soil properties that studied in this work are natural moisture content, plasticity index, dry density, and cohesion. Standard penetration test value (SPT value) of the site soil has been taken as a prompt indication of soil stiffness or hardness which resists RPP driving and consequently its influence has been studied.

4.2.1 Influence of Natural Moisture Content of the Soil

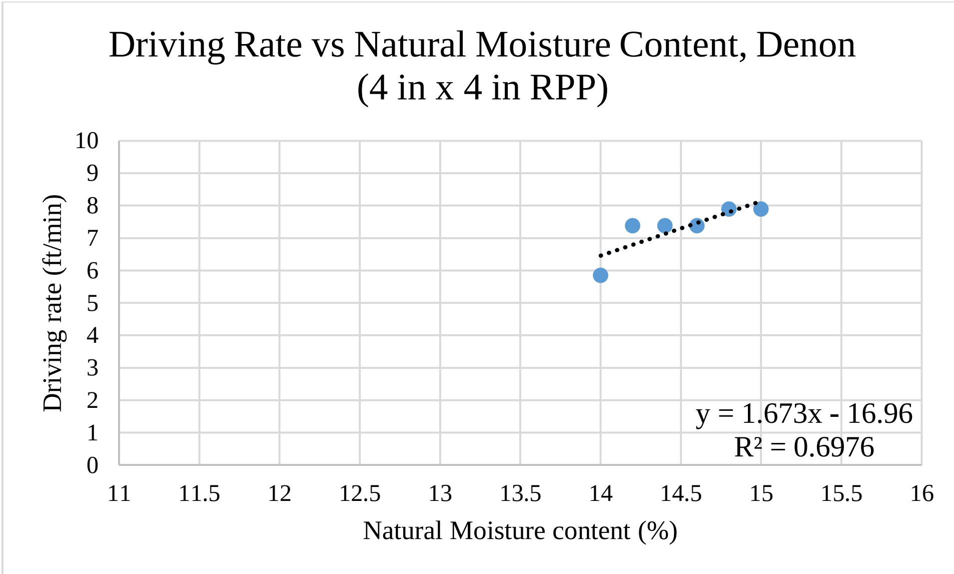
Natural moisture content of the site soil varies largely with season. Also it varies instantly with rainfall. However, the percentage of water (moisture content) existing in soil at any moment signifies its stiffness and thereby affecting the driving of RPP.

The results obtained from four different areas are first discussed individually and then amalgamated to get the encyclopedic idea.

4.2.1.1 Denton, Texas

The driving time of RPP for the vertically loaded and laterally loaded test sections in the City of Denton Landfill and the respective natural moisture contents of the soil were collected from the study of Zaman (2019). The field installation of RPP for both vertical and lateral loaded test sections were carried out during July, 2017. The size of RPP installed included 4 in. x 4 in. and 6 in x 6 in.

Figure 4.1 shows the variation of driving rate of RPP installation with respect to varying moisture content for Denton area. The driving rate of RPP increased with an increase in moisture content for both the cases of 4 in x 4 in and 6 in x 6 in RPP (Figure 4.1). Increasing moisture content renders softness to the site soil and enables the ease of RPP driving and therefore driving rate increased while driving in high moisture zone.



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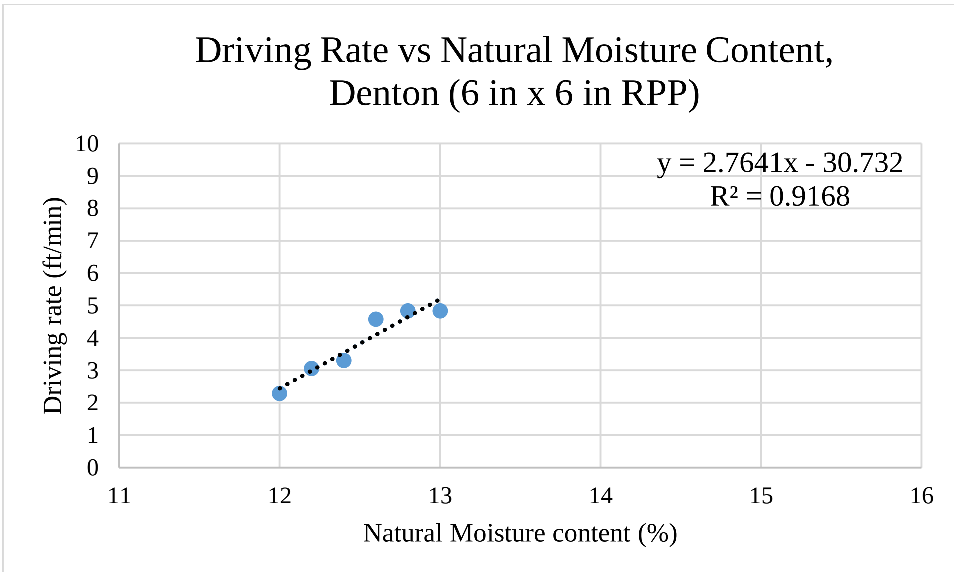


(b)

Figure 4.1: Variation of Driving Rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with Natural Moisture Content of Soil (Denton area).

4.2.1.2 Dallas, Texas

The driving time of RPP for the US 287 test sections and the respective natural moisture contents of site soil were collected from the study of Khan (2013). The field installation of RPP was carried out during February, 2012. The size of RPP installed was 4 in. x 4 in. Figure 4.2 shows the variation of driving rate of RPP installation with respect to varying moisture content for the test section in US 287 (Khan, 2013). The driving rate of RPP increased with an increase in moisture content as shown in Figure 4.2. Increasing moisture content renders softness to the site soil and enables the ease of RPP driving and therefore driving rate increased while driving in high moisture zone.

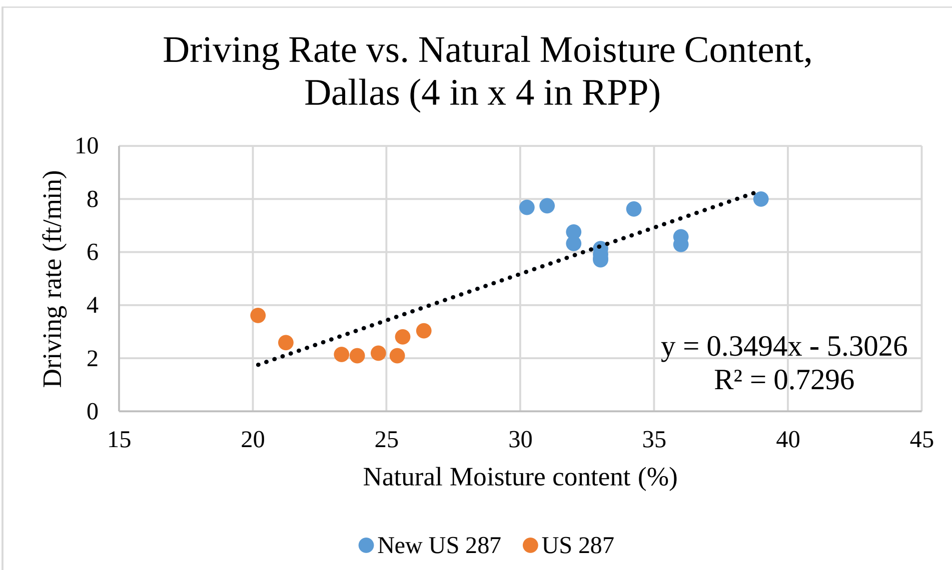


Figure 4.2: Variation of Driving Rate with Natural Moisture Content (Dallas area).

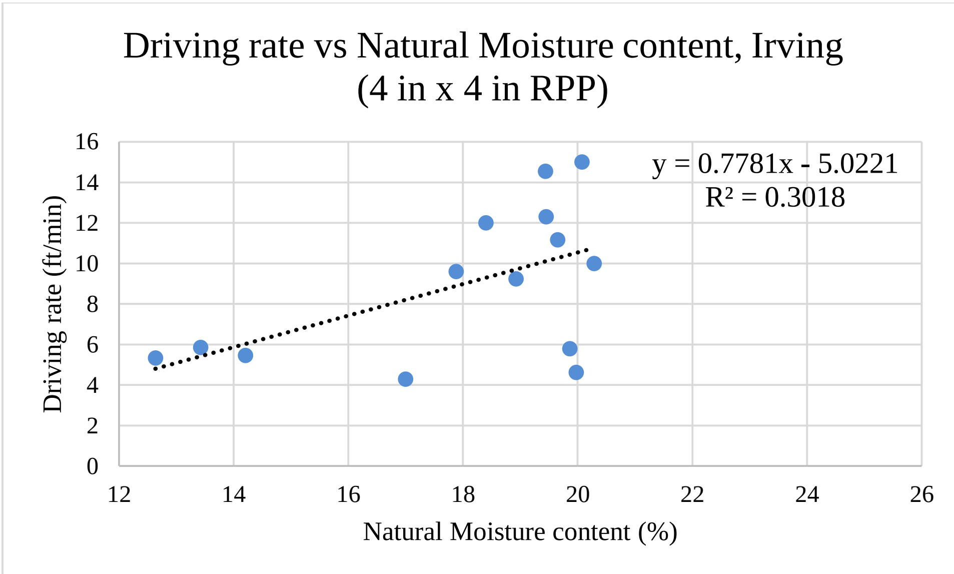
The driving time of RPP for the New US 287 test sections as well as the respective natural moisture contents of the site soil were collected from the study of Sapkota (2019). The field installation of RPP was carried out during July, 2017. The size of RPP used was 4 in. x 4 in. whereas the driven length was kept constant as 10 feet. Also for this case driving rate increased with increasing moisture content of the site soil.

Data obtained from all the test sections in different locations in Dallas showed a general pattern of increasing driving rate with an increase in soil moisture content.

4.2.1.3 Irving, Texas

The selected study area was the City of Irving Hunter Ferrell landfill. Three types of test sections were built for studying namely vertically loaded section, laterally loaded section, and very soft soil section. The field installation was carried out during February, 2019 to May, 2019. The size of RPP used in this study were 4 in x 4 in and 6 in x 6 in.

Figure 4.3 shows the variation of driving rate of RPP installation with respect to varying moisture content for the test sections in Irving. The driving rate of 4 in x 4 in and 6 in x 6 in RPPs increased with increasing moisture content. Comparative softness of the soil is indicative of increase in moisture content and enables the easier driving of RPP.



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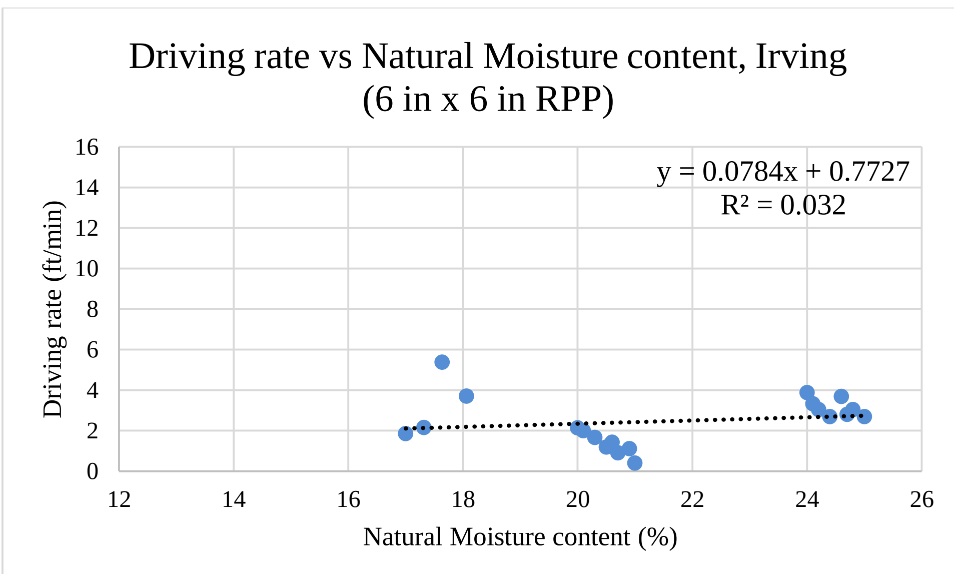


(b)

Figure 4.3: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with natural moisture content (Irving area).

