PERFORMANCE EVALUATION OF PAVEMENT REHABILITATION BY POLYURETHANE FOAM INJECTION : A CASE STUDY OF CONCRETE ROADS IN SURANAREE UNIVERSITY OF TECHNOLOGY

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การศึกษาประสิทธิภาพของพื้นผิวถนนหลังจากการซ่อมบำรุงโดยการฉีดพอลิยูรีเทน : กรณีศึกษาถนนคอนกรีต ในมหาวิทยาลัยเทคโนโลยีสุรนารี

นายซังกีย์ ปาซัง

วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิศวกรรมศาสตรมหาบัณฑิต
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Suranaree University of Technology has approved this thesis submitted in fulfillment of the requirements for a Master's Degree.

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การซ่อมแซมถนนคอนกรีต 4 ช่วงถนนโดยการฉีดพอลิยูรีเทนโฟม (PFI) เพื่อตรวจสอบประสิทธิภาพของการฉีดพอลิยูรีเทนโฟม (PFI) สำหรับซ่อมบำรุงถนนคอนกรีตในบริเวณช่วงถนนที่ทรุดตัวโดยได้ประเมินประสิทธิภาพและความเหมาะสมของวิธีการซ่อมบำรุง จากการตรวจสอบการยุบตัวของถนนภายหลังการฉีดพอลิยูรีเทนโฟมแล้ว 6 เดือนพบว่ากระบวนการ PFI นี้สามารถช่วยยกระดับถนนและปรับปรุงคุณภาพของถนนด้วยเวลาดำเนินการที่น้อยกว่าวิธีเดิม ช่วยลดความยุ่งยากในการปิดช่องจราจร อย่างไรก็ตามจากการตรวจสอบอย่างต่อเนื่องพบว่า บริเวณที่ทำการฉีดพอลียูรีเทนบางช่วงถนนเกิดการยุบตัวเนื่องจากพื้นดินใต้ถนนไม่แข็งแรงและถูกชะล้าง นอกจากนี้ยังสังเกตุเห็นรอยแยกใหม่ ๆ และความไม่สม่ำเสมอของผิวถนนอันเนื่องมาจากการฉีดพอลิยูรีเทนโฟมไม่สม่ำเสมอ

การทดสอบประกอบไปด้วย การสำรวจการทรุดตัวของถนน การหาดัชนีสภาพผิวทาง (PCI) ที่ทำควบคู่ไปกับการเจาะทดสอบพื้นคอนกรีต การทดสอบค่าซีบีอาร์ความหนาของชั้นวัสดุในสนาม (Dynamic Cone Penetrometer, DCP) และการจำแนกชั้นดิน ข้อมูลจากการเก็บรวบรวมได้ถูกนำมาวิเคราะห์หาความสัมพันธ์กันในแต่ละสถานที่ ซึ่งจากการเจาะทดสอบพื้นคอนกรีตพบว่าโฟมกระจายตัวไม่ดีกับเนื้อของคอนกรีต ในขณะเดียวกันการทดสอบ DCP และการจำแนกชั้นดินถูกใช้สำหรับตรวจสอบคุณภาพของวัสดุมวลรวม การสำรวจการทรุดตัวของถนนถูกแสดงให้เห็นในรูปของคุณภาพพื้นผิวถนนขณะขับขี่ และ PCI ถูกกำหนดโดยใช้ ASTM 6433-03 ที่แสดงให้เห็นว่าพื้นทางยังคงจำเป็นต้องได้รับฟื้นฟูเพิ่มเติม

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SANGEY PASANG : PERFORMANCE EVALUATION OF PAVEMENT REHABILITATION BY POLYURETHANE FOAM INJECTION : A CASE STUDY OF CONCRETE ROADS IN SURANAREE UNIVERSITY OF TECHNOLOGY. THESIS ADVISOR : ASST. PROF. THIRAYOOT LIMANOND, Ph.D., 141 PP.

POLYURETHANE FOAM INJECTION/ PAVEMENT REHABILITATION/ DYNAMIC CONE PENETROMETER/ PAVEMENT CONDITION INDEX

Four project sites of Reinforced concrete pavement were recently rehabilitated for distresses by using polyurethane foam injection (PFI). To evaluate the performance and suitability of the rehabilitation method an investigation was performed. Differential settlement at the rehabilitated pavement sections was monitored for a period of six months. It was found that the PFI process had successfully lifted and aligned the pavement at the desired level, and improved the ride quality. In addition, the process was less time consuming than conventional method of slab jacking, which minimizes the disruption to the traffic flow. However, the continuous monitoring showed some settlements at the polyurethane injected pavement sections, which was possibly caused by the weaker subgrade and scouring of the base material. In addition, several new cracks and uneven slabs settlements were observed at those sections, which could be because of uneven support created while injecting polyurethane foam beneath the pavement sections.

Various tests were conducted with distress survey and Pavement Condition Index (PCI) along with removing concrete core, Dynamic Cone Penetrometer (DCP) and Soil classification. The data collected were used for correlating it to the result obtained from differential settlement. Removal of the core testing showed that the polyurethane foam was not well distributed along with the condition of the concrete while the DCP and soil classification were used to inspect quality of the base material. Distress survey presented a compressive summary on the ride quality of the road pavement and PCI was determined by using ASTM 6433-03 which indicated that the pavement still required additional rehabilitation.

School of Transportation Engineering	Student's Signature ________________
Academic Year 2010	Advisor's Signature ________________
	Co-advisor's Signature ______________

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D.47 Modulus of subgrade reaction from Core#15 at Sikkhawithee 1(b) Road 130

D.48 Modulus of subgrade reaction from Core#16 at Sikkhawithee 1(b) Road 130

CHAPTER I

INTRODUCTION

1.1 General Background

In road way system, continuous maintenance, repair, and reconstruction are required to be executed. When complete reconstruction of a highway is unnecessary, other various rehabilitation techniques are used to renew pavements more economically. The rehabilitation of existing rigid pavements has been a challenging course of action for decades despite advancement in pavement technology. Several methods exist for the correction of differential settlement of concrete pavement such as slab jacking, overlays; and slab replacement. To reduce the probability of recurring pavement failures, the root causes of the problems need to be known and the lessons learned integrated into future rehabilitations and project designs. Using the information from the existing and the current rehabilitation method, the underlying causes of the problem can be determined so that decisions can be made regarding rehabilitation strategy.

Conventional methods for raising concrete slabs to align roadway sections or to remedy subsidence require: (1) pressure injecting grout under the slab or (2) full-depth repair (Soltesz, 2002). However, these two methods take a long time to complete and cause traffic disruptions. A patented process that uses dense polyurethane foam to lift, realign, underseal, and to fill voids under concrete slabs via 16 mm diameter holes has been used by several States (Gaspard and Morvant, 2004; Soltesz, 2002).

Polyurethane foam injection was tried in many places since its introduction for correcting the pavement settlement and alignment problems with minimal traffic disruption or downtime. It was reported by Chen and Won (2008) that although polyurethane foam injection was able to raise the slabs to the target levels, on some projects they were not able to maintain the target elevation or prevent slabs from further settlement. New cracks are seen in some slabs that had previously received polyurethane foam injection. Chen, Won, and Hong (2008) concluded that polyurethane foam injection worked on most projects in raising slabs to the target levels; however, the long-term effectiveness in keeping slabs at target levels is doubtful.

1.2 Project Location: Suranaree University of Technology

It has been 20 year since the establishment of Suranaree University of Technology in Nakhon Ratchasima, Thailand. It is a public autonomous university under Royal Thai Government supervision. Since its foundation in 1993, there are 54.2 km lengths of road. There is very little history on the major rigid pavement rehabilitation until year 2009. Suranaree University of Technology carried out an advanced approach for lifting pavement by injecting polyurethane, a technique relatively new in Thailand, to slab jack the distressed pavements.

Rigid pavements in Suranaree University have been noticed to experience severe pumping and differential settlements among slabs of the pavement, resulting in slab cracking and exhibiting poor ride quality. Thus, SUT tendered out the work of polyurethane injection beneath these distress pavements in 2009. The following are the road pavements undergone polyurethane injection in august of 2009.

1. Withayawithee 2 Road- 55 m of the road and 138.51 m from Withayawithee Road.
2. Withayawithee 3 Road- 30 m of the road and 342.09 m from Withayawithee Road.
3. Sikkhawithee 1(a) Road- 27.5 m of the road and 263.41 m from Sikkhawithee Road.
4. Sikkhawithee 1(b) Road- 12.5 m of the road and 28.15 m from Sikkhawithee Road.

These roads have very minimal traffic in comparison to the Highway since it is utilized only for vehicle complying within the university. The heaviest trucks are those ten wheeler concrete mixer trucks of 23-27 tons weight and the repetition is of maximum 3-4 times a day, in addition these heavy vehicles use only Withayawithee 2 Road. Other vehicles travelling through these roads are light vehicles belong to the staff and student of the university.

1.3 Objective of the Research

1. Evaluation and visual monitoring of the road segments injected with polyurethane to realign and underseal the pavement.
2. Assessment of the pavement structure, profile, and the ride quality of the polyurethane injected roads in SUT.

1.4 Scope and Limitation of the Research

In this study, the pavements which were rehabilitated by injection of polyurethane to realign and level the road are investigated by carrying out leveling survey on road, before and after the injection. The latter leveling were performed one month, two months, four months and six months after the injection. This contributed in monitoring the differential settlement during the mentioned period. Distress survey was also correlated with the settlements at the sites.

A Dynamic Cone Penetration test was conducted at the rehabilitated pavements to find the California Bearing Ratio (CBR) of soil in place to check the correlation of differential settlement to the strength of the subgrade. A soil classification is also performed to study the nature of the sub-grade and its contribution to the distress of the pavement.

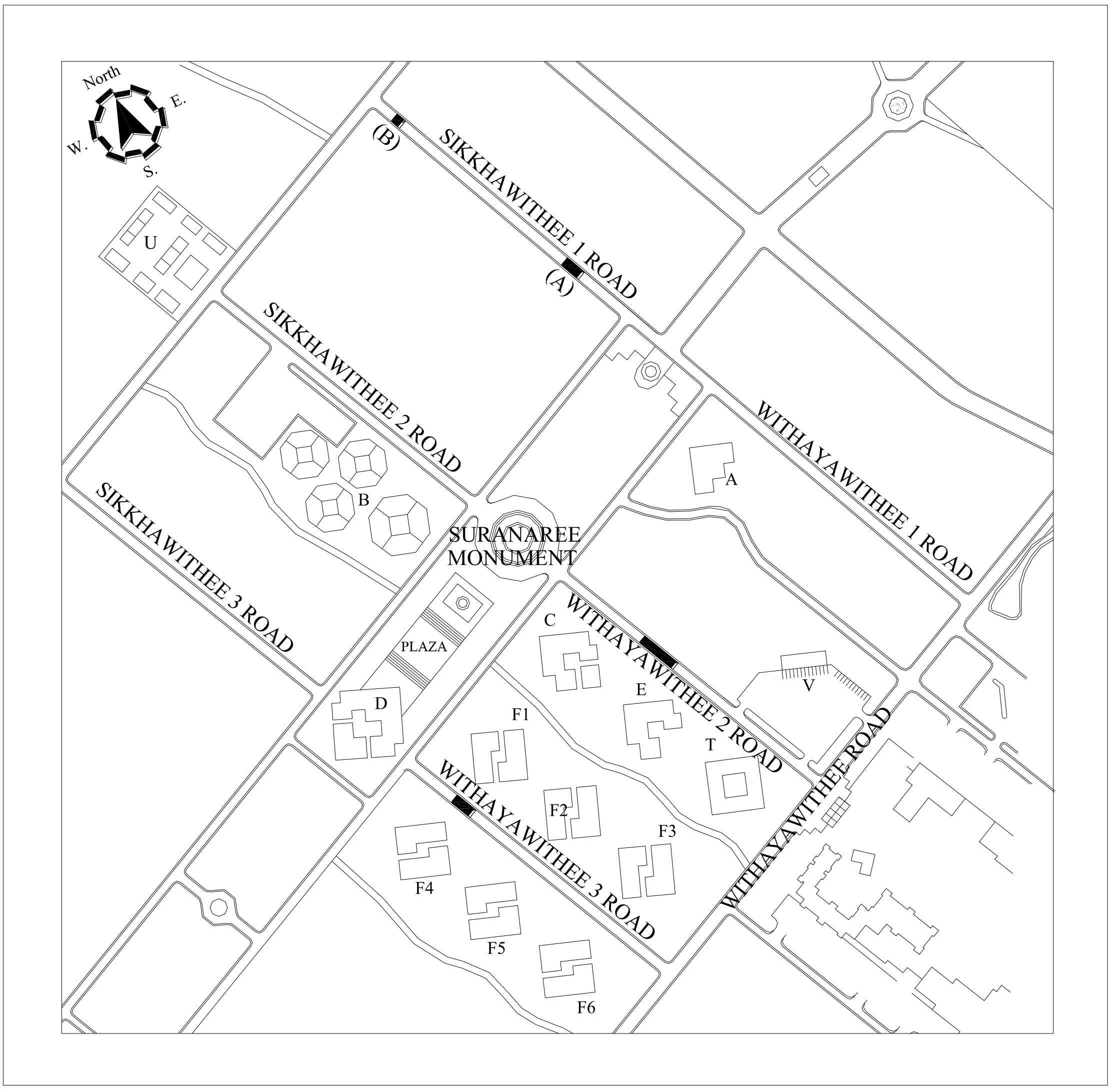

Figure 1.1 Project site map. The shaded parts on the roads are the rehabilitated section.

CHAPTER II

LITERATURE REVIEW

2.1 Pavement Distress

Distresses in rigid pavement are caused by two main problems: deficiency of pavement material; and structural inadequacy of the pavement system. The former is caused by use of nondurable materials, scaling, freezing and thawing while the latter is due to the overloading (Yoder and Witczak, 1975). However, a failure in the pavement may be due to combinations of number of different distresses. Common type of distresses in a concrete pavement are pumping, faulting and cracking.