4.2.1.4 Arlington, Texas

The test sections were in the region of I-20 Park Spring Boulevard, I-30 Fielder (North), and I-30 Fielder (South). The field installation was carried out during August, 2019 to September, 2019. The cross section of RPP used in this study were 4 in x 4 in.

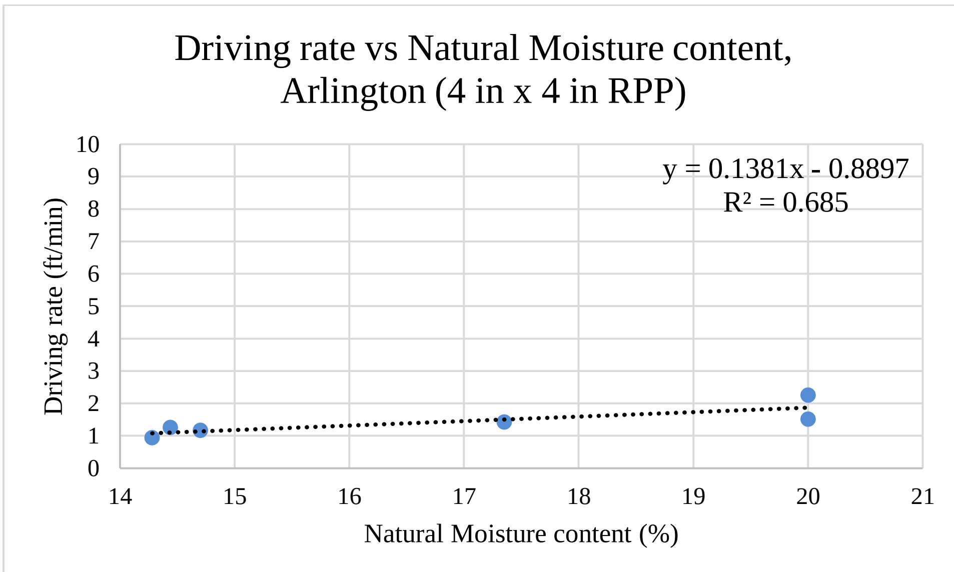


Figure 4.4: Variation of Driving Rate with Natural Moisture Content (Arlington area).

Figure 4.4 shows the variation of driving rate of RPP installation with respect to varying natural moisture content for Arlington area. The driving rate of RPP increased with an increase in moisture content for 4 in x 4 RPP (Figure 4.4). Increasing moisture content renders softness to the site soil and enables the ease of RPP driving and therefore driving rate increased while driving in high moisture zone.

4.2.1.5 Combined Analysis for the Effect of Moisture Content

In all the cases (Denton, Arlington, and Dallas) driving rate of 4 in x 4 in RPP increased with an increase in the natural moisture content of the site soil. Therefore, required installation time decreased for the same phenomenon. An increase in moisture content offers softness (Topp and Ferre, 2002) to the soil and enables the ease of RPP driving and therefore driving rate increased while driving in high moisture zone and therefore it can be perceived that driving time would be less which resulted in an increase in driving rate.

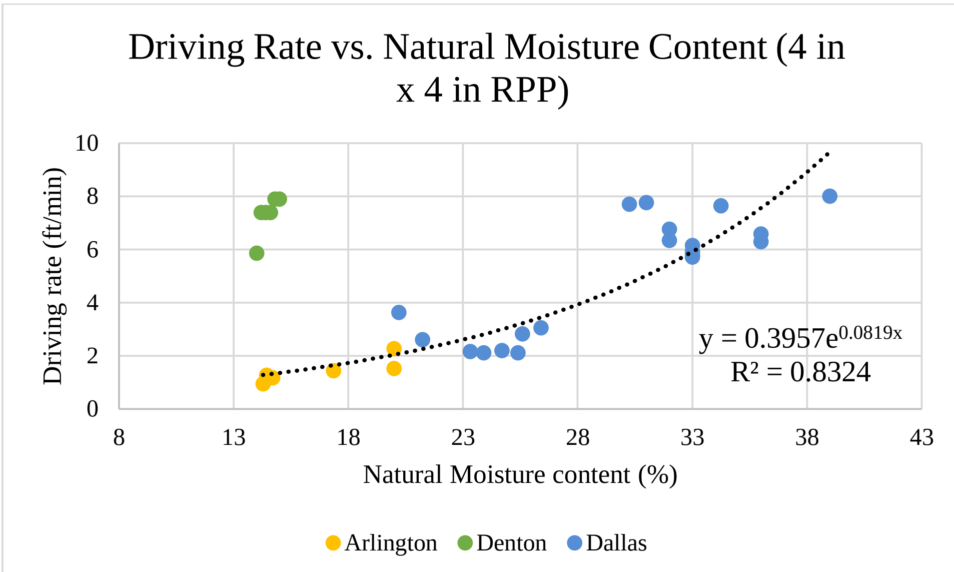


Figure 4.5: Combined scatter plot of moisture content vs. driving rate with trend line.

It can be seen from Figure 4.5 that there is an exponential increase in driving rate as a general trend with increasing moisture content of the site soil. The value of coefficient of determination R^2 was found to be 0.8324 indicating that the model equation signifies the 83.24% of the variation in data.

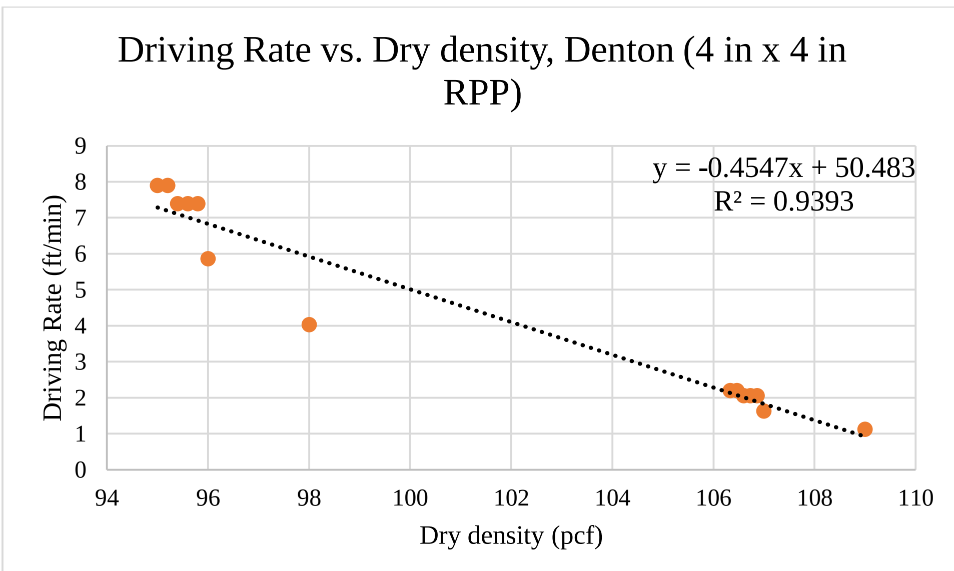
The driving rate data of 6 in x 6 in RPP were not considered for combined analysis because those RPPs were installed with different machine with varying capacity in different test sections. Also data from Irving area was discarded in combined analysis as these data were more scattered.

4.2.2 Influence of Dry Density of the Soil

The ratio of total dry mass to the total volume of a soil mass is known to be dry density. When degree of compaction will be more, dry density of that soil mass will be more. Driving rate of pile highly depends on the relative density of the soil which in turn is an indication of compaction.

4.2.2.1 Denton, Texas

The driving time of RPP installation and the respective dry density of the soil were collected from the study of Zaman (2019). The size (cross sectional area) of RPP installed included 4 in. x 4 in. and 6 in x 6 in.



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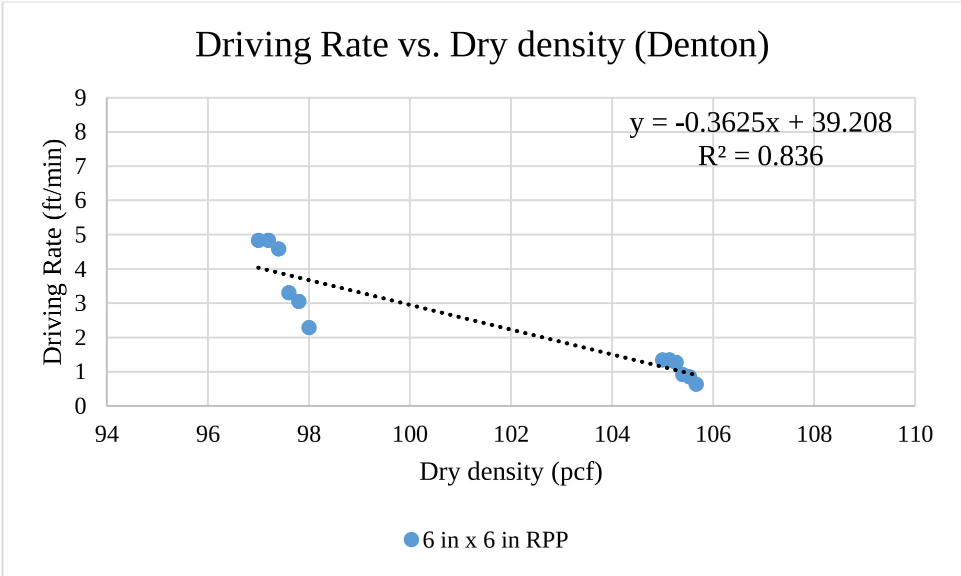


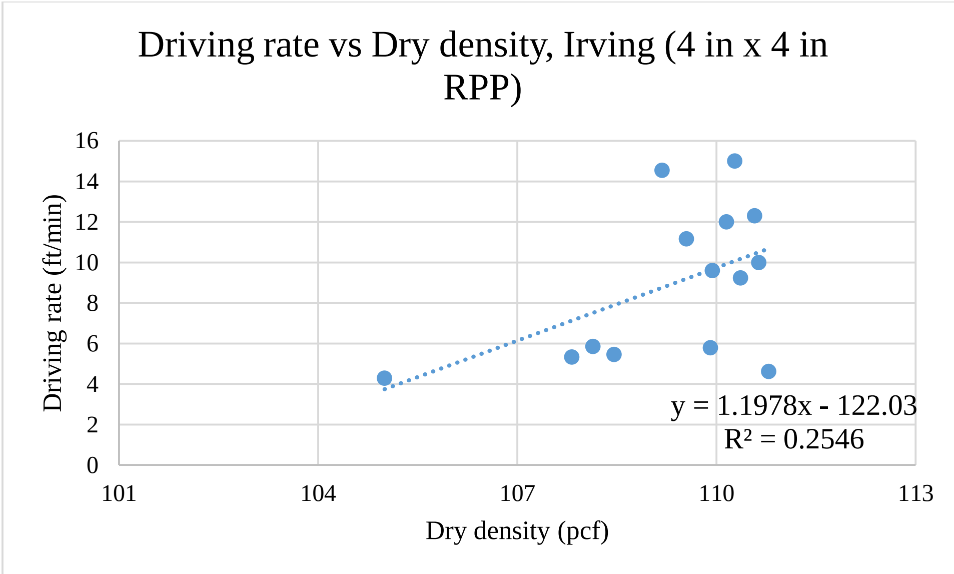
(b)

Figure 4.6: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with dry density (Denton area).

Figure 4.6 illustrates the change in driving rate of RPP with changing dry density of soil samples collected from the test sections in Denton. Driving rate for 4 in x 4 in RPP increased with an increase in dry soil density while for 6 in x 6 in RPP it decreased.

4.2.2.2 Irving, Texas

Figure 4.7 illustrates the change in driving rate of RPP with various dry density of soil samples in the case of test sections in Irving. Driving rate decreased with an increase in dry soil density for 6 in x 6 in RPP as shown in Figure 4.7. However, an increasing pattern of driving rate was found with an increase in soil dry density for 4 in x 4 in RPP. In both the cases, the change of driving rate with dry density was slight.



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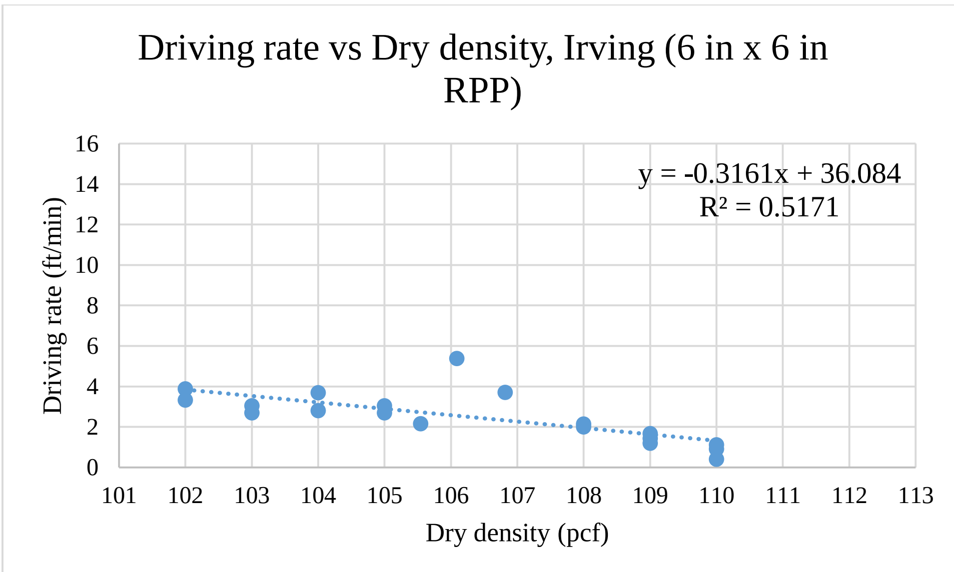


(b)

Figure 4.7: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with dry density of soil (Irving area).

4.2.2.3 Dallas, Texas

The driving time of RPP for the US 287 test sections and the dry density of site soil were collected from the study of Khan (2013). The field installation of RPP was carried out during February, 2012. The size of RPP installed was 4 in. x 4 in. Figure 4.8 shows the variation of driving rate of RPP installation with respect to varying soil density for the test section in US 287 (Khan, 2013). The driving time of RPP for the New US 287 test sections as well as the respective dry density of the site soil were collected from the study of Sapkota (2019). The field installation of RPP was carried out during July, 2017. The size of RPP used was 4 in. x 4 in. whereas the driven length was kept constant as 10 feet. The driving rate of RPP decreased with an increase in dry density as shown in Figure 4.8.

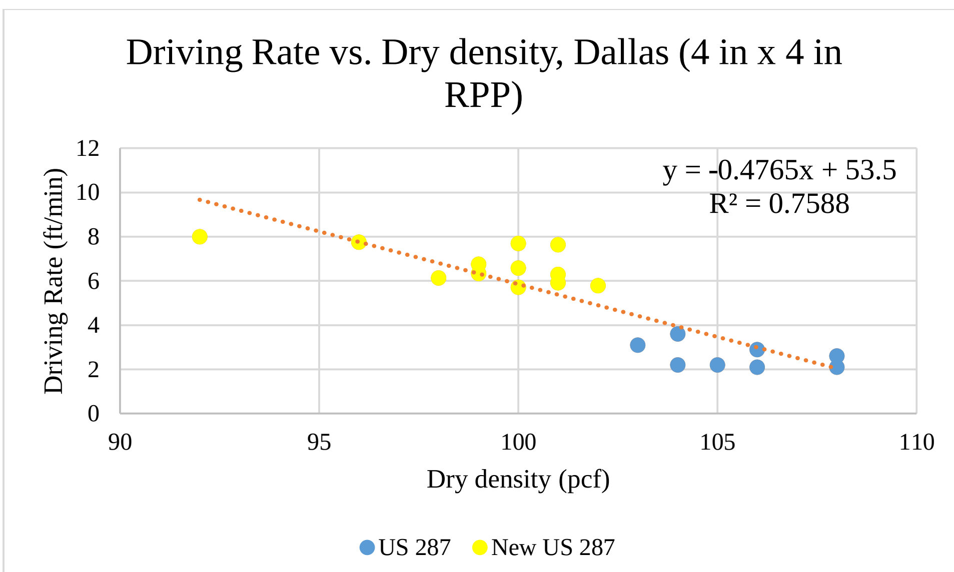


Figure 4.8: Variation of driving rate with dry density of soil (Dallas area).

4.2.2.4 Arlington, Texas

The test sections were in the region of I-20 Park Spring Boulevard, I-30 Fielder (North), and I-30 Fielder (South). The field installation was carried out during August, 2019 to September, 2019. The cross section of RPP used in this study were 4 in x 4 in.

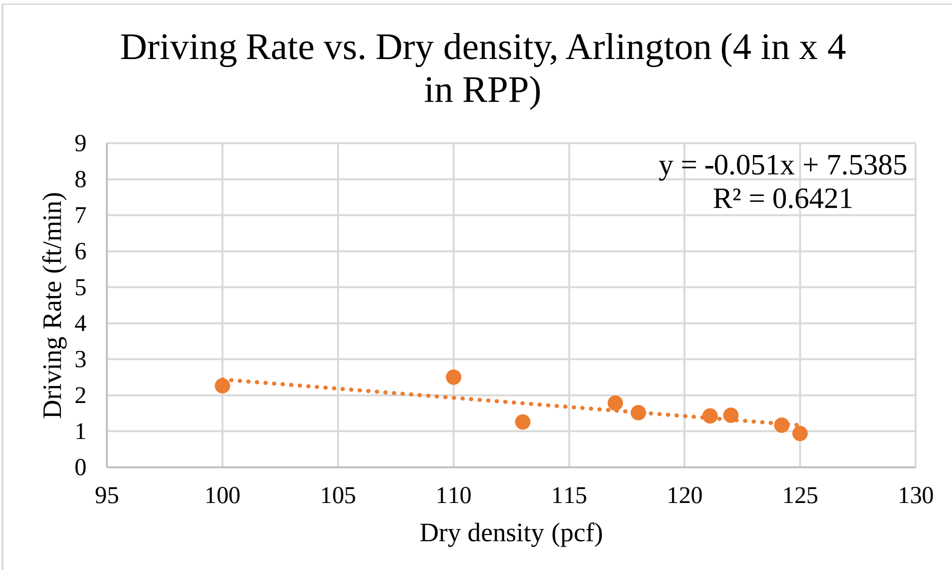


Figure 4.9: Variation of driving rate with dry density of soil (Arlington area).

Figure 4.9 shows the variation of driving rate of RPP installation with respect to dry density of the site soil for Arlington area. The driving rate of RPP decreased with an increase in dry density for 4 in x 4 RPP.

4.2.2.5 Combined Analysis for the Effect of Soil Dry Density

In general driving rate of RPP appeared to decrease with an increase in the dry density of the soil as shown in Figure 4.10. Conversely, with an increase in dry unit weight required installation time will be higher.

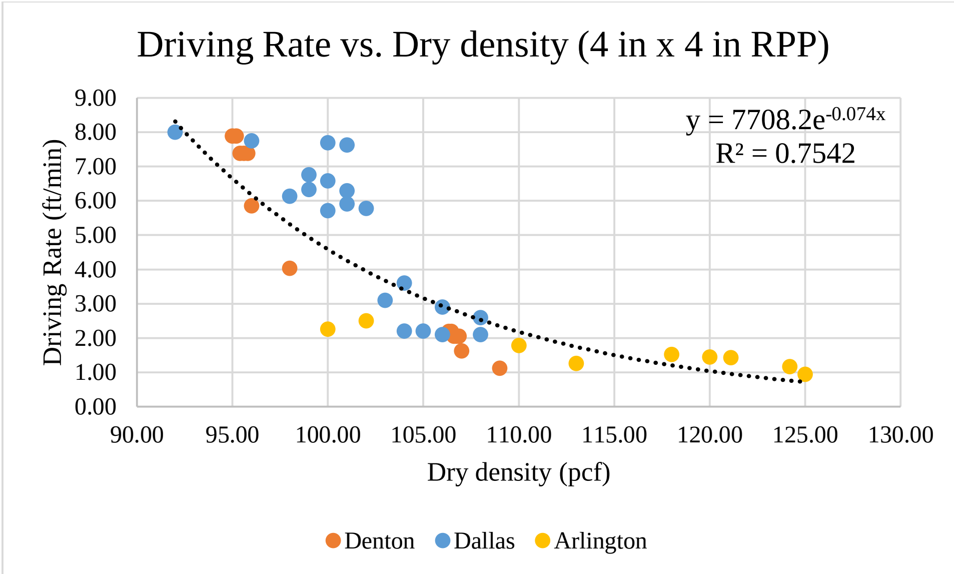


Figure 4.10: Combined scatter plot of dry density of soil vs. driving rate of RPP.

Figure 4.10 showed an exponential decay of driving rate with dry density of soil. The value of coefficient of determination R^2 was found to be 0.7542 indicating that the model equation signifies the 75.42% of the variation in data.

The driving rate data of 6 in x 6 in RPP were not considered for combined analysis because those RPPs were installed with different machine with varying capacity in different test sections. Also data from Irving area was discarded in combined analysis as these data were more scattered.

4.2.3 Influence of Soil Cohesion

The shear strength or the force that binds together soil particles in the structure of a soil mass is called cohesion. Cohesion exists without any compressive stress.