2.1.1 Pumping

Pumping is the ejection of water and sub-grade materials through the joints and cracks or through the pavement edge, caused by deflection of the slab after free water has accumulated under the slab. When more materials are pumped out, there is a substantial loss in the fine grained soil from the sub-grade, resulting in considerable loss of sub-grade support at these locations. The pavement cracking due to mud pumping is generally a progressive type of failure in rigid pavement.

2.1.2 Blow Ups

Blowups occur in hot weather at a transverse joint or crack that does not permit the expansion of concrete slabs. The infiltrations of incompressible materials into the joints or cracks cause blockage to the expansion joints in the rigid pavement. Thus a localized upward movement of slab edges or shattering occurs in vicinity of the joints (Huang, 1993).

2.1.3 Faulting

Faulting is the difference in elevation across a joint or a lane or a crack. It can be either longitudinal or transverse faulting. Faulting is caused partly by a buildup of loose materials under the trailing slab near joint and cracks as well as the depression of the leading slab (Huang, 1993).

2.1.4 Cracking

Cracking is associated with all the distresses. Cracking may be of longitudinal, transverse or diagonal. Longitudinal cracking generally occurs parallel to the centerline of the pavement which is usually caused by heavy load repetition, loss of foundation support, and curling and warping stresses. Improper construction of the longitudinal joints can also lead to this cracking. Transverse and diagonal cracking are caused by a combination of heavy load repetitions and stress due to temperature gradients, moisture gradient and drying shrinkage. (Huang, 1993)

2.1.5 Spalling

Spalling-cracking, breaking, chipping, or fraying of concrete slab edges at joints and cracks-is a common distress in jointed concrete pavements. Spalling reduces pavement serviceability, and if left unrepaired, becomes hazardous to highway users. Spalling is caused by high-compressive stresses that are developed in the concrete when joints or cracks cannot properly close because of presences of incompressible materials. The depth of spalling in a concrete slab may vary from a few millimeters to the full depth of the slab. Once begun, spalls tend to grow or propagate under repeated thermal stresses and traffic loadings.

2.2 Methods to Rehabilitate Pavements

2.2.1 Mud Jacking and Under Sealing

Mud jacking material general consists of a mixture of soil and cement mixed into slurry, which is then pumped as mud jacking. It is used as a maintenance measure after pumping has taken place. Bituminous under sealing are generally made up of relatively heavy cements or slow to medium curing cutback materials. The materials are heated and pumped under the slab until some materials are extruded at joints or at the pavement edge.

Usually it is one day work; the pavement can be reopened to traffic immediately after completion of the process. Mud-jacking typically costs $50 to $70 per square meter of pavement. The process is environmentally friendly because only cement and soil are used as grouting materials. However, several lanes of traffic must be closed at once, and if voids remain, the pavement can settle and break. In addition, the access holes are relatively large and must be filled (Al-Eis and LaBarca, 2007).

2.2.2 Hot Mix Asphalt Overlays

Strengthening may be done by providing additional thickness in one or more layers over the existing pavement, which is called overlay. The cost of a 3.8 cm thick HMA overlay, including milling of the existing pavement, is between $54 and $78 per square meter (Al-Eis and LaBarca, 2007). The pavement may be used on the day of completion. This method requires regular crack sealing and there are requirements for minimum lift thickness of HMA overlay.

2.2.3 Slab Replacement

Slab replacement is the process of completely replacing the old pavement concrete with a new one. This involves construction of a new section of slab which takes several days to complete. The total cost of the process depends on the type of concrete used and thickness of the pavement. It involves hidden cost while disrupting the traffic for long time.

2.2.4 Injecting Polyurethane Beneath Distress Pavement

The polyurethane injection Method is a process that employs expansive high-density polyurethane foam to lift, realign, and under-seal concrete slabs, and to fill voids between the pavement and the base or sub-grade. The polyurethane foam injection was first distributed and used with license in 1988. The installation cost of the rehabilitation is determined by the amount of polyurethane material used during the repair. The price that was typically charged for labor, equipment use and material was $14 per kg of foam used (Al-Eis and LaBarca, 2007). And recently it has been introduced in Thailand for treating building and road foundations. Thus, Suranaree University of Technology decided to use the polyurethane foam injection Method to realign the sections of pavement within it campus.

As described in the white paper company article “The URETEK Method Drives More Effective Road and Highway Maintenance” by Janothan Kantor, it involves two process of injecting the polyurethane. The first step involves finding the location and the size of the voids. This is usually conducted by using a Hyper Optic Void Detection System which scans the pavement to give the location and the size of the voids. When the Hyper Optic Void Detection System is unavailable or inappropriate, a Dynamic Cone Penetrometer (DCP) is used to conduct a series of test to measure the density of the underlying soil across multiple locations of the repair area. This information is used to identify and determine the specific areas that have weaker soil base and helps in determining the exact locations where the injections needs to take place (Kantor, 2004).

The second step illustrates the way to fix the voids. Once the penetrometer tests have been completed, the specific locations and depths of weaker soil strata are logged. Using this information, special probe drills are used to measure and drill an injection pattern that includes a series of 1.27 to 3.81 cm holes encompassing the entire identified area. The injection grid is designed to strengthen the soil directly underneath the structure as well as the surrounding area. The polyurethane, which is a combination of two different chemicals, is injected into the ground to facilitate the injection process. As the pavement is lifted, its movement is precisely monitored on the surface using laser level measuring devices. The soils are compacted and nearby fissures and voids are filled. Since the polymer is hydro-insensitive, any ground water in this expansion path is pushed aside. This property allows for injections to occur under any type of ground condition, including heavily saturated soils (Ureteck USA, 2004; Uretek USA, 2005).

The followings are the advantages of the polyurethane foam injection method compared to conventional slab jacking techniques (Uretek USA, 1998):

1. Shorter repair time- The polyurethane reaches 90% of its full compressive strength within 15 minutes from injection, at which time the roadway can be reopened to traffic.
2. Good void filling characteristics.
3. High compressive and tensile strengths.
4. Fewer holes and smaller holes reduce the chance of weakening the slab.

5. Injected material is lightweight, reducing the likelihood of settling or further subsidence.
6. Void-filling characteristics of the material reduce the chance of water infiltration.
7. Inert behavior in many environments provides a long-term, stable support for the slab.
8. Repair process is more controllable. Successive lifts can be applied easily and quickly, providing the means to incrementally raise slabs.

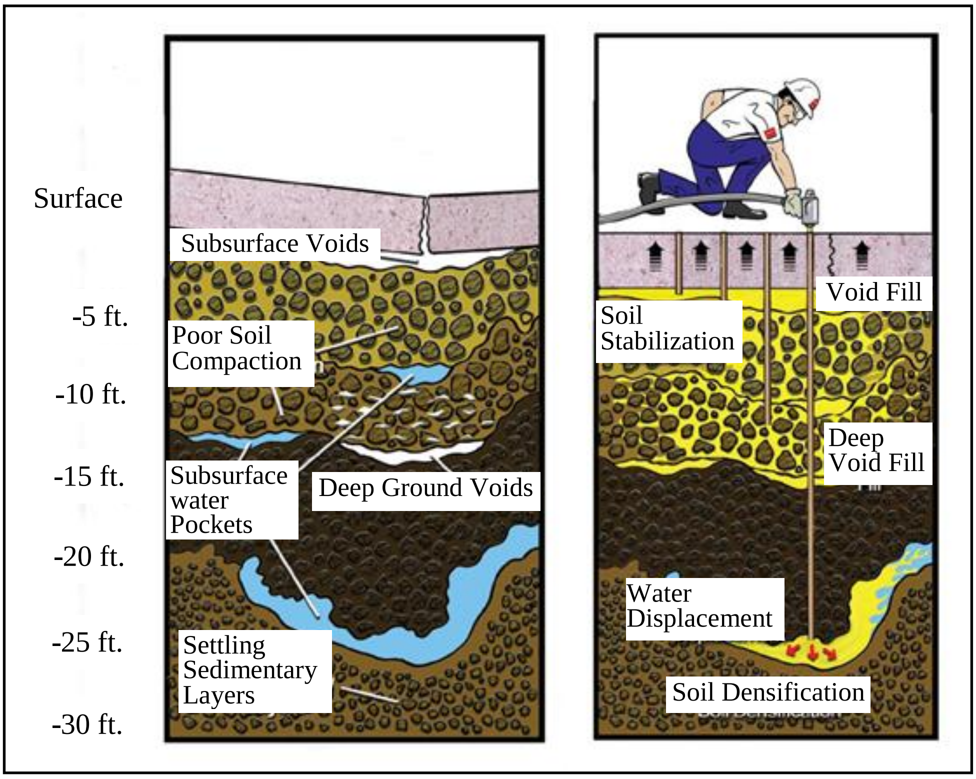

Figure 2.1 (Left) Elements of a shifting/unstable/damaged underground base and (Right) the polyurethane material injection (Ureteck USA, 2004).

2.3 Performance of Polyurethane Injection Beneath Pavement

Time constraint, weakening of pavement by large access holes, ineffective void fillings are some of the few difficulties encountered when conventional method of rehabilitation of pavement is used. Thus a more advanced method of lifting the concrete pavement by injecting polyurethane was introduced. This method also gave rise to several researches which all have a mixed conclusion; some reported improvement in the ride quality while other pointed out that it is only a temporary improvement where a series of cracks are noticed after the polyurethane injection.

A research effort conducted by the Louisiana Department of Transportation yielded positive results even after several years. Gaspard and Morvant (2004) evaluated the rehabilitation method utilizing the injection of polyurethane into the pavement structures on continuously reinforced concrete pavement (CRCP), jointed concrete pavement (JCP), and bridge approach slabs. The polyurethane injection was used to fill voids and level the CRCP and bridge approach slabs. On JCP, it was used to reduce faulting, fill voids, and under seal. Test results indicated that injecting polyurethane into the pavement structure is an effective method of leveling CRCP and bridge approach slabs. On the CRCP and bridge approach slabs, IRI values were reduced from 33 to 68 percent, while as much as 2 inches of depression was removed from the slabs. The approach slabs are still in good condition after four years, and no additional slab faulting has yet occurred.

A polyurethane foam injection method evaluation project that focused primarily on raising bridge approach slabs was conducted in Oklahoma in 1994. Pavement lifting was conducted in six divisions around the state, and in three of these divisions, cracking occurred during or just after the injection process. In one case, a

PCC slab broke in half during injection. However, data from subsequent monitoring of the Oklahoma test locations are not available (Brewer, Hayes, and Sawyer, 1994).

An evaluation of polyurethane injection was conducted by the Michigan Department of Transportation after raising and under sealing concrete pavement slabs in Monroe County. Opland and Barnhart (1995) selected three sites on I-75 (truck lane) for test and control sections. The pavement consists of 25.4-27.9 cm reinforced concrete on an open-graded base. The polyurethane did improve the base support where the pavements were cracked, but it raised the pavement and provided a temporary increase in base stability where the pavement was severely faulted. There was also an initial improvement in ride quality. However, the ride values reverted back to pre-construction values after just one year. It was also recommended in the report that polyurethane injection was not to be used as a substitute for mud jacking for pavements with open-graded bases. However, polyurethane should only be considered as an alternate to mud jacking on pavements with dense-graded aggregate bases.

In June 2000, the state of Oregon carried out a bridge and approach slab lifting and re-aligning project using polyurethane injection. Soltesz (2002) studied the pavement performance every three to six months for two years. Three PCC slabs were raised approximately 89 cm using a total of 4,650 pounds of polyurethane material. After three-month, crack and elevation survey showed that cracking and settlement had occurred in the slabs. In addition, after two years it was noted that several injection holes had not been properly sealed and experienced raveling during that time. Injection holes that had been properly sealed were performing adequately. Thus injected polyurethane can successfully raise concrete slabs to a target profile but it is unable to maintain the profile for long.

The Wisconsin Department of Transportation (WisDOT), in 2007, reported the effectiveness of using the polyurethane to re-establish Portland cement concrete pavement elevations. Al-Eis and LaBarca (2007) conducted the polyurethane injection at two test sites; both are leading approach slabs to two bridges. Though the process of injecting polyurethane beneath the distress slab was successful and lifted the slabs to desired level, the time taken was more than anticipated. They carried out the study for five at one site and at other sit for one-half years and finally concluded pavement ride quality improved at both the test sites. Hairline crack was developed at the approach slabs after six month. However the ride quality remained at a comfortable level. Minor settlement of the approach slab was also noticed during ride quality survey after one year.

Polyurethane foam injection was also tried by several Texas Department of Transportation (TxDOT) districts to correct pavement settlement and alignment problems with minimal disruption or downtime. Chen et al. (2008) presented in their paper that the performance of three rehabilitation methods on six highways to restore the condition of concrete pavement: (1) polyurethane foam injection (PFI), (2) dowel bar retrofit (DBR), and (3) full depth repair (FDR). Polyurethane foam injection worked on most projects in raising slabs to the target levels; however, the long-term effectiveness in keeping slabs at target levels is again doubtful. As for the effectiveness of full-depth repair to restore pavement condition, additional pavement distresses were quite often observed around these repaired areas. In addition, new cracks have been observed in some slabs and it was believed that polyurethane foam injection might have created uneven supports that allowed the truck traffic to shatter the slabs.