4.2.3.1 Dallas, Texas

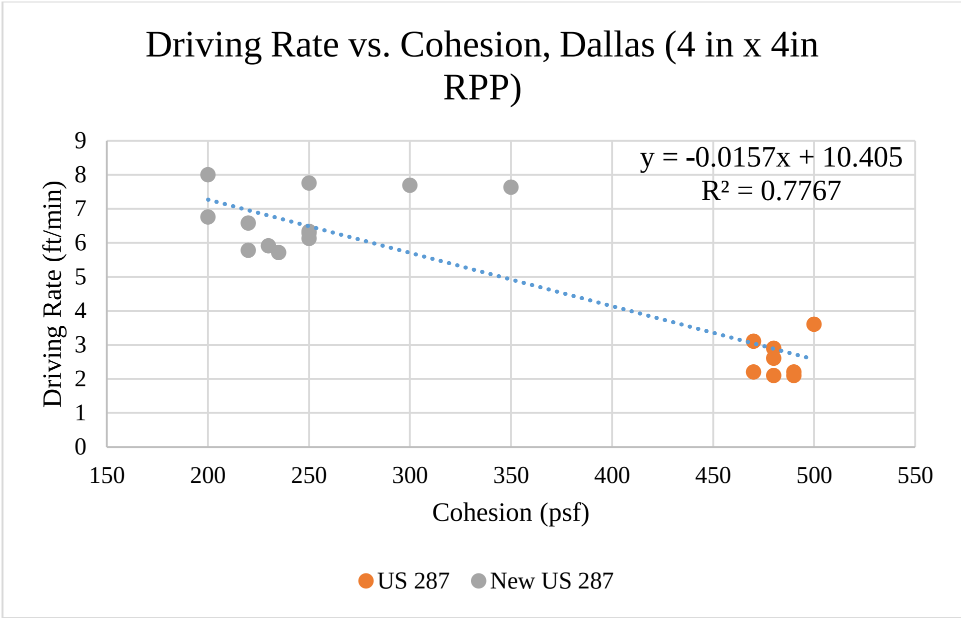
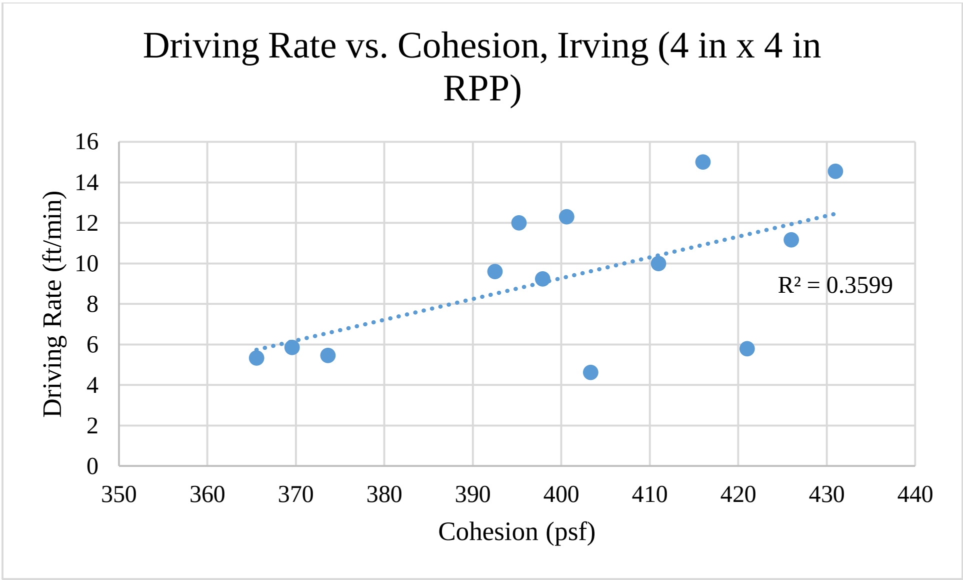


Figure 4.11: Variation of Driving Rate with Cohesion of Soil (Dallas Area).

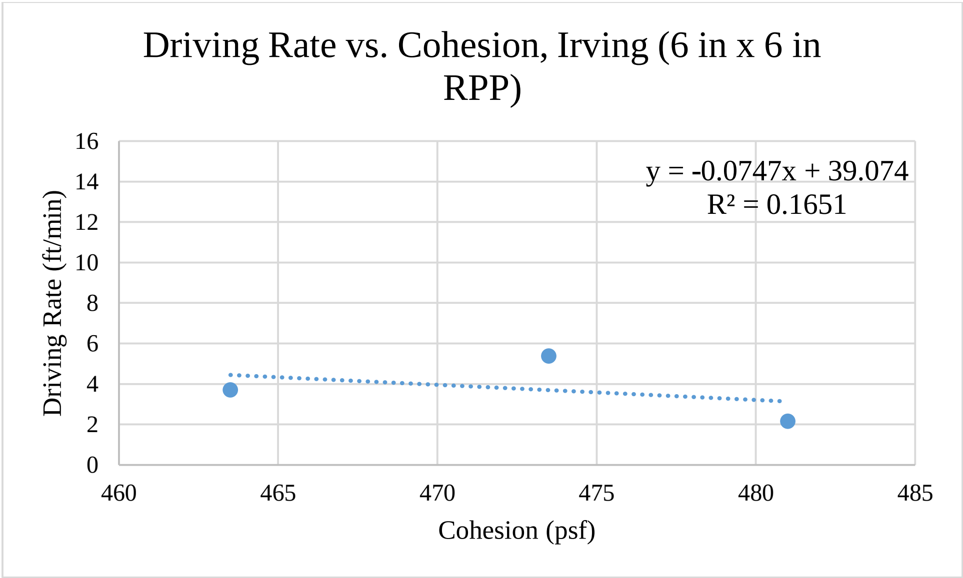
Figure 4.11 represents the change in driving rate of RPP installation with respect to the cohesion of the soil samples from Dallas. It can be noticed that driving rate decreased with increasing cohesion value (Figure 4.11). Cohesion value refers to soil strength and it is the dominating shear strength parameter in case of clayey soil. The expansive clayey soil of the site which is typical in North Texas renders hardship to pile driving when cohesion value is higher.

4.2.3.2 Irving, Texas

The shear strength or the force that binds together the soil particles in the structure of a soil is called cohesion. In this case the cohesion of the sample soil was determined by either direct shear test or triaxial (consolidated undrained) test.



(a)



(b)

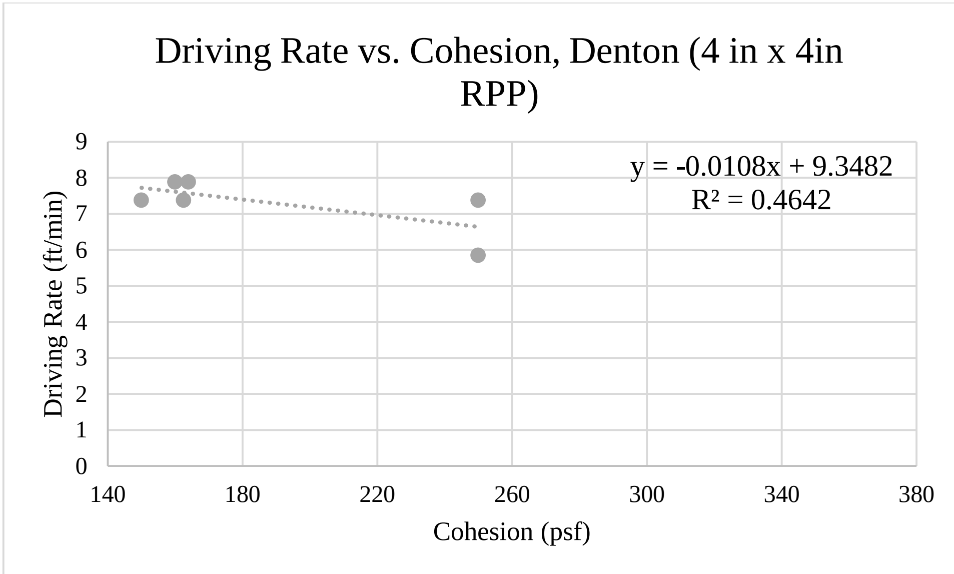
Figure 4.12: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with cohesion of Soil (Irving area).

Figure 4.12 presents the variation of driving rate of RPP installation with respect to the cohesion of the soil samples for the case of Irving area. It can be seen that driving rate decreased with increasing cohesion value for 6 in x 6 in RPP while it followed opposite pattern for 4 in x 4 in RPP.

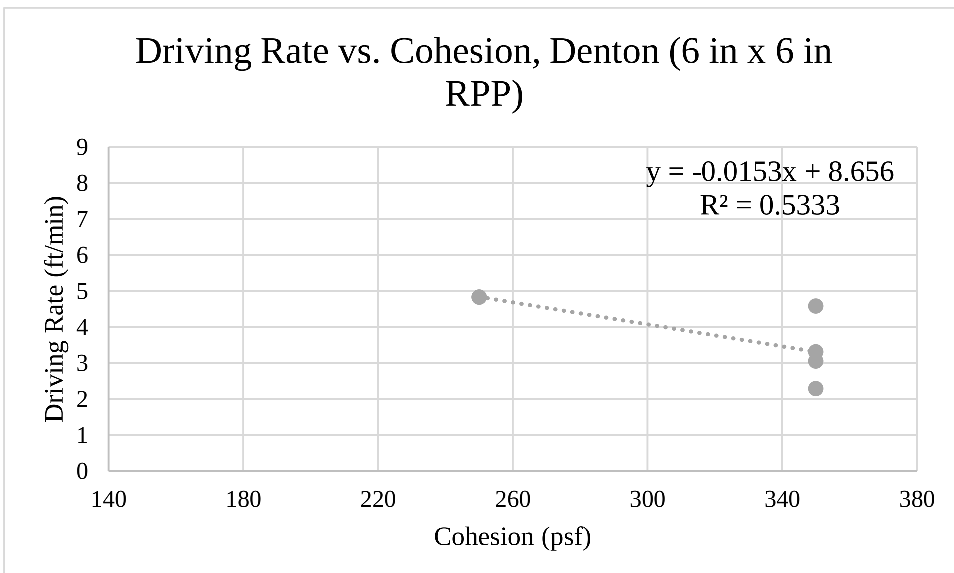
4.2.3.3 Denton, Texas

The driving time of RPP for the vertically loaded and laterally loaded test sections in the City of Denton Landfill and the respective cohesion of the soil were collected from the study of Zaman (2019). The field installation of RPP for both vertical and lateral loaded test sections were carried out during July, 2017. The size of RPP installed included 4 in. x 4 in. and 6 in x 6 in.

Figure 4.13 shows the variation of driving rate of RPP installation with respect to soil cohesion for Denton area. The driving rate of RPP decreased with an increase in cohesion value for both the cases of 4 in x 4 in and 6 in x 6 in RPP (Figure 4.13).



(a)



(b)

Figure 4.13: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with cohesion of Soil (Denton area).

4.2.3.4 Arlington, Texas

The test sections were in the region of I-20 Park Spring Boulevard, I-30 Fielder (North), and I-30 Fielder (South). The field installation was carried out during August, 2019 to September, 2019. The cross section of RPP used in this study were 4 in x 4 in.

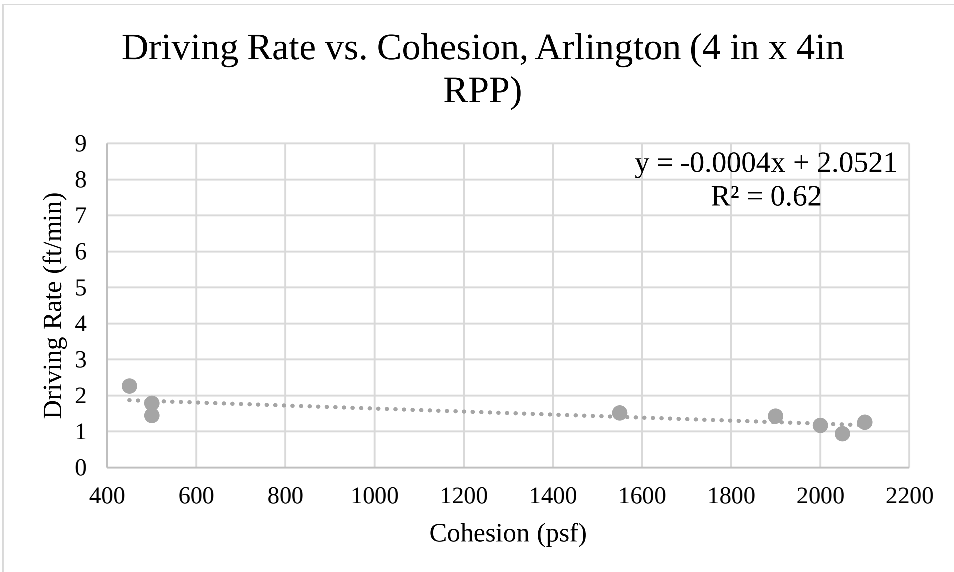


Figure 4.14: Variation of driving rate of RPP with cohesion of Soil (Arlington area).

Figure 4.14 presents the variation of driving rate of RPP installation with respect to the cohesion of the soil samples for the case of Arlington area. It can be perceived that driving rate decreased with increasing cohesion value of the site soil

4.2.3.5 Combined Analysis for the Effect of Cohesion

Driving rate was found to decrease with an increase in the cohesion property of site soil for 4 in x 4 in RPP as shown in the Figure 4.15. This trend was found to be similar for the areas of Dallas, Denton, and Arlington. It implied that required installation time increased when the soil was more cohesive.

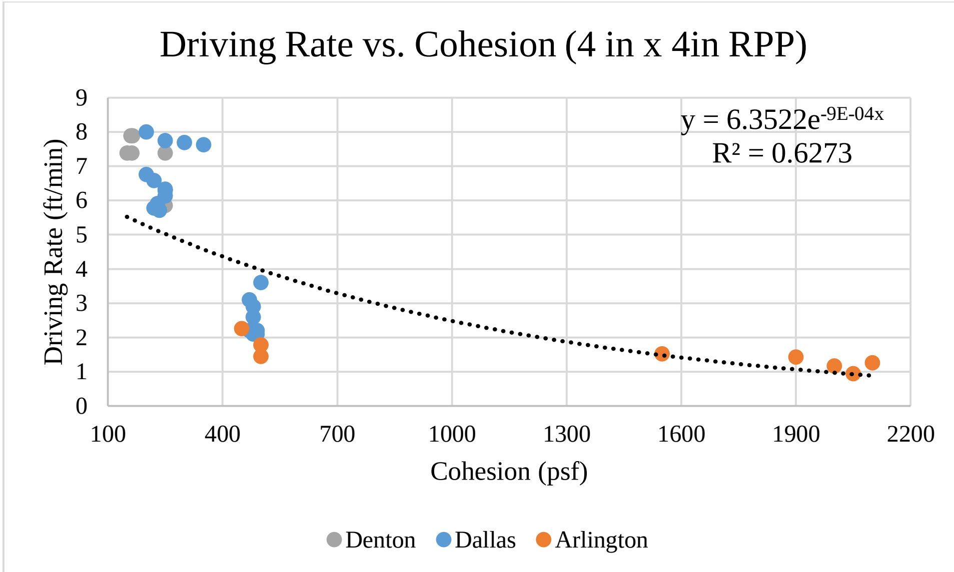


Figure 4.15: Combined scatter plot of cohesion of soil vs. driving rate of RPP.

Figure 4.15 showed an exponential decay of driving rate with cohesion of soil. The value of coefficient of determination R^2 was found to be 0.6273 indicating that the model equation signifies the 62.73% of the variation in data.

The driving rate data of 6 in x 6 in RPP were not considered for combined analysis because those RPPs were installed with different machine with varying capacity in different test sections. Also data from Irving area was discarded in combined analysis as these data were more scattered.

4.2.4 Influences of Plasticity Index (PI)

The presence of water in the voids of soil body can affect the engineering behavior of fine grained soil. For analysis and design purposes we need to compare the existing natural water content against standard of engineering behavior which is known as Atterberg limits. In this study the driving rate of RPP was analyzed with respect to the plasticity index (PI) of site soil.

4.2.4.1 Dallas, Texas

The driving time of RPP for the US 287 test sections and the respective plasticity index of site soil were collected from the study of Khan (2013). The field installation of RPP was carried out during February, 2012. The size of RPP installed was 4 in. x 4 in. The driving time of RPP for the New US 287 test sections as well as the respective plasticity index of the site soil were collected from the study of Sapkota (2019). The field installation of RPP was carried out during July, 2017. The size of RPP used was 4 in. x 4 in. whereas the driven length was kept constant as 10 feet.

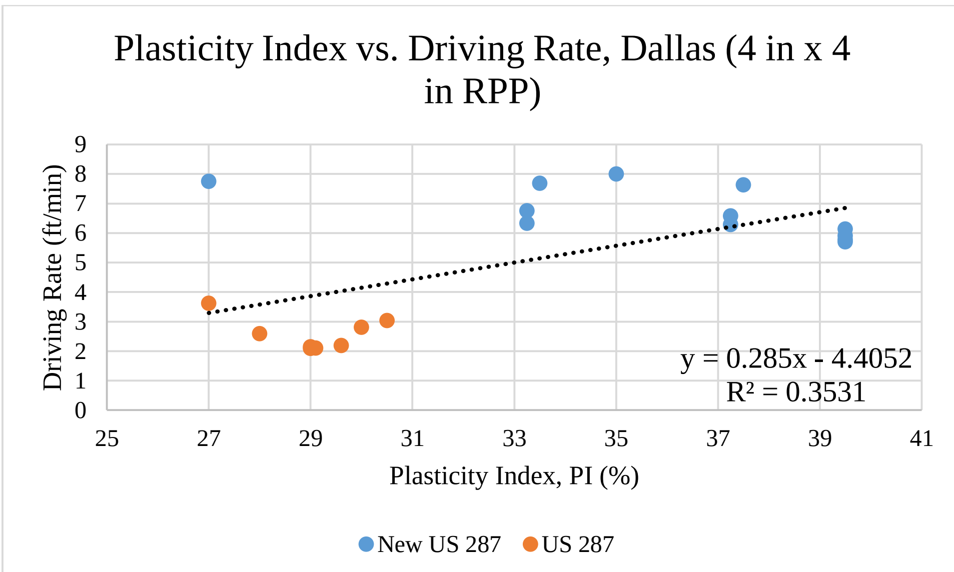
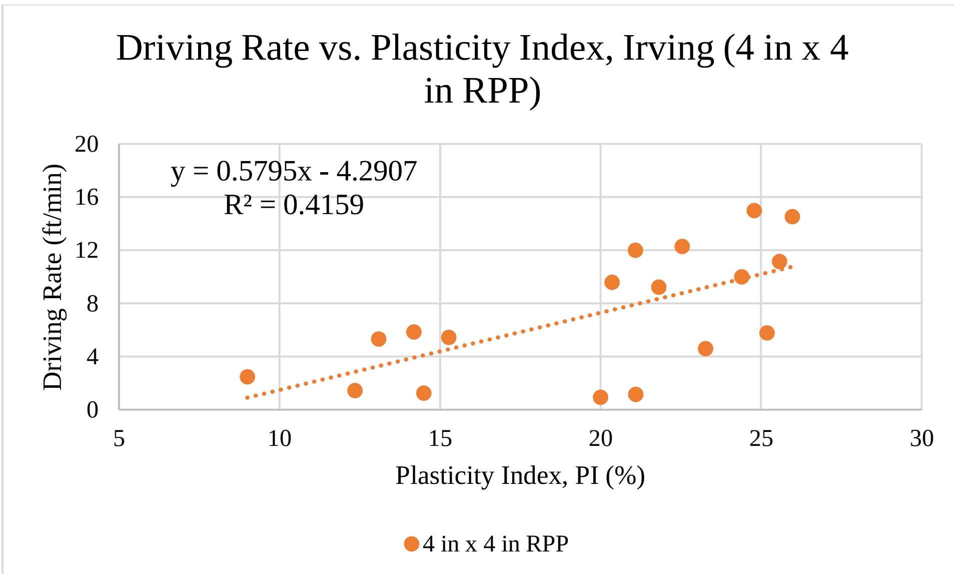


Figure 4.16: Variation of driving rate with plasticity index of soil (Dallas area).

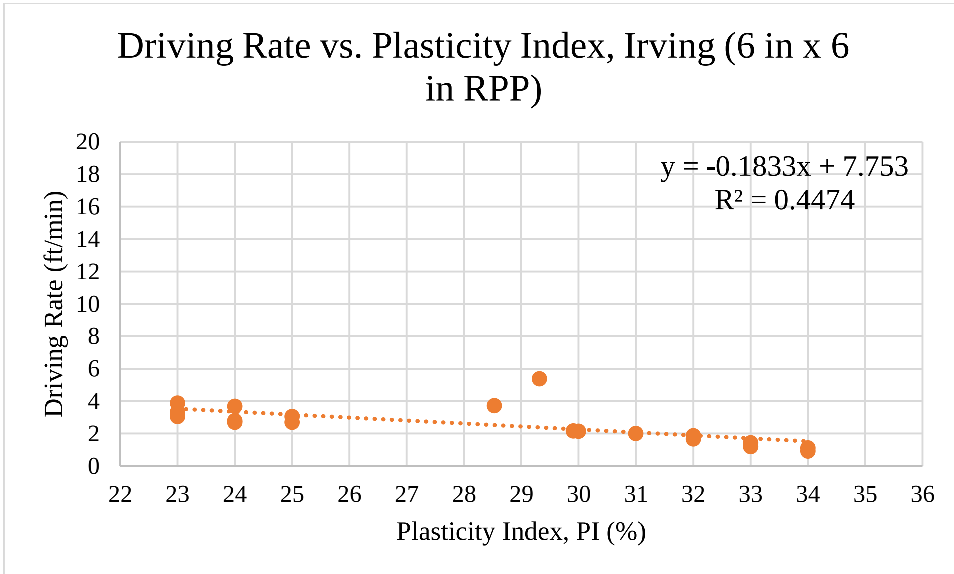
Figure 4.16 represents the change in driving rate of installing RPP with variation in Plasticity Index (PI) for the study area of Dallas. It can be seen that driving rate increased with an increase in plasticity index.

4.2.4.2 Irving, Texas

Figure 4.17 represented the variation in driving rate of RPP with varying plasticity index (PI) for the test sections in Irving. It can be seen that driving rate increased with an increase in plasticity index for the case of 4 in x 4 in RPP. On the other hand, for the case of 6 in x 6 in RPP driving rate decreased with an increase in plasticity index.