2.4 Properties of Polyurethane Foam Material

The Oregon Department of Transportation researchers investigated the ability of the Polyurethane material to penetrate small holes when extensive void filling in the sub-base would be necessary to provide pavement lift. The hole penetration was checked to find out the expected characteristic of polyurethane injection that if the material infiltrates small openings as it stabilizes the sub-grade. The capacity to fill small voids also reduces the overall water permeability of a grade, which can protect the grade from further instability. This also determined the smallest hole that the material could pass through as a function of the distance from the injection point. Tubes were designed and constructed with varying size of holes drilled ranging from 1.6 mm to 13 mm. After sealing the tube at both ends, polyurethane was injected. The result shows that none of the 1.6 mm holes was filled but polyurethane penetrated through all holes 6.4 mm in diameter and larger (Soltesz, 2002). Based on the above results, it was concluded that the polyurethane injection would have penetrated all the opening (based on the smallest dimension) as small as 6.4 mm holes drilled at every 1.2 m. Openings with a minimum dimension of 3.0 mm would have penetrated up to 0.62 m from the injection point. It was also theorized that actual field injection probably resulted in a greater penetration due to higher pressures from the constraining weight of the slab. This ability to fill small spaces should allow injected polyurethane to effectively stop the flow of water through the sub-grade wherever the polyurethane has been successfully injected. Soltesz (2002) also studied the compression strength of polyurethane foam from the sample and concluded that the strength of polyurethane product does not appear to decrease after 23 months of exposure to air and underground conditions.

Figure 2.2 Penetration tubes after injection (Soltesz, 2002).

Gauer and Schmalz (2007) conducted cyclic compression tests on samples of polyurethane resin. The result showed that total deformation after 20,000 load cycles is dependent on the range of the maximum load. For the polyurethane samples after 20,000 load cycles the total deformation was around 0.4 mm (for density 100 kg/m^3) and 0.2 mm (for density 200 kg/m^3), although most of the deformation (0.2 and 0.3 mm) could be attributed to the consolidation at the start of the tests. Hence, subsequent creep deformation was only around 0.2 or less. It was concluded that under normal traffic loads practically no creep deformation can be expected in polyurethane injections. The study also conducted water permeability of the polyurethane foam and concluded that no water can penetrate the internal cavities of polyurethane due to its close-cell structure.

2.5 Experimentation

2.5.1 Profiling

Soltesz (2002) carried out elevation monitoring by drilling twelve surveying caps into the slabs to monitor vertical displacement over time. Baseline elevation measurements were made four day after the slab jacking. Additional measurements were made at 3, 6, 12, 18, and 24 month after the slab jacking. The relative change in elevation as a function of time, using the first set of measurements as a baseline had been deduced from the survey.

From the elevation study, it was clear that the roadway settled at all twelve positions with a maximum decrease in elevation of 10.5 mm. The majority of the settling occurred within the first three month after injection with one position sinking 7.2 mm during that period. The settling continued for two years after the injection. The reason for the settling was not investigated, and it was not known whether the settling would continue further. The largest settlement was at the joint between the roadway slab and the end-panel, which had the largest elevation discontinuity before injection. At some joints the settlement was found to be nearly the same amount; therefore, the ride over the joint was not affected by the settlement. However, the difference in elevation between the ends of the end-panel increased over two years. This change in elevation also led to new cracks after the injection at the end-panel.

The observed settling though minor, after two years of service, highlights the fact that slab jacking, regardless of method, does not guarantee a static slab. In addition, raising a slab could crack the slab or open up existing cracks. Good technique and experience are essential to achieve a successful outcome, but they do not guarantee success. Arguably, injected polyurethane provides more control in the lifting process, and may produce a higher percentage of successfully raised slabs, then injected grout or mud.

Gaspard and Morvant (2004) used a walking profiler to measure the elevation different before and after the injection of polyurethane beneath the distress continuous reinforced concrete pavement (CRCP). The ARRB Walking Profiler is a precision instrument designed to facilitate the efficient collection and presentation of continuous paved surface information, including distance, profile, grade, and International Roughness Index (IRI) measurements. The Walking Profiler enables accurate recording of measurements for the actual profile, grade, and level for surfaces such as paved roads, footpaths, runways, building slabs, and sporting surfaces. From the analysis of the profile and the IRI values before and after the polyurethane injection, the pavement was raised 5.128 cm on average while IRI was significantly reduced from 57 to 68 percent.

2.5.2 Pavement Condition by Coring

Elifino and Hossain (2007) collected seven cores from the investigated sections to assess the in situ condition of the failed pavement sections. The cores included soil cement, concrete slab, drainage layers. The core collected from the midslab crack was propagated through the drainage layer but did not damage the soil cement layer. Core samples from the longitudinal joint showed that the differential settlement between the travel and acceleration lane caused the tie bar to bend. The dowel bar was found to be loose and the hole around the dowel bar was elliptical shape instead of circular indicating poor load transfer. A consistent wet condition was observed under the damage slabs while coring. The water was noticed infiltrating through the drainage layer from the surrounding pavement areas into the bore holes during the coring operation especially where there was severe damage pavement slabs. The damage slabs were also found to have clogged drainage layer with red soil. However, the undamaged slabs drainage layers were clean.

Chen and Won (2007) investigated the cracking on concrete pavement which was caused by the late or shallow saw cutting of longitudinal saw cut by coring. Numerous cores were taken from on and around the irregular cracks. Since there was no clear facture face on the core it was concluded that the cracking is not related to the loss of support or subsidence. Figure 2.3 shows the condition of crack on the core. The proper dowel bar installation requires one side firmly bonded to the concrete and loose on the other side of the joint. The cores showed that the dowel bars are not properly installed. Both the ends of the dowel bars were strongly bonded to surrounding concrete which restraint the concrete from relieving the environmental stresses. This caused the mid-depth horizontal cracks near the dowel bar. The core through the transverse crack joint shows that cracks had developed under the saw cut. It was theorized that late or shallow saw cutting of longitudinal saw cut joints had caused irregular longitudinal cracks. The cracks developed under the saw cut which are wider had relieved the stress and prevented the irregular cracks appearing on the surface. Those wider cracks had mitigated the stresses due to environmental loading (Chen and Won, 2007). However, those tighter cracks under the saw cut showed numerous irregular cracks since the space was not sufficient to relieve the stress that caused the irregular surface cracks.

Figure 2.3 (Left) Core#2 condition as cracks appear to have had formed long ago. (Right) Condition of Core#1 and Core#3 as the mid-depth horizontal crack near the dowel bar was due to the locking action (Chen and Won, 2007).

Chen et al. (2009) investigated the settlement of a jointed concrete pavement on U.S. 75 in the Paris District of the Texas Department of Transportation (TxDOT) experiencing severe pumping and settlement along its longitudinal construction joints. Four core samples were taken along the longitudinal construction joints based on settlement: very severe, severe, moderate, and no settlement. At each site, the exact coring location was determined by Ground Penetrating Radar (GPR) so that the cores would contain tie bars. The core at the largest settlement showed that the tie bar was ruptured and corroded which led to extensive reduction in cross section called necking. The same tie beam failure was also observed on the core at severe base settlement. While condition under the slab examined at those core holes, voids were detected and only coarse aggregate at the base were obtained. From the cores from slabs with moderate and no base settlements, it indicated the use of keyed joints as well as tie bars along the longitudinal construction joint.

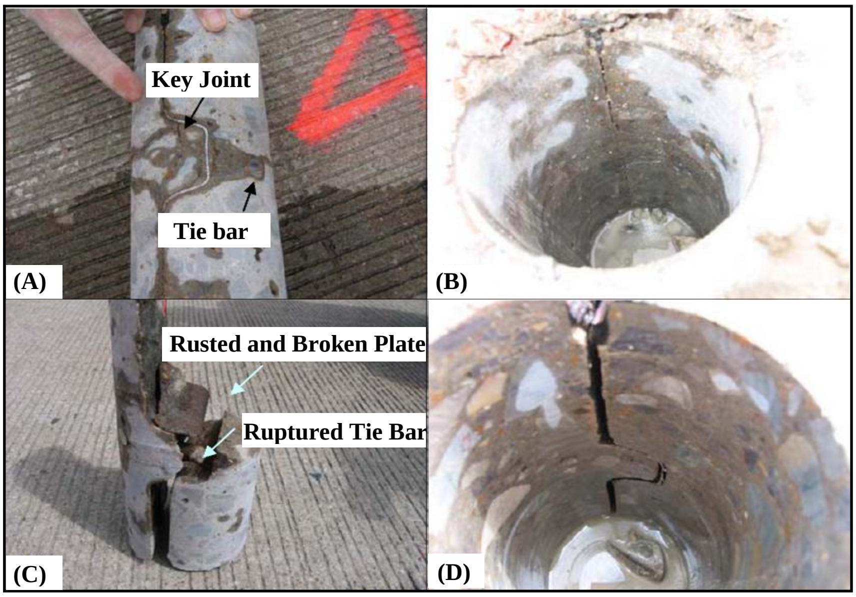

Figure 2.4 Comparisons of the key joint conditions (A) Core showing good core with key joint configuration. (B) Core hole condition- joint width is very tight and no settlement. (C) Core showing broken core with rusted plate and ruptured tie bar. (D) Core hole condition-joint width is much wider and with statement (Chen et al., 2009).

However, shown in Figure 2.4(D), the keyed joint did not prevent the rupture of the tie bar and settlement of the slab, even though the settlement was quite small. The core at the no settlement shows a much smaller width of the longitudinal construction joint than that of core at moderate settlement, as shown in Figure 12(B) and 12(D). Thus the condition of the keyed joint at no settlement might be expected to deteriorate, in the long run, to the condition as in moderate settlement. Through the examination of the core holes, it was determined that there was no void under these locations, and the extraction of the base material through the core holes showed no evidence of pumping, as the fine material was present (Chen et al., 2009).

Gaspard and Morvent (2004) assessed the polyurethane injection on Continuously Reinforced concrete pavement and Bridge approach slab. Coring was carried out on both the sites to investigate the condition of the base material and the distribution of the polyurethane underneath the pavements. At the Continuously Reinforced concrete pavement site, even though settlement was not noticed, the coring was carried out. Two core samples (one at the outside lane and another at the inside lane) were collected at this site and examined. The outside lane core sample was broken during the coring operation. The asphaltic concrete base course was permeated with polyurethane. It was speculated that the asphaltic concrete in this area was stripped and weak which allowed the polyurethane to infiltrate it. The polyurethane specimens were found to be so dense that a ball point pen could not be pushed into the specimen. Since the core hole was filled with water, the sand shell base course could not be observed. The core at the inside lane was intact during the coring process but broke while taking out core hole. The asphaltic concrete base course did not show any sign of being infiltrated as in the case of outside lane core.

The sand shell base course was clearly visible and no sign of polyurethane foam was seen.

Eight cores were collected at the Bridge Approach site out of which three showed the presence of polyurethane. Though polyurethane was injected into the base course and sub-grade on an approximately 1.2 x 1.2 m grid pattern, it was found at only three locations. Thus concluded that the polyurethane is deeper than the core depths or it did not spread out to the locations cored. The cores with polyurethane contained in thickness of 1.27, 6.35, and 12.7 cm in three cores. However, it was concluded that an indication of voids being filled with polyurethane does not necessarily translate into increase in long term pavement performance.

Figure 2.5 Concrete cores with polyurethane (Gaspard and Morvant, 2004).

2.5.3 Dynamic Cone Penetrometer (DCP) Testing

Dynamic Cone Penetrometer (DCP) is one of the economical alternatives for characterization of pavement layer qualities. The dynamic cone penetrometer (DCP) has been used extensively in the past decade to evaluate penetration resistance in aggregate base course (ABC) and sub-grade layers of pavement structures. The DCP, also known as the Scala penetrometer, was developed in 1956 in South Africa and originally designed for evaluating pavement layer strength. Extensive research has been performed to develop empirical relationships between DCP penetration resistance and California bearing ratio (CBR) measurements (Gabr, Hopkins, Coonse, and Hearne, 2000).

In addition, it is fairly easy to collect and analyze data with DCP (Chen, Lin, Liau, and Bilyeu, 2005). The study after testing couple of equations for the calculation of California Bearing Ratio (CBR) and Base Modulus from the Penetration rate obtained by DCP suggested the following equations (2.1) and (2.2):

$$\log CBR = 2.465 - 1.12 (\log PR) \text{ or } CBR = 292/PR^{1.12} \quad (2.1)$$

$$E (psi) = 2{,}550CBR^{0.64} \text{ or } E (MPa) = 17.58CBR^{0.64} \quad (2.2)$$

where CBR is the California bearing ratio and PR is the DCP's penetration rate through the layer (mm/blow). The equation (2.1) was originated by the U.S Army Corps of Engineers and the equation (2.2) from Powell et al. (1984).

Chen and Scullion (2008) used DCP to evaluate cracks on the joint concrete pavement (JCP) section of Texas State Highway 342 (SH342) that are propagating to adjacent slab after a full depth repair (FDR) was performed. DCP tests were performed on the cracked and non-cracked areas to verify if the cracks observed on SH342 were due to weak foundation support. The equations suggested in previous study (Chen et al., 2005) were used to derive the CBR and Base moduli from DCP. A penetration rate exceeding 38 mm per blow indicates very weak foundation support, with a CBR value of 5 and modulus of 48 MPa (7 ksi). The DCP results showed that the base and sub-grade in the cracked areas are very weak. Some locations had a penetration rate exceeding 125 mm per blow. The DCP results showed that the base/sub-grade support at non-cracked areas is slightly better than that in the cracked areas, but the support was still considered to be quite substandard. It was theorized that, with time, the weak support under the concrete slab will cause the cracks to propagate to the non-cracked areas as traffic loads are applied (Chen and Scullion, 2008). A DCP test was also performed on a repaired area where cracks have reappeared and found that the base support is very poor (DCP determined base moduli ~28 MPa or 4 ksi) even after the FDR. It was concluded that the base was not properly repaired during the FDR which used crushed limestone aggregates in the FDR concrete patch. DCP results indicated very low stiffness of the base, and the effectiveness of the lime stabilization has disappeared.

Chen et al. (2008) conducted a total of 11 DCP tests; samples were from good and bad areas at frontage road of Cement Treated Base (CTB). All DCP data were collected in the outside lane of the bad area in conjunction with core hole and FWD test locations. Although the maximum penetration depth is only 1 m, DCP data provided a good indication of sub-grade stiffness, and they were used to verify FWD deflections and back-calculated moduli. The DCP results indicated that the penetration rates in all test holes were typical for lime treated sub-grade, and raw sub-grade. From the DCP results, the modulus values for the lime treated sub-grade layer range from 227 to 469 MPa (33–68 ksi) and for the subgrade range from 69 to 145 MPa (10–21 ksi). On average, the modulus values for the CTB are high, but there were substantial variation. Some locations have stiffness values of less than 1.03 GPa (150 ksi), but there are as high as 13.79 GPa (2,000 ksi). It was observed that the variation in CTB modulus values along the length of the project were very similar to the variation in pavement roughness (Chen, Scullion, Lee, and Bilyeu, 2008).

Chen and Won (2007) tested longitudinal cracks due to late or shallow saw cutting by using DCP. The longitudinal cracks were reported shortly after the construction in early 1990. The cracks were measured in 1995 and were approximately 6−13 mm wide at the time. Some cracks measured in 2005 showed width as much as 57 mm. The DCP were conducted at two locations with different crack width: 3 mm and 57 mm. It was found that the base stiffness (393 MPa) at the location with the 3 mm was twice more than base stiffness (186 MPa) at the 57 mm crack width location (Chen and Won, 2007). Thus concluded that with good base support (393 MPa), the irregular cracks that occur at an early age (due to late or shallow saw cutting of longitudinal joints) will not continue to widen, even after decades of truck trafficking.

Chen et al. (2009) used Dynamic cone penetration (DCP) to determine the base and subgrade support condition of a jointed concrete pavement. DCP tests were performed at twelve locations on the slabs with and without visible settlement. The CBR and the base moduli were calculated from the penetration rate in accord with pervious study (Chen et al., 2005). A penetration rate of 2.5 mm or less per blow represents a good aggregate base, with a California bearing ratio (CBR) of approximately 100 and a modulus of approximately 345 MPa (50 ksi). A penetration rate that exceeds 38 mm per blow indicates that the subgrade foundation support is very weak, with a CBR value of 5 and modulus of 48 MPa (7 ksi). The test at the locations gave average base moduli of 194 MPa. This implied that the slab supports were weak; however, a little difference in base modulus values from areas with and without slab settlement was noticed. Thus concluded that the weak slab support was not the primary cause for slab settlement, even though, it could have accelerated the deterioration (Chen et al., 2009). Some causes of the settlement problems such as voids under the slabs created by poor LTE and pumping along the longitudinal construction joint were also stated.

CHAPTER III

METHODOLOGY

The data acquisition equipment and procedures used for this project are described in this chapter. This includes a description of the systems which are used and how these instruments are utilized to obtain measurements regarding the pavement.

3.1 Profile Monitoring

To monitor the elevation of the road through a timeline, surveying control points are marked at the both edges and the centre of the pavement at 2.5 m interval. Figure 3.1 show the layout of the survey control points.

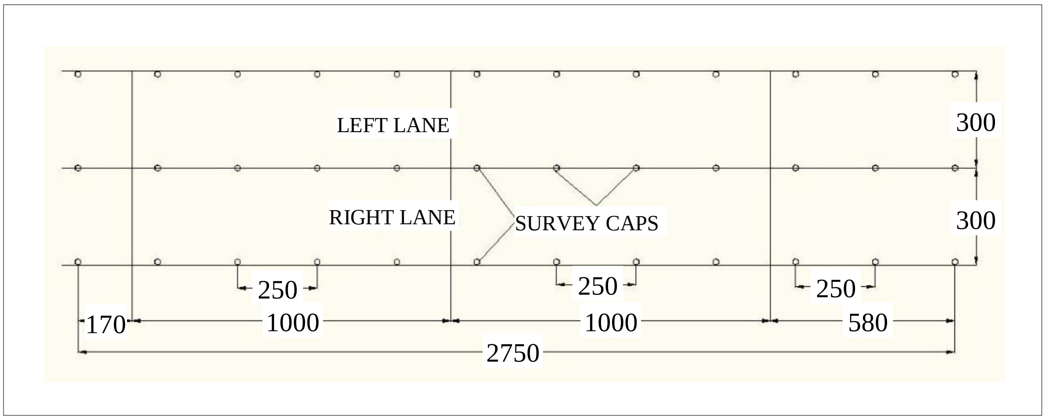

Figure 3.1 Layout of survey marks at Sikkhawithee 1(a) Road.

The profile surveying was conducted by a dumpy level surveying equipment and control point elevation measurements were made before and after the slab jacking. Additional measurements are taken at one, two, four and six months after the polyurethane injection.

The elevation study would present differential settlement at the pavement, if any. The data collected are computed to derive differential settle during the different time intervals. Thus, the data are analyzed to know where then maximum settlement occurred.

3.2 Distress Survey Testing

It was reported in Chen et al. (2008) that new cracks appeared in some sections of slab where they previously received polyurethane foam injection. Relying on visual observation, the polyurethane foam injected pavement sections were quantified for defects especially cracks. These were carried out in accordance with the ASTM D6433-03 “Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys”. The distresses observed at the site were categorized using distress symbols and recorded in the distress map. According to ASTM D6433-03, all the distresses observed at the sites were assigned for severity level to show a clear picture of the distress. The distress survey testing would give a clear indication of the new distresses appeared on the pavements, thus briefly showing the effectiveness of the polyurethane foam injection.

3.3 Concrete Core Removal Testing

The purpose of the coring at the pavement was to verify the pavement layer thickness and to look for pavement deterioration. It also served as an opening on the pavement for conducting Dynamic Cone Penetrometer (DCP) test. Three to five cores were drilled at each site by a 10 cm diameter wet core drilling machine. The potential locations of the cores were at sections where more differential settlement has occurred which were determined by the profile leveling survey during the months of observation. Other locations for the core were at the new crack areas and at the traverse joint to see the crack and dowel bar conditions respectively.

The core holes and the extracted cores related an idea about the polyurethane foam distribution and its thickness beneath the pavement. It also verified the diameter of the dowel bars and its spacing that was given by Micro Covermeter, a Non-destructive testing to locate the steel in concrete. In addition, the subgrade and the pavement conditions were reviewed.

3.4 Dynamic Cone Penetrometer (DCP) Testing

Dynamic Cone Penetrometer (DCP) test was used to measure the soil resistance to penetration similarly to field CBR but within less time (Shahin, 1994). The DCP tests were conducted at the core locations where the soil beneath the pavement was already exposed. An 8-kg DCP dynamic cone penetrometer with an 8 kg hammer (Figure 3.2) was used to assess the in situ strength of undisturbed soil materials and the test method followed was as per ASTM 6951-03. The DCP consists of a 15.8 mm diameter steel rod with a steel cone attached to one end. The tip of the cone has an inclined angle of 60 degrees and a diameter at the base of 20 mm. The cone was driven into the pavement layers being tested by dropping 8 kg sliding hammer from 575 mm fixed height. The total penetration for a given number of blows were measured and recorded in mm/blow, which was used to describe stiffness, estimate an in situ California bearing ratio (CBR) strength by using an appropriate correlation (ASTM, 2003; Shahin, 1994).

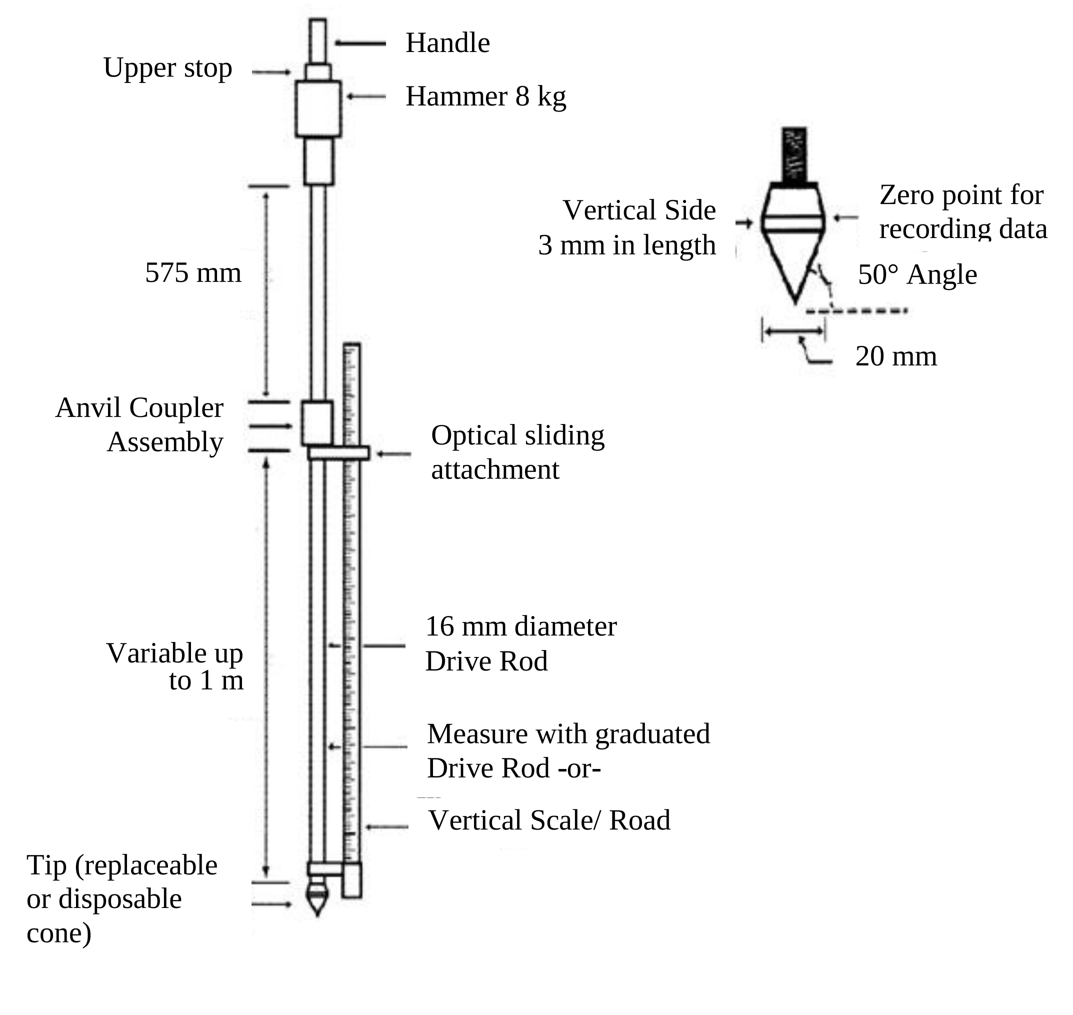

Figure 3.2 The Dynamic Cone Penetrometer (modified from ASTM D6951).

The rate of penetration is an indication of the support condition under the concrete slab. For example, a penetration rate of 2.5 mm or less per blow represents a top class granular base with a California bearing ratio (CBR) value of approximately 100 and modulus of approximately 345 MPa (50 ksi) when the following equations are used (Chen et al., 2005):

$$\text{Log CBR} = 2.465 - 1.12\ (\log \text{PR}) \text{ or CBR} = 292/\text{PR}^{1.12} \quad (3.1)$$

$$E\ (\text{psi}) = 2{,}550\text{CBR}^{0.64} \text{ or } E\ (\text{MPa}) = 17.58\text{CBR}^{0.64} \quad (3.2)$$

where CB is the California bearing ratio and PR is the DCP’s penetration rate through the layer (mm/blow). The equation (3.1) was originated by the U.S Army Corps of Engineers and the equation (3.2) from Powell et al. (1984).

Dynamic cone penetrometer showed the blow counts against the penetration data from which penetration index was calculated. The penetration index (PI) is defined as the difference in between each blow and the one preceding it. This penetration index showed the uniformity of subgrade soil and the difference of CBR at each site.

3.5 Pavement Condition Index (PCI)

Calculation of the Pavement condition Index (PCI) were carried out using the data collected during distress survey by adding up the total quantity of each distress type at each severity level and recording the data under the total severities section of the data collection form. Each total quantity were divided by the total area of the sample unit and multiplied by 100 to obtain a percent density of each distress type and severity. Percent density values and level of severity were used to generate deduct points from deduct value curves (ARMY, 1982; ASTM, 2004). Using the deduct value method; pavements are ranked on a 100 point index as shown in Figure 3.3.

The procedure states that if the PCC slabs have a joint spacing larger than 8 m, as in the case at the site locations, then the slabs must be further divided into imaginary slabs which are less than or equal to 8 m where imaginary joints are considered to be in perfect condition (ASTM, 2004). Since the profile survey was marked every 2.5 m, the slabs were divided into slabs of 7.5 m for PCI testing.

PCI	RATING
100	EXCELLENT
85	VERY GOOD
70	GOOD
55	FAIR
40	POOR
25	VERY POOR
10 0	FAILED

Figure 3.3 PCI scaling and condition rating (ARMY, 1982).

3.6 Classification of Soils

Even though the soil classification solely cannot yield sufficient information to identify the behavior of subgrade soil, however, it can group the soil type and estimate the behavior of the soil (Yoder and Witczak, 1975). Samples from the core holes and from the pavement area were collected to classify the soil type. Unified Soil Classification system (USCS) was used for classifying the soil samples. This helped in determining the group of soil at the sites and studying the nature of the soil at the subgrade.

Soils classification was done based on the characteristics of the soil that indicated how it would behave as a construction material. In the USCS, all soils are placed into one of three major categories. They are Coarse-grained, Fine-grained and Highly organic which were divided into the major soil categories by letter symbols, such as S for sand, G for gravel, M for silt and C for clay. A soil that meets the criteria for sandy clay would be designated (SC). There are the cases of borderline soils that cannot be classified by a single dual symbol, such as GM for silty gravel. These soils may require four letters to fully describe them. For example, (SM-SC) describes sand that contains appreciable amounts of silt and clay.