(a)

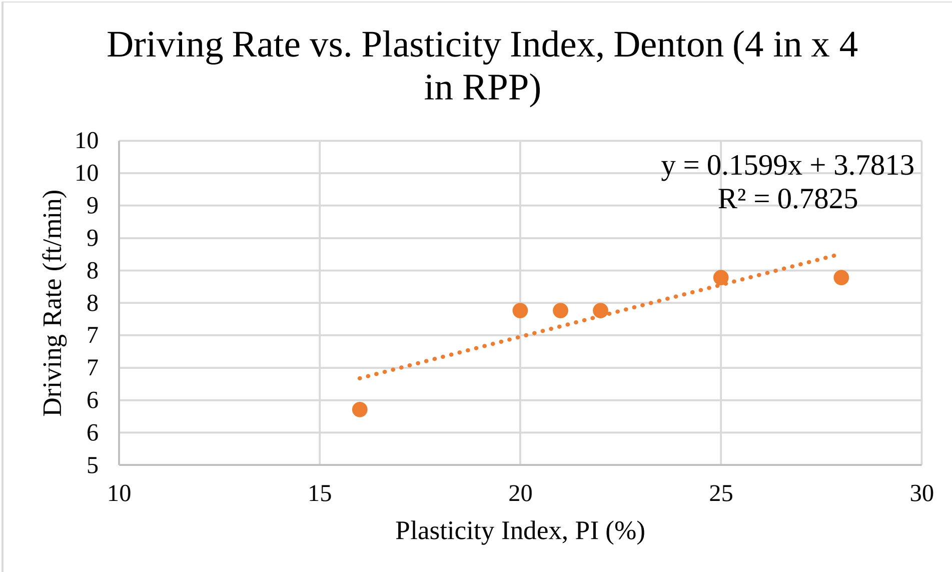


(b)

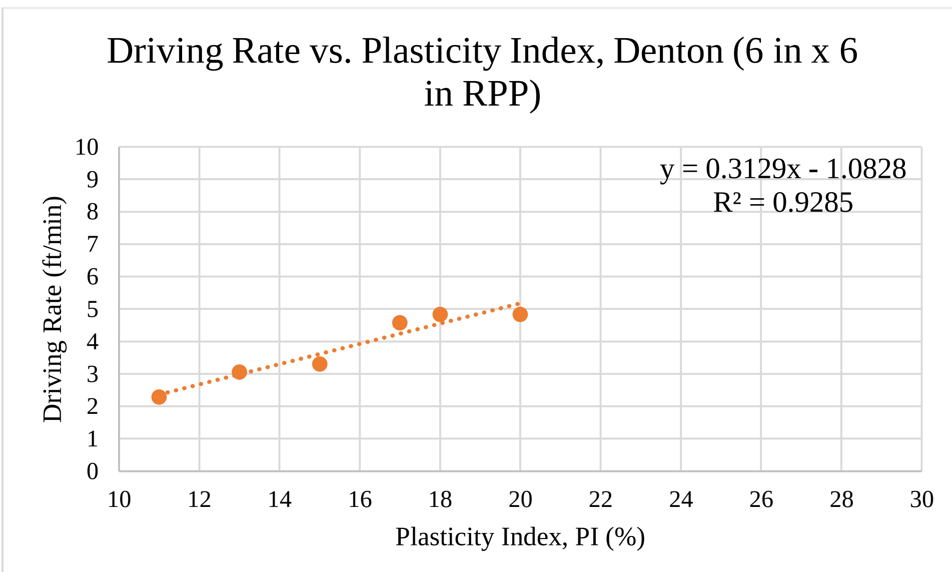
Figure 4.17: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with plasticity index of soil (Irving area).

4.2.4.3 Denton, Texas

The driving time of RPP for the vertically loaded and laterally loaded test sections in the City of Denton Landfill and the respective plasticity index of the soil were collected from the study of Zaman (2019). The field installation of RPP for both vertical and lateral loaded test sections were carried out during July, 2017. The size of RPP installed included 4 in. x 4 in. and 6 in x 6 in.



(a)



(b)

Figure 4.18: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with plasticity index of soil (Denton area).

Figure 4.18 shows the variation of driving rate of RPP installation with respect to varying plasticity index for Denton area. The driving rate of RPP increased with an increase in plasticity index for both the cases of 4 in x 4 in and 6 in x 6 in RPP (Figure 4.1).

4.2.4.4 Arlington, Texas

The test sections were in the region of I-20 Park Spring Boulevard, I-30 Fielder (North), and I-30 Fielder (South). The field installation was carried out during August, 2019 to September, 2019. The cross section of RPP used in this study were 4 in x 4 in.

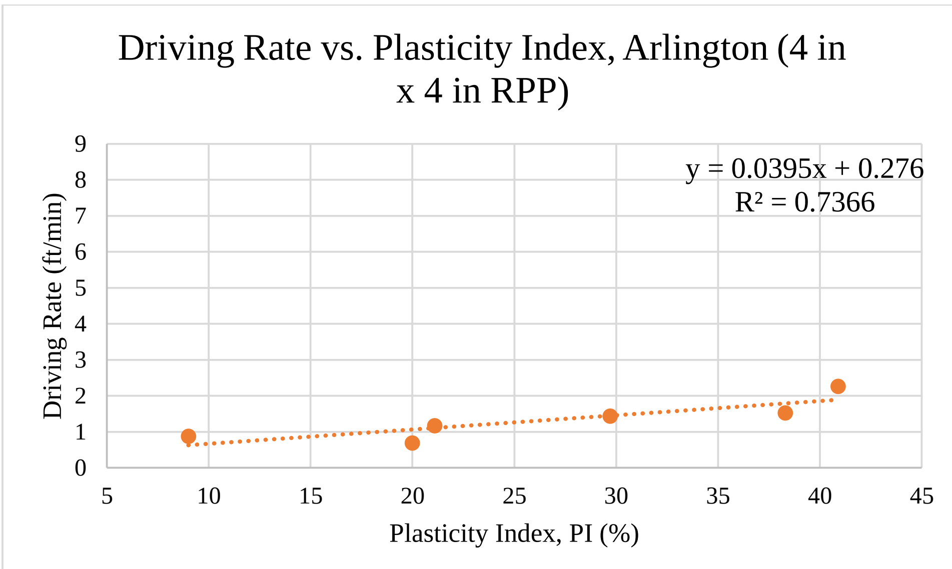


Figure 4.19: Variation of driving rate of RPP with plasticity index of soil (Arlington area).

Figure 4.19 shows the variation of driving rate of RPP installation with respect to varying plasticity index for Arlington area. The driving rate of RPP increased with an increase in PI for 4 in x 4 RPP.

4.2.4.5 Combined Analysis for the Effect of Plasticity Index (PI)

When analyzing driving rate in connection with plasticity index (PI), driving rate was found to increase with an increase in PI for all the test sections in Denton, Arlington, and Dallas for 4 in x 4 in RPP as shown in Figure 4.20.

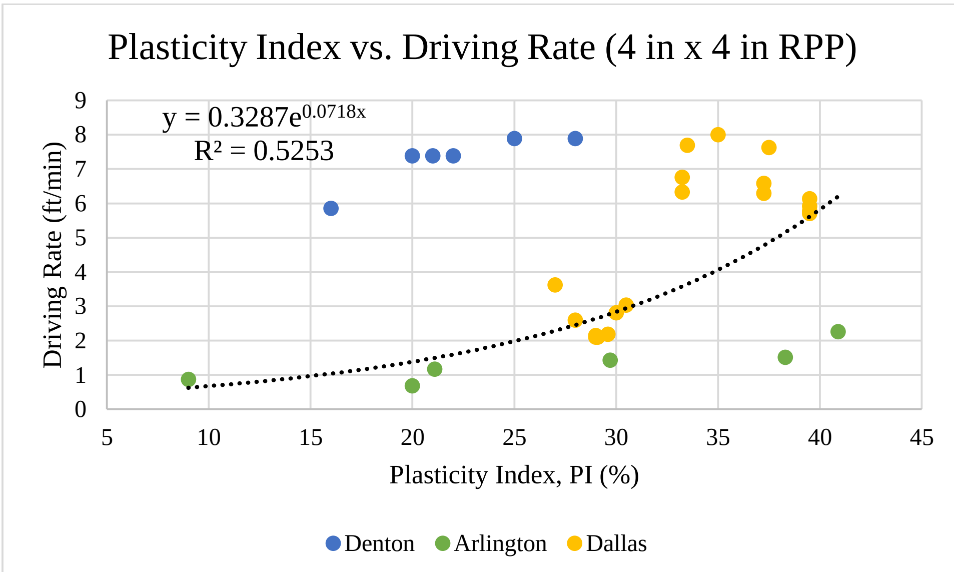


Figure 4.20: Combined scatter plot of plasticity index vs. driving rate of RPP.

Figure 4.20 showed an exponential increase of driving rate with plasticity index of soil. The value of coefficient of determination R^2 was found to be 0.5253 indicating that the model equation signifies the 52.53% of the variation in data.

The driving rate data of 6 in x 6 in RPP were not considered for combined analysis because those RPPs were installed with different machine with varying capacity in different test sections. Also data from Irving area was discarded in combined analysis as these data were more scattered.

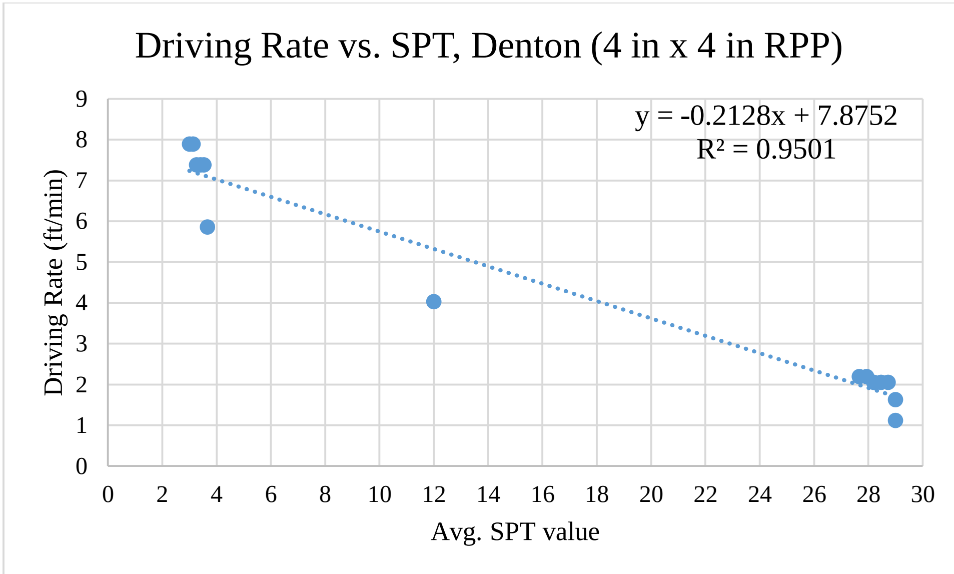
4.2.5 Influence of Varying SPT of the Sites

The standard penetration test (SPT) known to be an in-situ dynamic penetration test is designed to provide information on the properties of soil. This test is considered as the most widely used subsurface exploration technique performed worldwide. The samples obtained from the test provides with soil identification and a measure of penetration resistance which in turn can be used for design purposes. Widely accepted correlations between blow count or SPT N-value to the engineering soil properties are available for geotechnical engineering purposes.

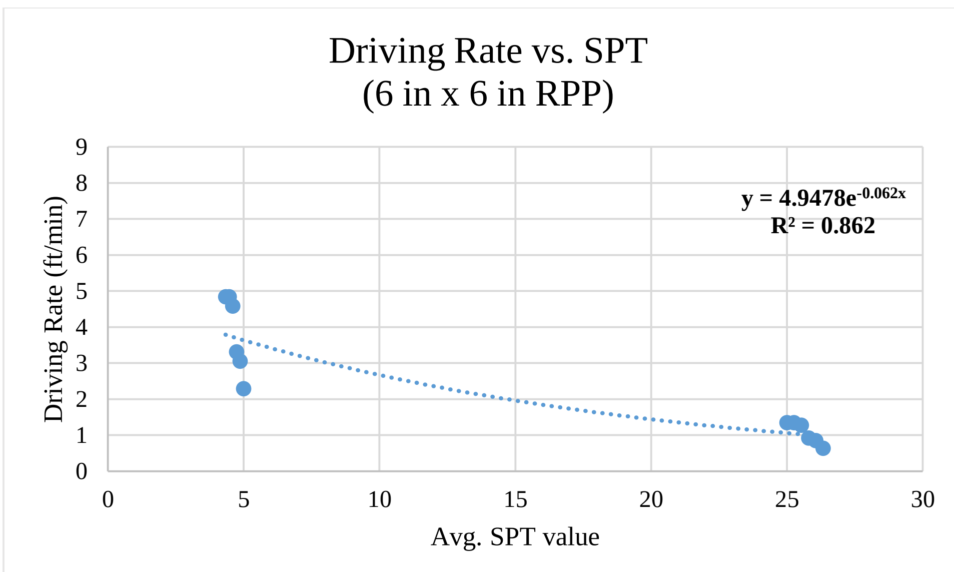
The value obtained from SPT represents the resistance of soil against penetration. The higher the SPT value of a soil mass, the more resistant the soil will be when penetrating by piles or RPP. Therefore, SPT value of a site soil is an impactful factor to assess the driving rate of RPP. It can be inferred easily that driving rate of RPP would be decreased when SPT value is higher.