CHAPTER IV

RESULTS AND DISCUSSIONS

4.1 Introduction

The polyurethane foam injection beneath the distressed pavements in Suranaree University of Technology was executed in July 2009. A brief procedure of the polyurethane foam injection is explained followed by the data presentation and the results of the tests conducted on each site location.

4.2 Polyurethane Foam Injection Procedure as in SUT

The procedure of injecting polyurethane under SUT distressed pavements is as follow. The injection was carried out at night to minimize the disruption to the flow of traffic. An initial profile of the roadway was made to determine which pavement segments needed to be raised. Injection holes of 16 mm diameter were drilled through the pavement and into the soil below. Figure 4.1 shows the grid of holes drilled at one lane of the pavement, however, drill holes were also made at random once all the initial holes are injected with polyurethane. Since the roadway is a two way lane, the injection was done lane wise so that the traffic flow was minimal interrupted.

A two-component system is used to create the polyurethane. One component consists of a mixture of a polyhydroxy compound, catalysts, and water; the second component is an isocyanate compound. The two components are injected simultaneously through the drilled holes with air pressure.

The chemicals start reacting immediately at the injector nozzle forming rigid polyurethane foam after spreading beneath the pavement. The volume of the foam was several times that of the reactants; consequently, the reaction produces an expansive force that lifts the slab. After the injection, the amounts of polyurethane reactants used in each site locations were as given in the Table 4.1. A string and laser level was used to monitor elevations during the process. Two workers performed the injection process to minimize the risk of cracking and the amount of rise was controlled by the rate at which the reactants were injected through the holes. Multiple lifts were used to reach the desired profile level if necessary. The foam material reaches 90% of its maximum compressive strength after 15 minutes (Uretek USA, 1998). After each hole was injected, any excess foam was removed from the holes which then were sealed with cementitious grout.

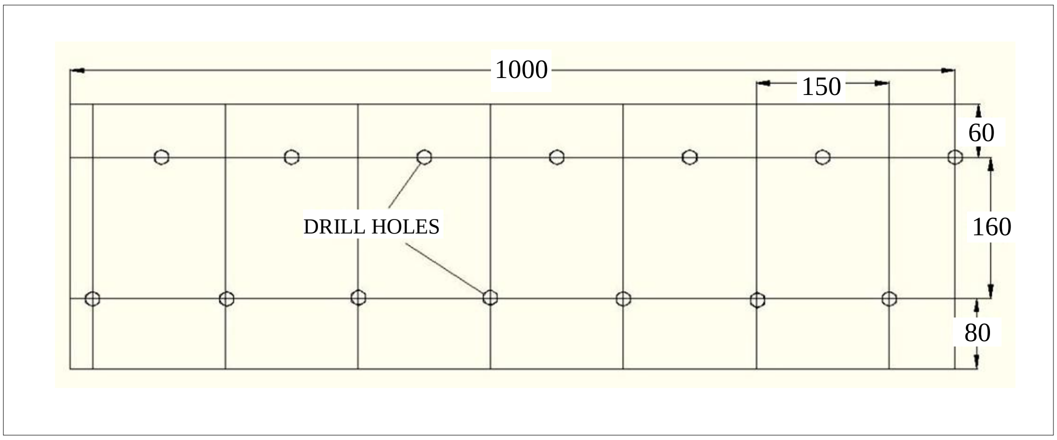

Figure 4.1 Layout of drill holes (dimension in cm).

Figure 4.2 (A) Drill hole for polyurethane injection. (B) Layout of drill hole at site. (C) Voids beneath the pavement as seen through a Flexible Borescope.

The polyurethane foam expanded into voids in the subgrade, improving the stability of the subgrade and increasing the capacity of the subgrade to withstand weight. In addition, the instable subgrade caused by water infiltration would be reduced because of closed cellular structure of foam. Due to the fact that the foam has lower density in comparison to grout or mud, the polyurethane would cause less weight-induced settling (Uretek USA, 2005).

Table 4.1 Amount of polyurethane reactants used in each site location.

Name of the Road	Length of the distressed section (m)	Date of the repair	Amount of polyurethane reactant used (kg)
Withayawithee 2 Road	55	14th July 2009	805.17 kg
Withayawithee 3 Road	30	20th July 2009	778.79 kg
Sikkhawithee 1(a) Road	27.5	12th July 2009	592.67 kg
Sikkhawithee 1(b) Road	12.5	19th July 2009	149.9 kg

4.3 Profile Monitoring Testing

4.3.1 Withayawithee 2 Road

Withayawithee 2 Road, being the longest section to be injected with polyurethane within the university, has experienced the maximum distresses and poor ride quality. The road section is a two lane having 6 slabs on each lane and the slabs sizes are non-uniform. On July 20th 2009, this section was injected with polyurethane foam and it was noticed that the road section was lifted and aligned to a desired level. The data acquired from experiments conducted on this site were presented and analyzed in this section.

On July 19th 2009, a day before the polyurethane foam injection, profile level of the road section was carried out and another level was taken just after the injection. It was found that pavement was lifted by the polyurethane foam by a maximum lift of 114 mm at the middle of the lane through 45 m distance from the beginning of the section (i.e. station 0.0) as shown in Figure 4.3. For left and right lane, there was a maximum lift of 79 mm and 99 mm respectively at station 47.5 m. In addition, settlements of pavement were also noticed at certain areas just next to sections lifted by polyurethane foam. The maximum settlement for the right lane was 66 mm at station 37.5 m, 44 mm for the middle at station 30 m and 42 mm for left lane at station 37.5 m. The settlement might be because of some hard support acting as pivot which transfers lifts at one section to settlement at another end.

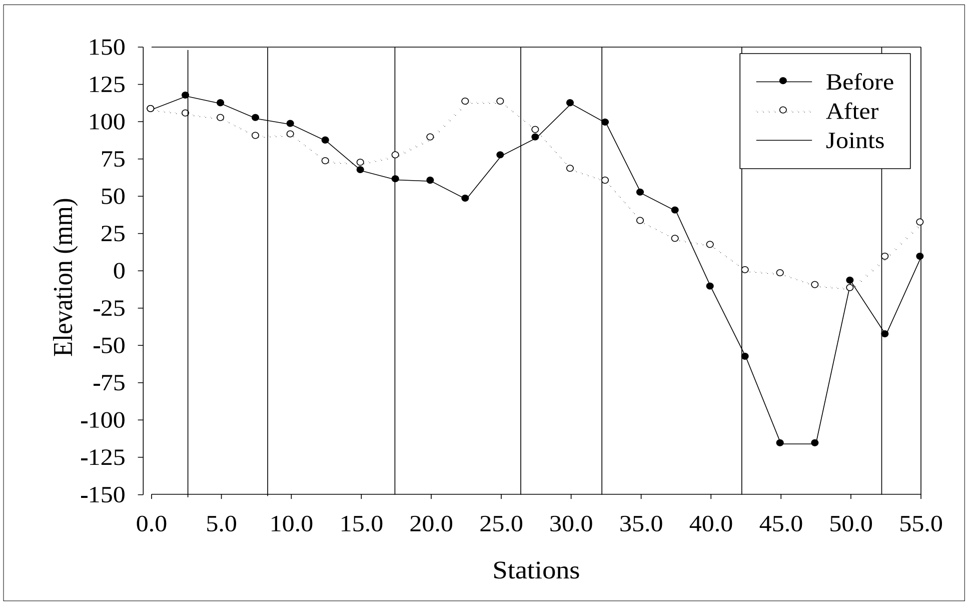

Figure 4.3 Profile leveling before and after polyurethane foam injection on the middle of lanes of Withayawithee 2 Road.

The pavement profile measurements were carried out multiple times during the period of 6 months and the conclusion drawn from the data were inconsistent. There were minimal settlements of 2 mm at the left lane and middle of lanes. As for the right lane, large amount of settlement was observed with maximum of 52 mm at station 47.5 m as shown in Figure 4.4. The settlements were more at the right lane because loaded trucks for ongoing construction passed through the right lane while the return trip of empty trucks were through the left lane. Figures 4.5 and 4.6 showed the differential settlement of the road section during the study period. In Figure 4.5, a large settlement was caused by disturbance to the station marked. The additional graphs from this site location were attached in Appendix (A).

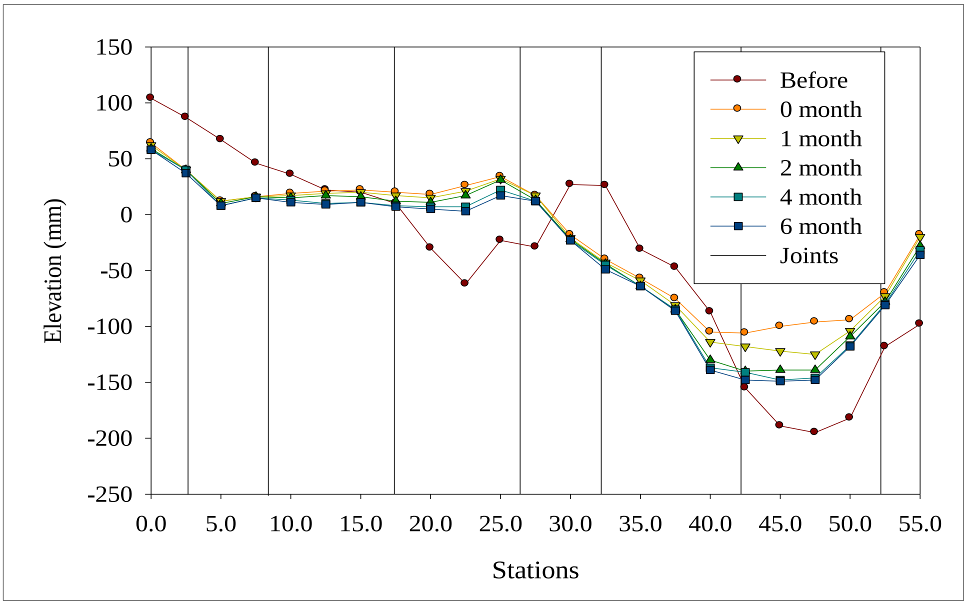

Figure 4.4 Settlements on right lane of Withayawithee 2 Road during the study period.

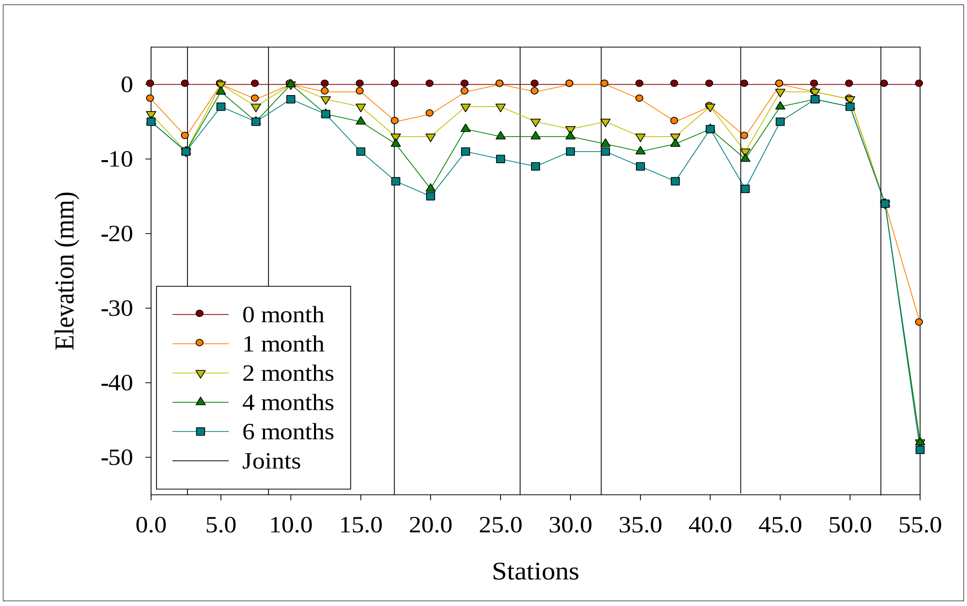

Figure 4.5 Differential settlement on left lane of Withayawithee 2 Road during the study period.

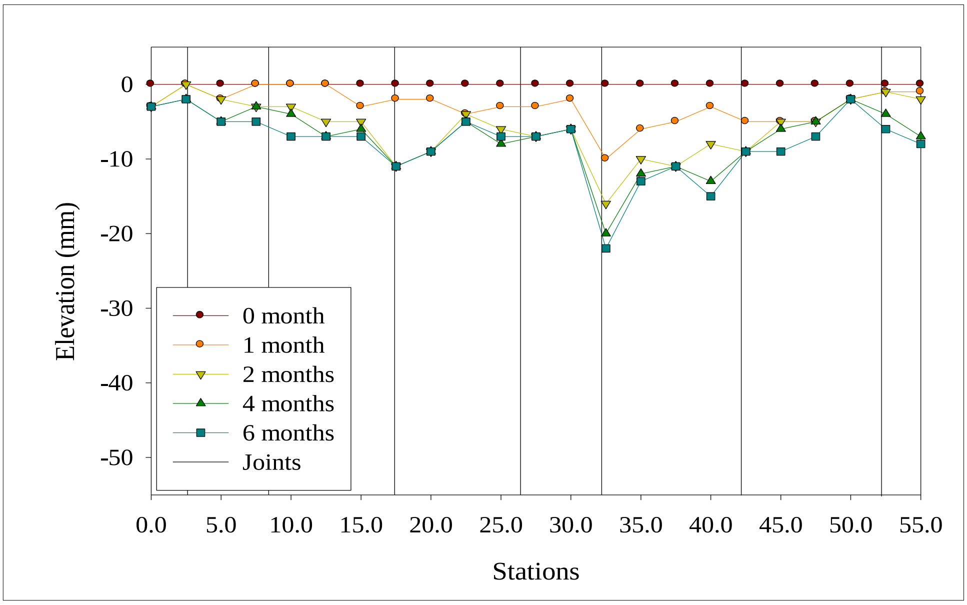

Figure 4.6 Differential settlement at the middle of lanes of Withayawithee 2 Road during the study period.

4.3.2 Withayawithee 3 Road

The distressed section on Withayawithee 3 Road, on July 14th 2009, was injected with polyurethane foam to rehabilitate the pavement and to improve the ride quality. The polyurethane foam lifted the pavement and there was improvement to the ride quality. To measure the performance of polyurethane foam injection, before and after the injection, the profile level was measured. Figure 4.7 shows the lift provided to pavement by the injection.

During the six months of observations of the pavement leveling, a maximum settlement of 12 mm was found at the middle of lanes and right lane at station 22.5 m and station 10.0 m respectively, whereas 10 mm settlement was seen at station 25.0 m on right lane. Minimum of 2 mm settlement was observed in all the stations except on one station at the left lane as shown in Figure 4.9.

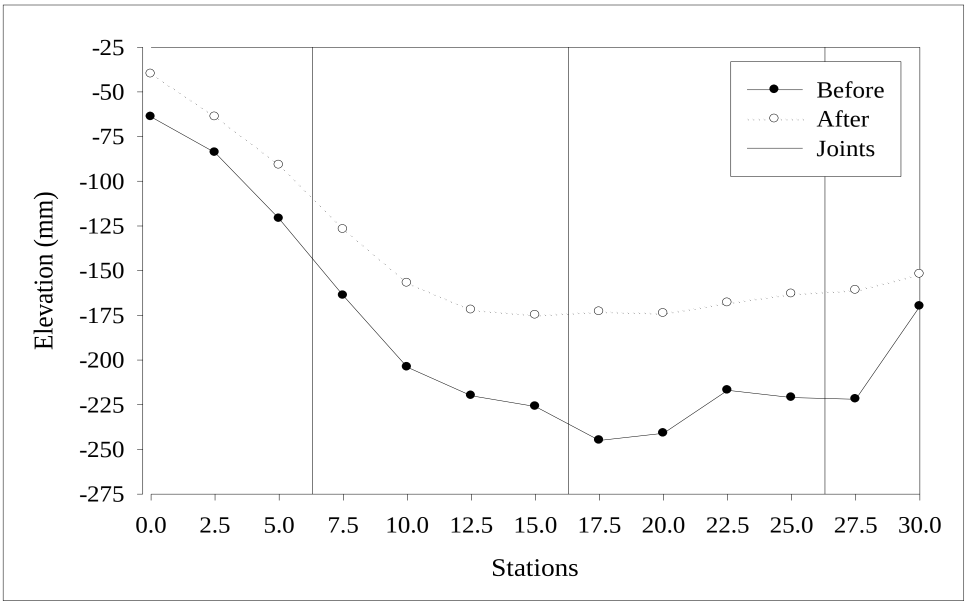

Figure 4.7 Profile leveling before and after polyurethane foam injection at right lane of Withayawithee 3 Road.

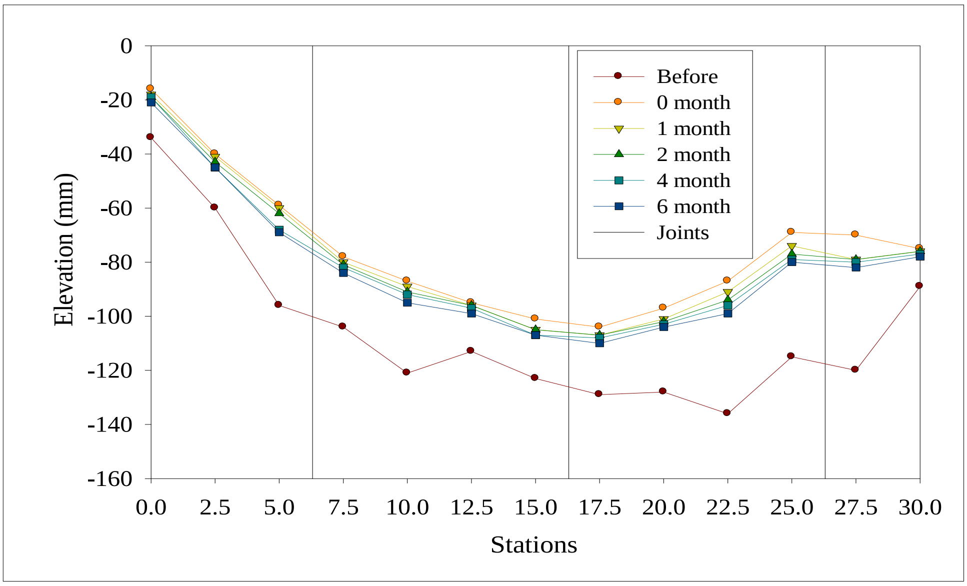

Figure 4.8 Settlements at the middle of lanes of Withayawithee 3 Road during the study period.

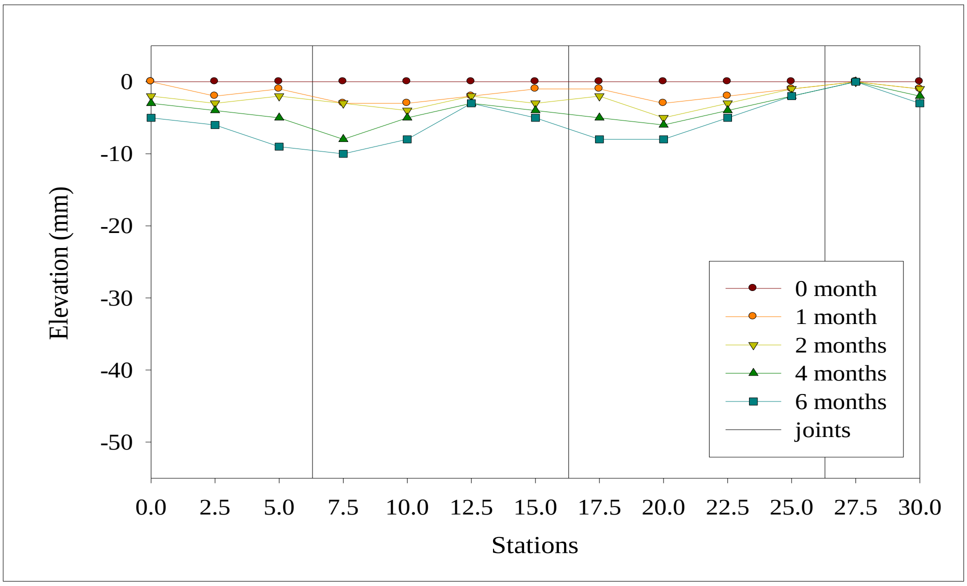

Figure 4.9 Differential settlement on the left lane of Withayawithee 3 Road during the study period.

4.3.3 Sikkhawithee 1(a) Road

On July 12th 2009, the first road in Suranaree University to experience polyurethane foam injection was Sikkhawithee 1(a) Road. To evaluate the settlement of the road, surveys were conducted and the results were given in the following paragraphs. The survey before/after the injection shows that the pavement has been lifted and aligned to a desired level as in Figure 4.10.

Figure 4.11 and 4.12 show the settlement during the study period as profile level and differential settlement. Settlements at the middle of lanes show maximum of 13 mm at station 12.5 m. It was also observed that all the stations suffer at least 2 mm of settlement.

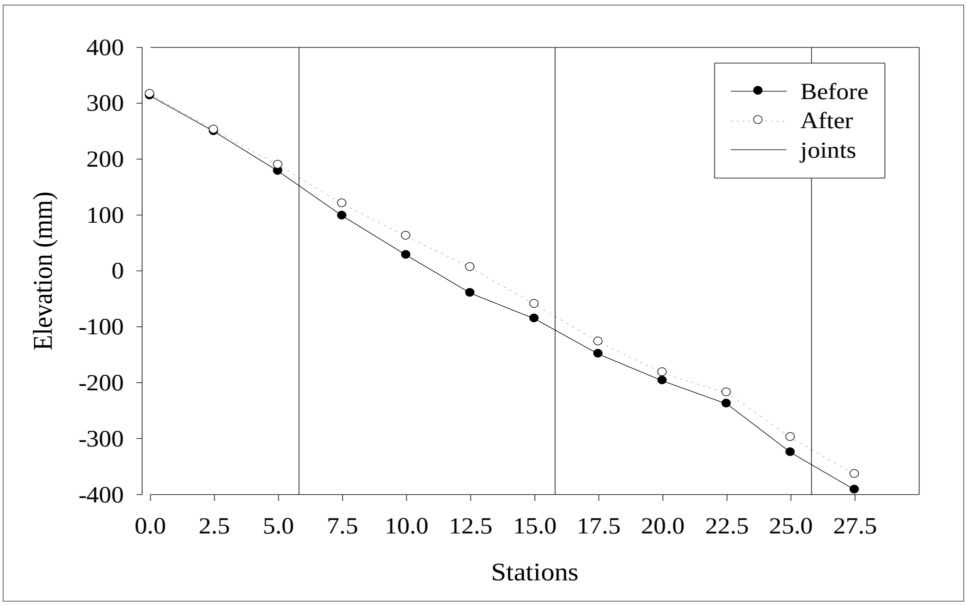

Figure 4.10 Profile leveling before and after polyurethane foam injection on the right lane of Sikkhawithee 1(a) Road.

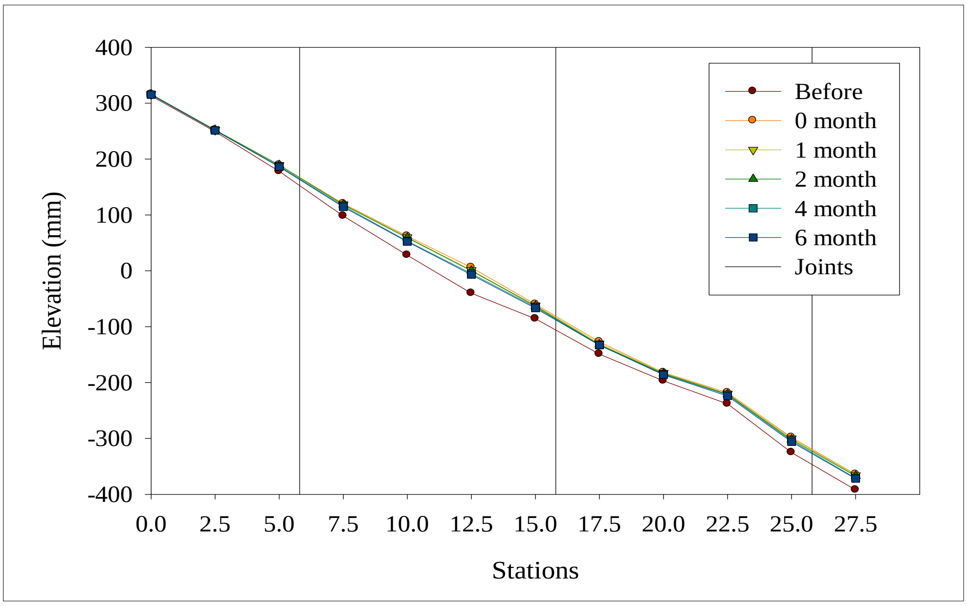

Figure 4.11 Settlements on the right lane of Sikkhawithee 1(a) Road during the study period.

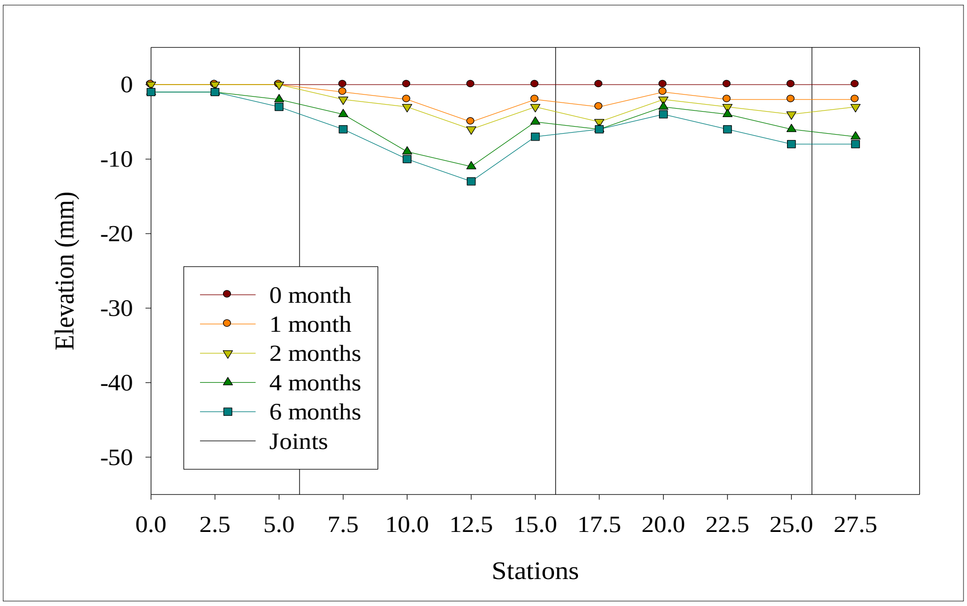

Figure 4.12 Differential settlement on the middle of lanes of Sikkhawithee 1(a) Road during the study period.

4.3.4 Sikkhawithee 1(b) Road

Though this was the shortest road segment amongst the road injected with polyurethane foam to repair the distress, road stitching was also carried out at some portion of the section where polyurethane was injected. The tests were conducted and results were given in the following.