4.2.5.1 Denton, Texas

Three boreholes namely BH_1, BH_2 and BH_3 represented the condition of the selected sites for the study of Zaman (2019). Soil properties of BH_1, BH_2, and BH_3 were considered to be representative for 6 in x 6 in (vertical), 4 in x 4 in (vertical), and 4 in x 4 in (lateral) sections.



(a)



(b)

Figure 4.21: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with average SPT value (Denton area).

In Figure 4.21, driving rate of RPP installation were plotted against respective SPT value of the Denton site (Zaman, 2019). It is conspicuous that in the cases of both sizes of RPP driving rate decreased with increasing SPT values. Increasing SPT value refers to the soil being increasingly harder. Driving RPP through harder soil would require more time provided the applied energy remains the same and therefore it is evident that driving rate decreased with increasing SPT value of site soil.

4.2.5.2 Dallas, Texas

There were two locations for test sections in Dallas namely New US 287 and US 287.

Four boreholes namely BH_1, BH_2, BH_3, and BH_4 represented the properties of the New US 287 site soil (Sapkota, 2019). Boreholes 1, 2, and 3 were on the crest of the slope and borehole 4 was at the toe of the slope. Soil properties of BH_1 and BH_3 were considered to be representative for the crest portion of the 'pin plus barrier' and 'pin only' sections respectively. Borehole 4 was considered to be representative for the toe portion of the 'pin only' section. For the middle portion of the slope sections soil properties were obtained using interpolation between crest boreholes and toe borehole. Three boreholes namely BH_1, BH_2 and BH_3 represented the properties of the site soil of US 287 (Khan, 2013). SPT values from BH_1 and BH_3 were interpolated to get values for RPPs in the middle of the section. As the maximum driven length of RPP is 10 feet, SPT value up to 10 feet was considered.

In Figure 4.22, driving rate of RPP installation was plotted against respective SPT value of the soil. It is perceptible that driving rate decreased with increasing SPT values for all the cases. Increasing SPT value refers to the soil being increasingly harder. Driving RPP through harder soil would require more time provided the applied energy remains the same and therefore it is evident that driving rate decreased.

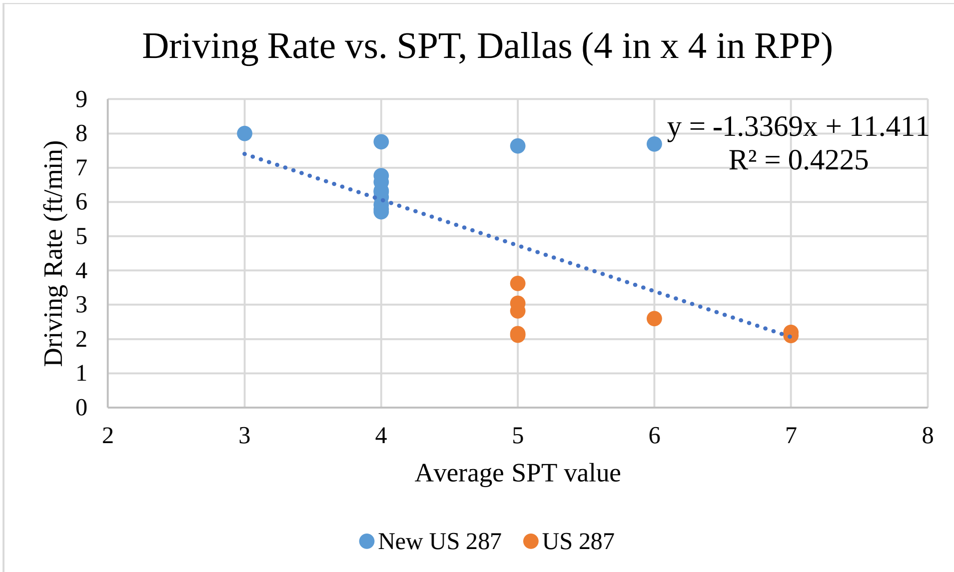
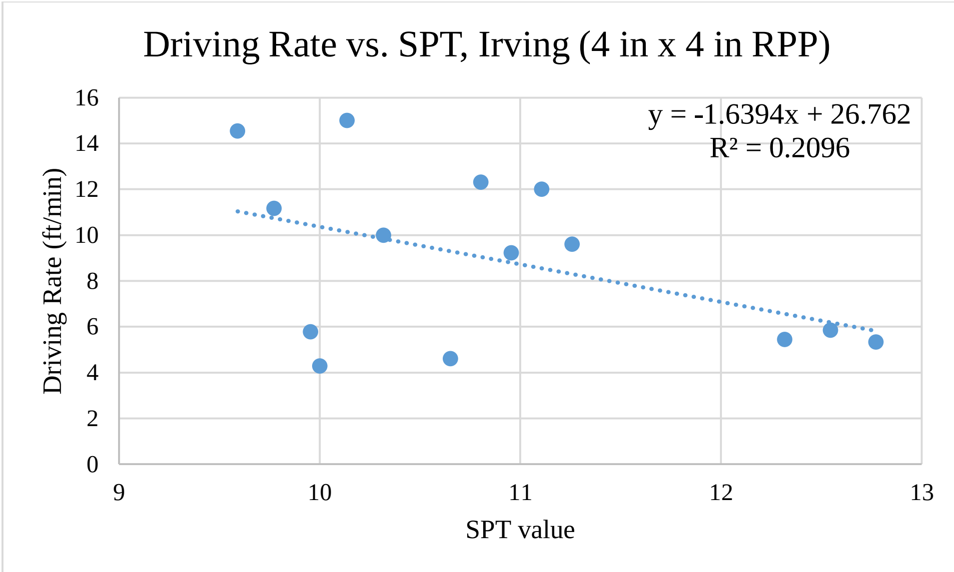


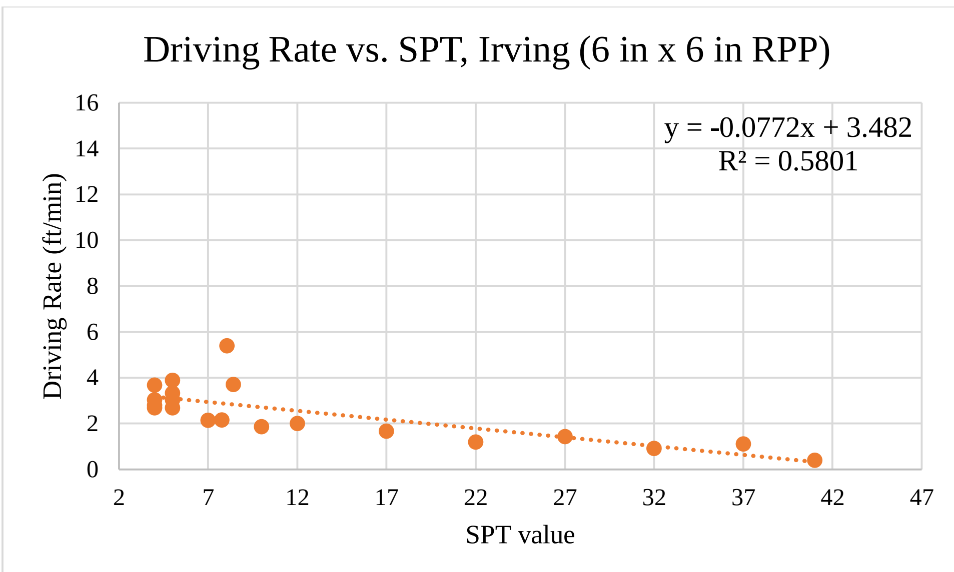
Figure 4.22: Variation of Driving Rate with average SPT value (Dallas area).

4.2.5.3 Irving, Texas

The City of Irving Hunter Ferrell Landfill was selected to be the location for field experiment. Two different sizes (cross section) were considered for the present study namely 4 in x 4 in and 6 in x 6 in RPPs.



(a)



(b)

Figure 4.23: Variation of driving rate of a) 4 in x 4 in and b) 6 in x 6 in RPPs with varying SPT of soil (Irving area).

In Figure 4.23, driving rate of RPP installation were plotted against respective SPT value of the soil. It can be seen that driving rate decreased with increasing SPT values for both sizes of RPP. Increasing SPT value refers to the soil being increasingly harder. Driving RPP through harder soil would require more time provided the applied energy remains the same and therefore it is evident that driving rate decreased.

4.2.5.4 Arlington, Texas

The test sections were in the region of I-20 Park Spring Boulevard, I-30 Fielder (North), and I-30 Fielder (South). The field installation was carried out during August, 2019 to September, 2019. The cross section of RPP used in this study were 4 in x 4 in. Two boreholes (BH_1 and BH_2) for each of the three test section locations represented the properties of the site soils.

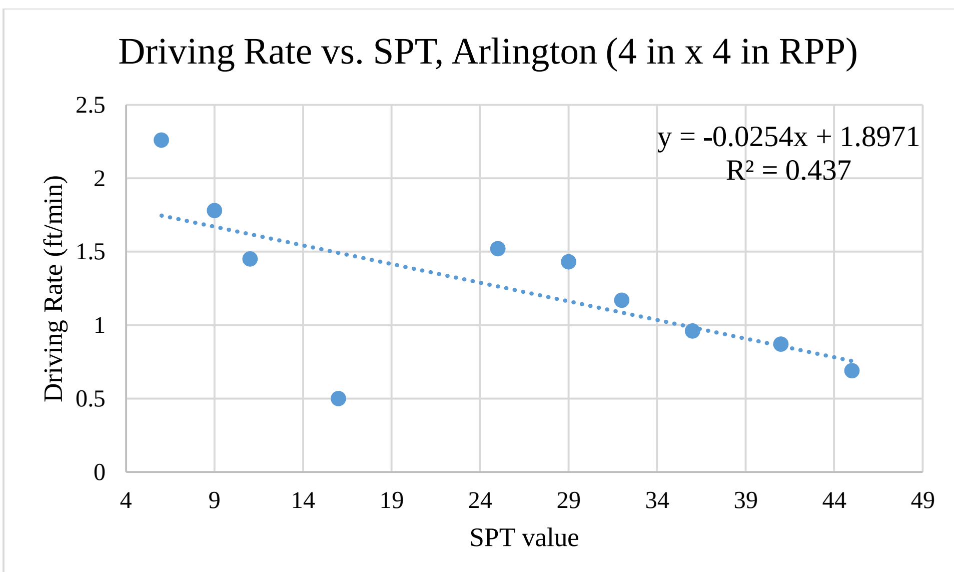


Figure 4.24: Variation of Driving Rate with average SPT value (Arlington area).

In Figure 4.24, driving rate of RPP installation was plotted against respective SPT value of the soil. It is perceptible that driving rate decreased with increasing SPT values for all the cases. Increasing SPT value refers to the soil being increasingly harder. Driving RPP through harder soil would require more time provided the applied energy remains the same and therefore it is evident that driving rate decreased.