There was a maximum raise of 44 mm on the middle of the lanes at station 7.5 m and a 42 mm raise of pavement on right lane at station 5.0 m while on left lane there was 39 mm raise at station 7.5 m. The Figure 4.13 shows the raise in the pavement and portions of settlements during the polyurethane foam injection. This might be due to the same reason as stated at Withayawithee 2 Road.

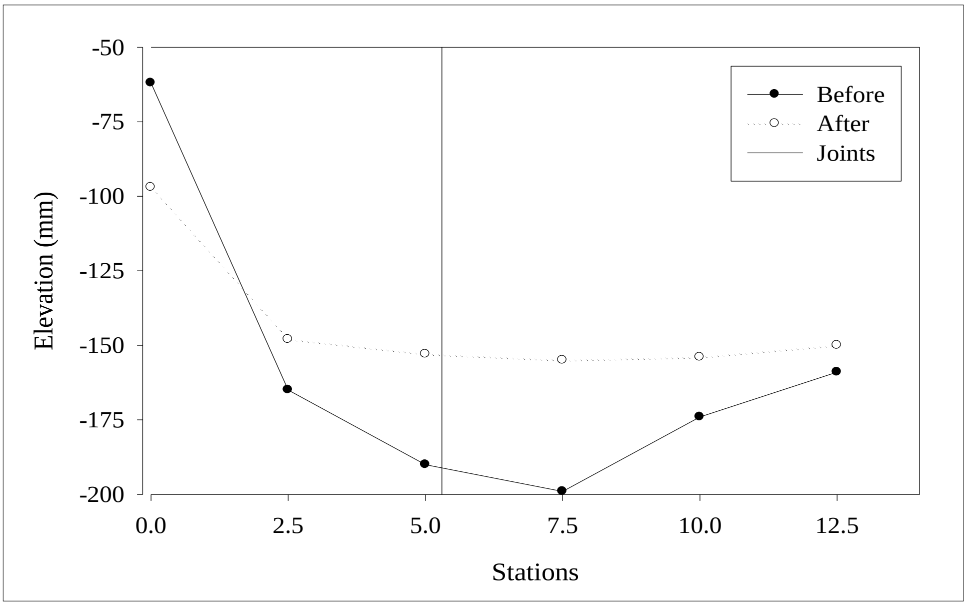

Figure 4.13 Profile leveling before and after polyurethane foam injection at the middle of lanes of Sikkhawithee 1(b) Road.

The settlement during the study period was varying from 3 mm to 9 mm with highest on the middle of lane at station 12.5 m followed by 8 mm settlement on the left lane at station 5.0 m. There was a sudden lift in the pavement between the 0^{th} month and the first month and this was due to the pavement repair stitching. However, after the first month a gradual minimal settlement was observed. Figure 4.14 shows the disruption in the graph by the pavement stitching along with the settlements at different stations while Figure 4.15 shows a differential settlement on the right lane of the road.

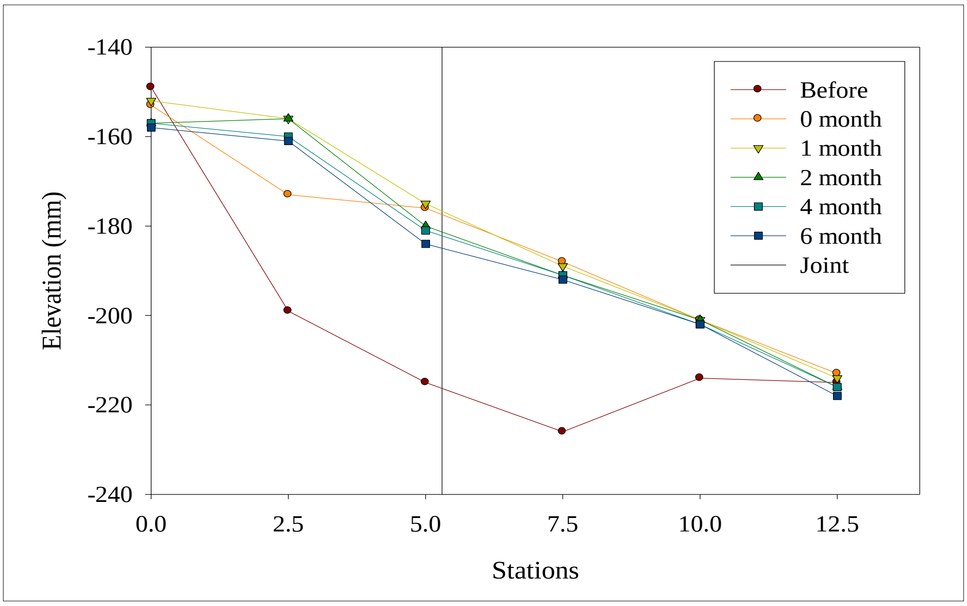

Figure 4.14 Settlements on the left lane of Sikkhawithee 1(b) Road during the study period.

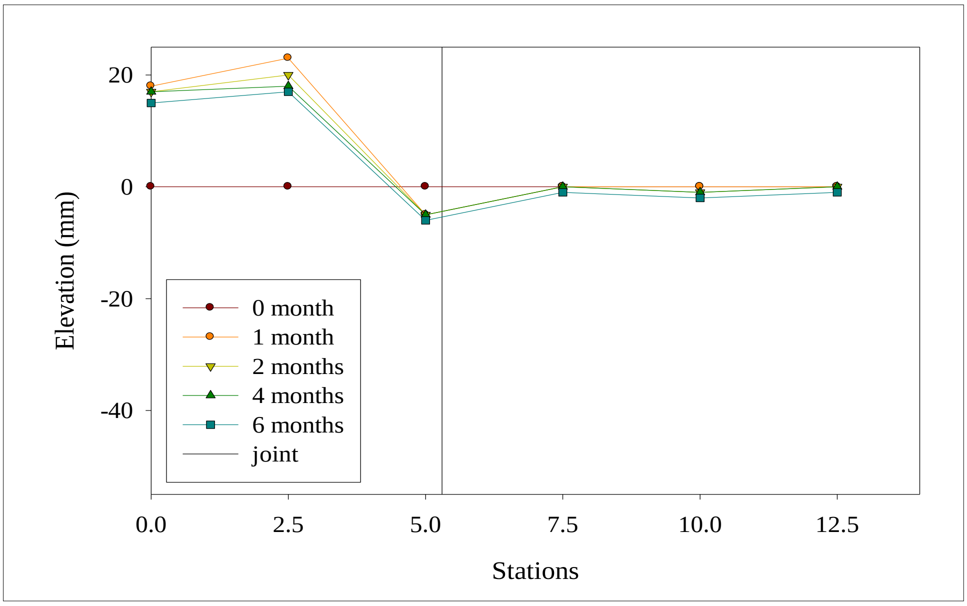

Figure 4.15 Differential settlement on the right lane of Sikkhawithee 1(b) Road during the study period.

4.4 Distress Survey Testing

It was reported in Chen et al. (2008) that new cracks appeared in some section of slab where it had previously received polyurethane foam injection. Relying on visual observation, the polyurethane foam injected pavement sections were quantified for defects, especially cracks. As mentioned in the previous chapter these were done in accordance with ASTM D6433-03 “Standard Practice for Roads and Parking Lots Pavement Condition Index Surveys”.

Within the four site locations surveyed there was evidence of the following distresses: corner breaks, divided slab, transverse and longitudinal cracking, faulting, lane/ shoulder drop-offs, spalling, and pumping. The most prominent distress deduced was the traverse cracking which could be mainly caused by a combination of repeated traffic loading and insufficient support for the slabs.

The distresses were categorized according to the description in the previous chapter into low, moderate and high severity levels and recorded in the appropriate distress map as seen in Figure 4.16. Lane/shoulder drop-offs were measured but not recorded on the distress maps. The entire set of the distress maps for the project is contained in Appendix (B).

The settlements seen by profile surveying and the cracks observed during the distress survey were merged together to check the correlation and it was observed that cracking were more at area with more settlements. Further it was also seen that settlements mainly occurred near cracks; new cracks were marked in red and others in black. It was clear from Figure 4.16, that settlements and crack show a close relation.

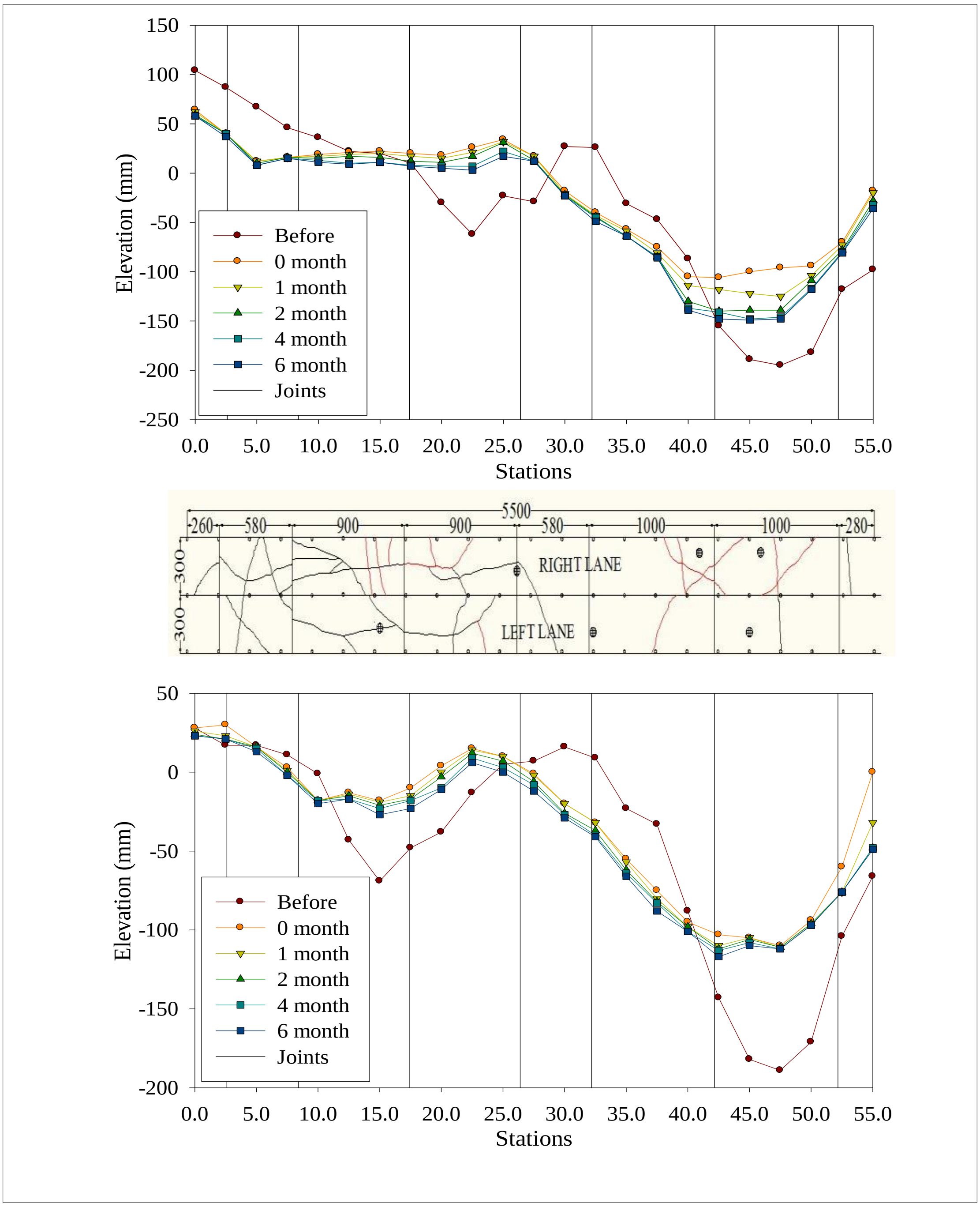

Figure 4.16 Correlating settlement and cracking on the pavement on the right lane (top) and left (bottom) of Withayawithee 2 Road.

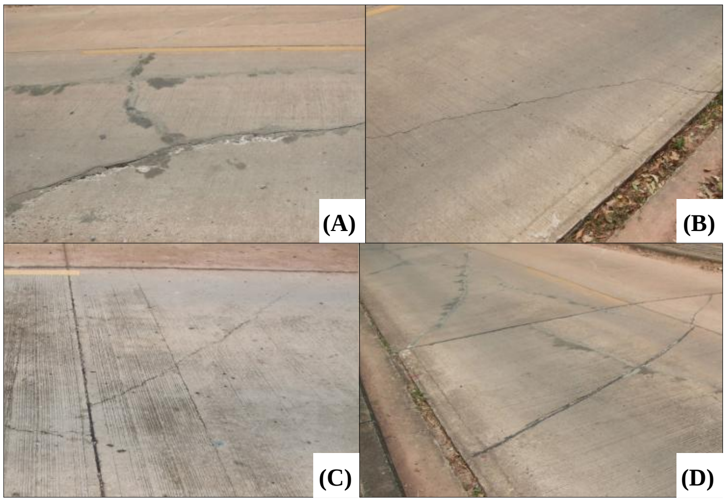

Figure 4.17 (A) Old cracks to creating divided slab with spalling. (B) New cracked developed at the pavement. (C) Corner break. (D) Old repaired cracks leading to linear cracking and divided slab distresses.