4.2.5.5 Combined Analysis for the Effect of SPT

The SPT value of a site represents the resistance of soil to penetration and consequently the hardness of the site soil. It can be inferred easily that driving rate of RPP would be decreased when SPT value is higher.

While the SPT value of the site soil increased, the driving rate of 4 in x 4 in RPP was found in all cases (Dallas, Arlington, and Denton) to decrease resulting in higher required installation time (Figure 4.25). Increasing SPT value refers to the soil being increasingly harder. Driving RPP through harder soil would require more time provided the applied energy remains the same and therefore it is evident that driving rate decreased.

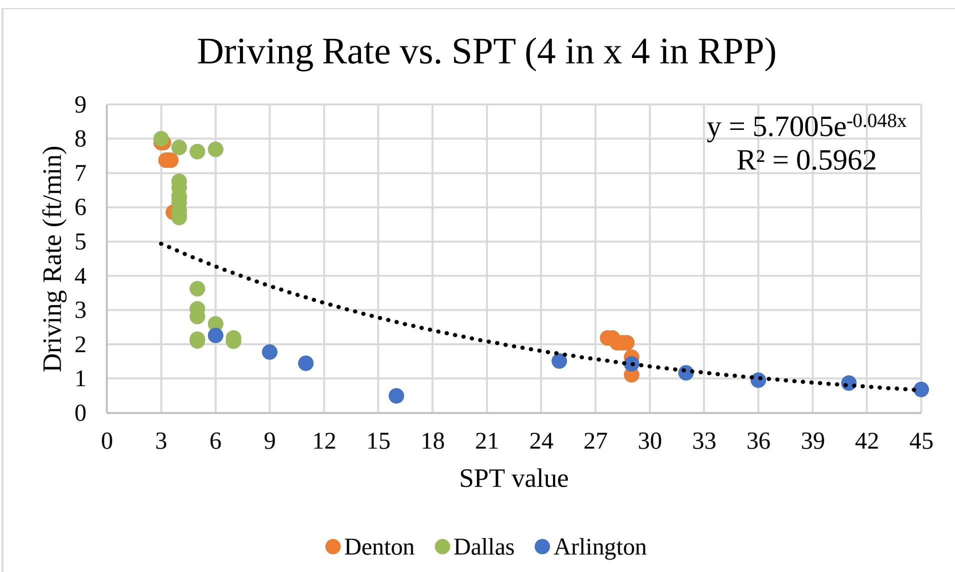
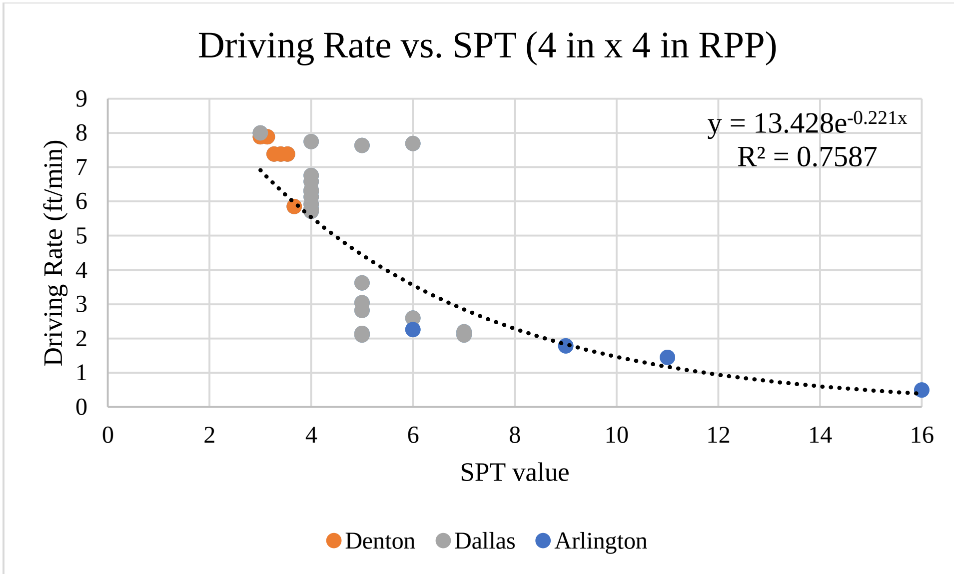


Figure 4.25: Combined plot of driving rate of RPP vs. SPT Value of site soil.

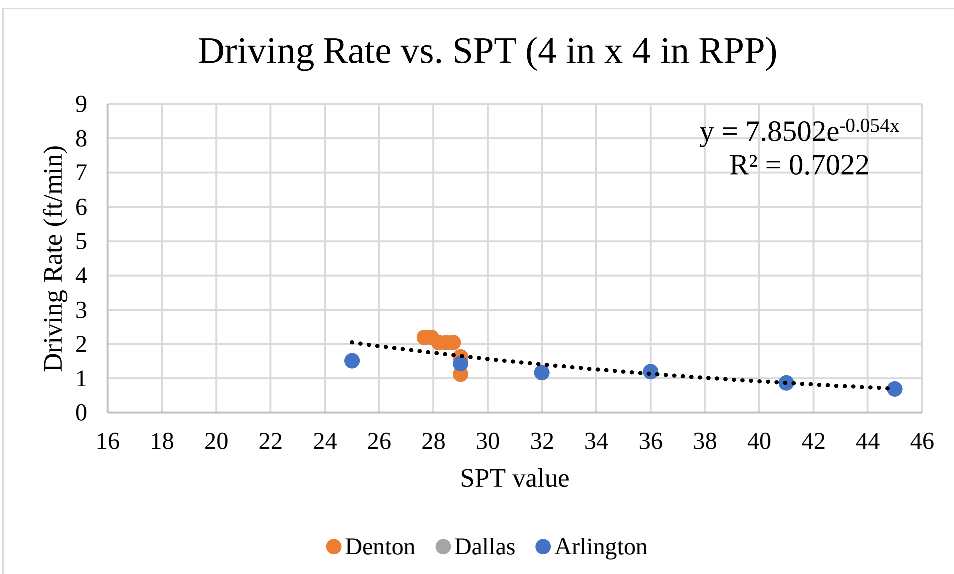
The driving rate data of 6 in x 6 in RPP were not considered for combined analysis because those RPPs were installed with different machine with varying capacity in different test sections. Also data from Irving area was discarded in combined analysis as these data were more scattered.

Figure 4.25 showed an exponential decay of driving rate with SPT value of soil. The value of coefficient of determination R^2 was found to be 0.5962 indicating that the model equation signifies 59.62% of the variation in data. However, it can be noticed from the Figure 4.25 that the change in driving rate for the lower SPT value zone where SPT value lies between 2 to 10 was very rapid. On the other hand, when SPT value was more than 20 change in driving rate was very small compared to the previous case. Therefore, an attempt to analyze the effect of lower and higher SPT value of soil separately on RPP driving rate was made. The range of SPT was divided in two as soft to stiff soil with maximum SPT value of 16 and very stiff to hard soil with SPT value more than 16.

Figure 4.26 (a) showed an exponential decay of driving rate with SPT value of soil for soft to stiff soil. The value of coefficient of determination R^2 was found to be 0.7587 indicating that the model equation signifies 75.87% of the variation in data. The asymptotic trend of RPP driving rate with SPT value of the hard soil can be found in Figure 4.26 (b). For this case, coefficient of determination R^2 was found to be 0.7022 indicating that the model equation signifies 70.22% of the variation in data.



(a)



(b)

Figure 4.26: Combined plot of driving rate of RPP vs. SPT value of site soil for a) soft to stiff soil (0<SPT<16) and b) very stiff to hard soil (16<SPT).

Chapter 5

SUMMARY, CONCLUSIONS & RECOMMENDATIONS

5.1 SUMMARY AND CONCLUSION

The present study mainly focused on the evaluation of driving rate of recycled plastic pin (RPP) with respect to different soil parameters. Assessment of driving rate and installation time of RPP has become a concern since its popularized and effective use in large geotechnical projects in North Texas. An important aspect of project planning is scheduling time with reasonable accuracy to avoid the cost escalation. Therefore, economic efficiency of RPP depends largely on the proper planning of the involving project where earlier estimation of installation time is a prime objective. Also, a better understanding of the interaction between the RPP and soil properties would assist in bringing about a more optimized design methodology for shallow slope reinforcement. The summarized outcome of the current study are as follows:

- Driving rate of recycled plastic pin (RPP) and soil properties of respective sites obtained during the installation of recycled plastic pins (RPP) in four different areas of North Texas namely Dallas, Denton, Arlington, and Irving were analyzed to study the correlation of RPP driving rate with soil parameters such as natural moisture content, dry density, cohesion, plasticity index, and standard penetration test (SPT) value.
- Driving rate increased with an increase in the natural moisture content of the site soil. Therefore, installation time decreased for the same phenomenon. The correlation was found as follows:

$$D_R = 0.3957e^{0.0819\,w};\ R^2 = 0.8324$$

Where,

D_R= Driving rate of RPP in ft./min

w = moisture content of the soil (%)

- Driving rate was found to increase with an increase in the plasticity index (PI) of the site soil. The correlation found for this case was as follows:

$$D_R = 0.3287e^{0.0718\,(PI)};\ R^2 = 0.5253$$

Where,

D_R= Driving rate of RPP in ft/min

PI = Plasticity index of the soil

- Driving rate of RPP appeared to decrease with an increase in the dry density of the soil. The correlation between RPP driving rate and soil dry density was found as follows:

$$D_R = 7708.2e^{-0.074\,\Upsilon_d};\ R^2 = 0.7542$$

Where,

D_R= Driving rate of RPP in ft/min

Υ_d = Dry density of soil (pcf)

- Driving rate was found to decrease with an increase in the cohesion property of site soil following the equation as followa:

$$D_R = 6.3522e^{-9E\text{-}04\,C};\ R^2 = 0.6273$$

Where,

D_R= Driving rate of RPP in ft/min

C = Cohesion of soil (psf)

- While the SPT value of the site soil increased, the driving rate was found in all cases to decrease and the required installation time to increase. Increasing SPT value refers to the soil being increasingly harder. Driving RPP through harder soil would require more time provided the applied energy remains the same and therefore it is evident that driving rate decreased and installation time increased. The driving rate of RPP showed a nonlinear decreasing patter with an increase in soil SPT value. It decreased exponentially with SPT value of the soil. The correlation was found to possess an R^2 of 0.5821 and was given by

$$D_R = 5.7005e^{-0.048\,(SPT)};\ R^2 = 0.5962$$

Where,

D_R= Driving rate of RPP in ft/min

SPT= Standard penetration value of soil

- The change in driving rate for soft to stiff soil was found to be very rapid. On the other hand, change in driving rate was very small for the case of very stiff and hard soil. For soft to stiff soil the correlation was found as follows:

$$D_R = 13.428e^{-0.221\,(SPT)};\ SPT \leq 16$$

$$R^2 = 0.7587$$

Where,

D_R= Driving rate of RPP in ft./min

SPT= Standard penetration value of soil

For very stiff to hard soil the correlation was found as follows:

$$D_R = 7.8502e^{-0.054\,(SPT)};\ SPT > 16$$

$$R^2 = 0.7022$$

Where,

D_R= Driving rate of RPP in ft./min

SPT= Standard penetration value of soil

5.2 Recommendations

The following recommendations are suggested for future research based on the findings of the present study:

- To study the effect of pore water pressure on driving rate of RPP.
- To study the skin friction of RPP and its influence on driving rate.
- To study the effect of topography and slope geometry on RPP driving rate and installation process.
- To study the driving mechanism of different shapes of RPP.

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