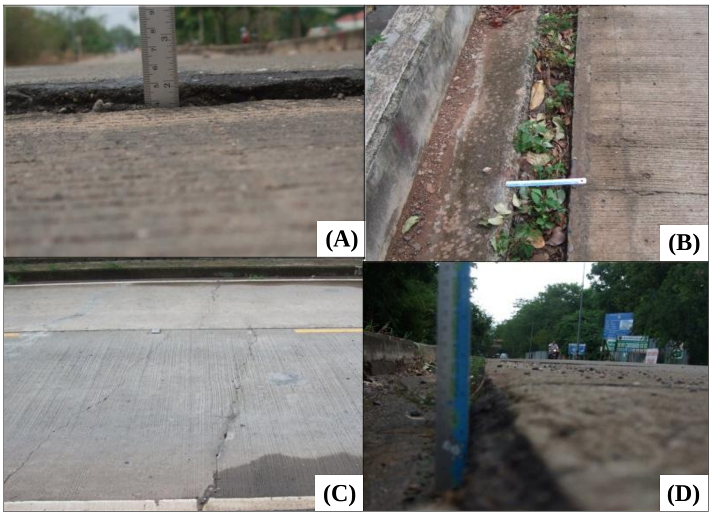

Figure 4.18 (A) Faulting at the traverse joint. (B) Space formed due to lane/shoulder drop-offs. (C) Linear cracks. (D) Lane/shoulder drop-offs with faulting.

Since the entire areas were being tested visually, the distress surveys were quite effective for developing a tangible sense of the pavement condition. Even while driving over the sections; it was apparent initially that many distresses were evident along with considerable faulting. The results from the distress survey served to further corroborate the deduced hypothesis.

From the Figures 4.19 and 4.20, it is clear that Withayawithee 2 Road suffered mainly from divided slab while Withayawithee 3 Road and Sikkhawithee 1(a) Road suffered from linear cracking. All the sites contain corner breaks except at Sikkhawithee 1(b) Road with equal number of slabs suffering divided slab and linear cracking.

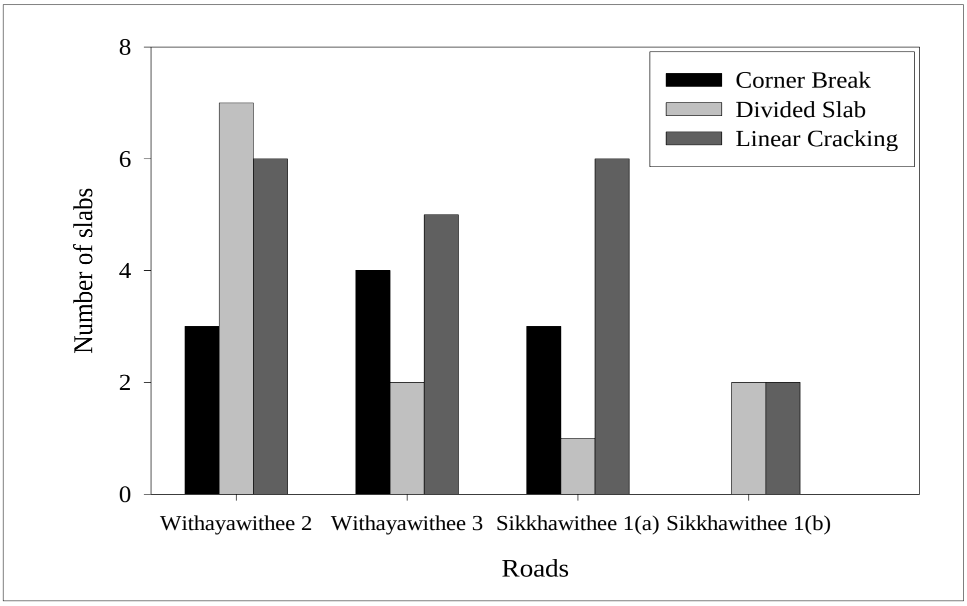

Figure 4.19 Number of slab affected by distresses at each site.

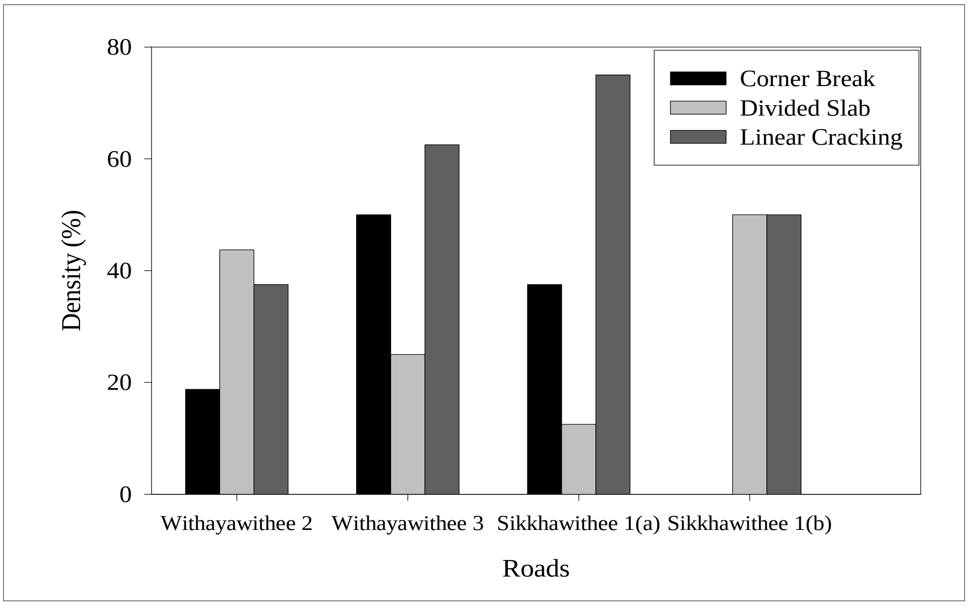

Figure 4.20 Density of distresses at each site.

4.5 Analysis on Core Removal Testing

Several 10.0 cm diameter concrete cores were extracted from the study sites for the purposes of conducting DCP testing and to study the conditions of the pavement and the polyurethane foam. Three cores each from Sikkhawithee 1(a) and 1(b) road, 4 cores from Withayawithee 3 Road, and 6 cores from Withayawithee 2 Road were extracted. The layout of the core locations are shown in Figure 4.21. The number of cores extracted from each sites were estimated depending on the differential settlement and the distresses on the pavement sections. These cores were measured to determine the thickness of pavement at the coring locations and studied the location of steel reinforcements found on the cores. The cores extraction also confirmed diameters of the tie bar and the dowel bar along with the diameter of the steel reinforcement. Moreover, coring confirmed the presence of the polyurethane foam beneath the pavements.

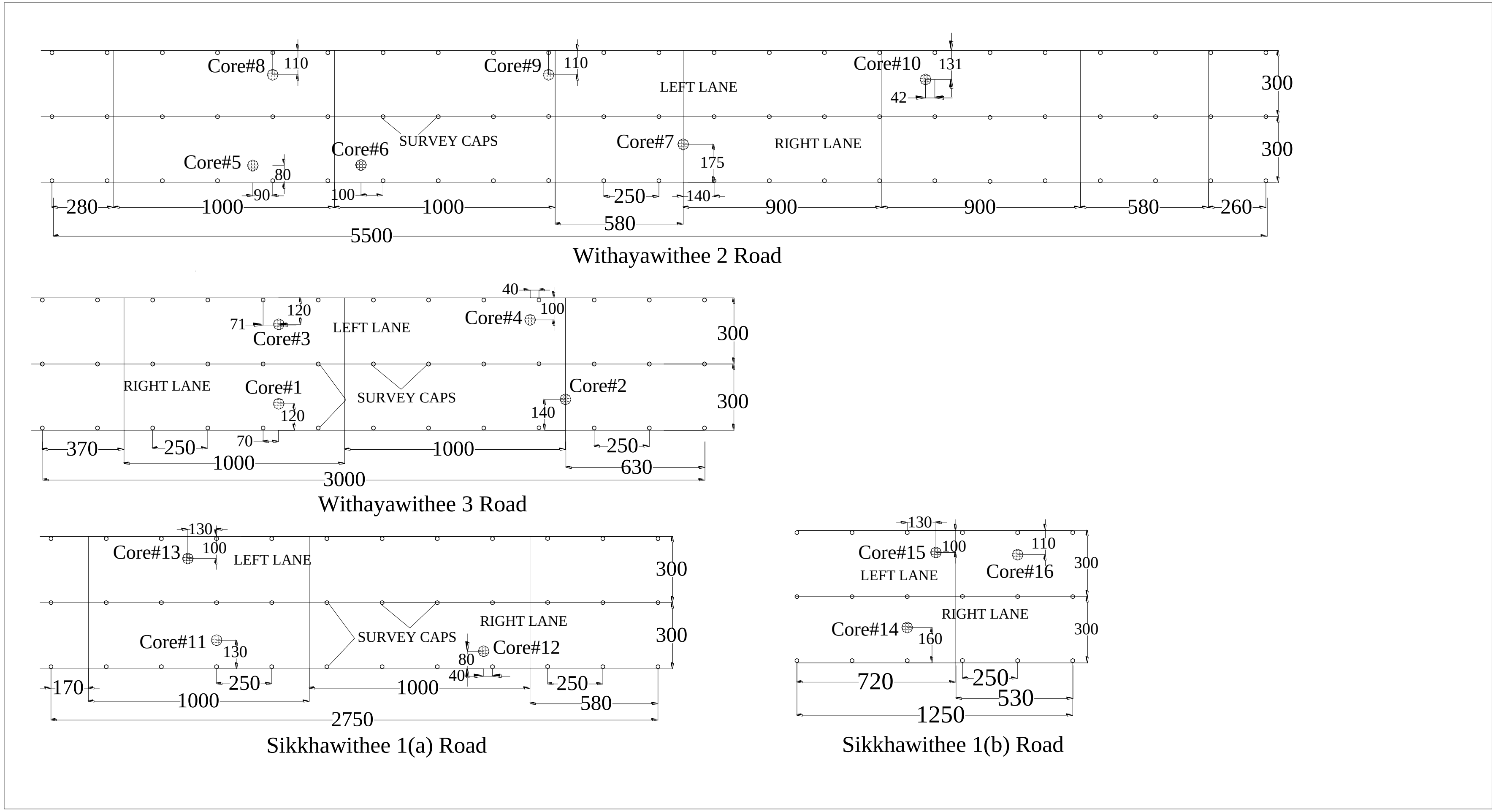

Figure 4.21 Layout of the core locations at each site locations.

The designed thickness for the existing concrete pavement was 20.0 cm and the extracted cores deviated considerably from this value. The thickness of the cores across all four sites varied from 17.5 cm to 22.0 cm, however a mean thickness 19.9331 cm was found throughout the sections. It was noted that the cores from Withayawithee 2 Road had lesser number of cores, thinner than the designed thickness while the cores from Sikkhawithee 1(a) Road had more cores thicker than the designed thickness. The cores from Withayawithee 3 Road and Sikkhawithee 1(b) Road had more core equivalent to the designed value. Figure 4.22 shows the variation of thickness for all the cores from the designed value. Table 4.2 illustrates the cores from the respective site locations and the mean core thickness of each site location. Graphs showing the mean thickness of cores at each site are given in Appendix (C).

Table 4.2 Corresponding cores for each site locations with mean thickness.

Name of the Road	Corresponding cores	Mean core thickness (cm)
Withayawithee 2 Road	Core#5, 6, 7, 8, 9, and 10	19.42
Withayawithee 3 Road	Core#1, 2, 3, and 4	20.17
Sikkhawithee 1(a) Road	Core#11, 12, and 13	21.23
Sikkhawithee 1(b) Road	Core#14, 15, and 16	19.33

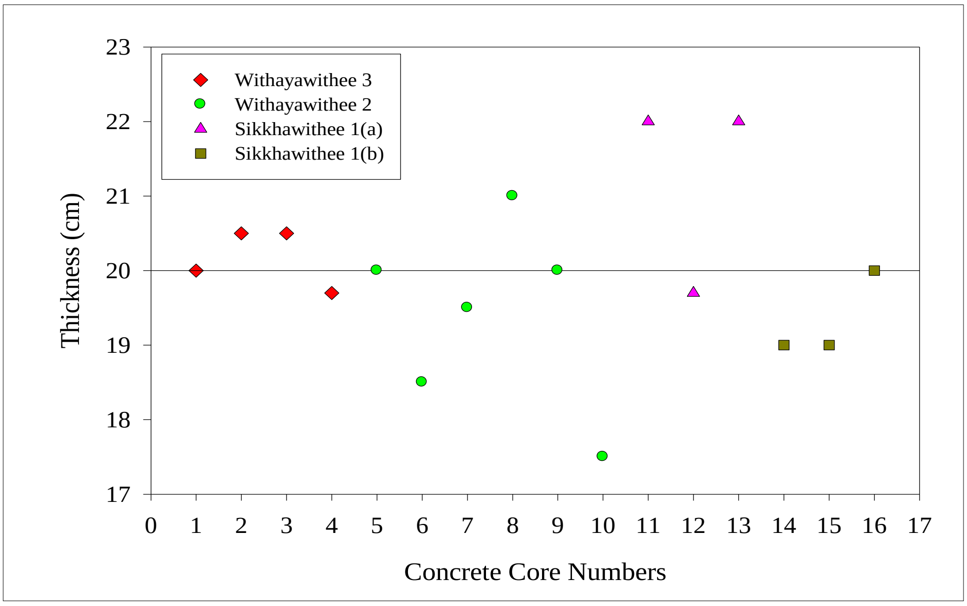

Figure 4.22 Variation of concrete core thickness.

4.5.1 Withayawithee 2 Road

Six cores were extracted from these locations and all the cores confirmed the presence of polyurethane foam of varying thickness except in Core#7, which was on a faulted traverse joint, and showed that polyurethane was unable to penetrate the joint, forming a void of 2-3 cm on one side of the fault (Figure 4.23 (B)). Core#6 and Core#10 were taken from old and new cracked area, respectively and showed that the crack was throughout the thickness of the slab (Figure 4.23 (A) and (D)).

4.5.2 Withayawithee 3 Road

At Wittayawithee 3 Road, all the cores have 3 cm to 4 cm polyurethane layer beneath the slab; however it was interesting to see that at Core#4 even though the nearest injection was 50 cm away, a void of 3-4 cm was formed below the slap surrounded by polyurethane foam as show in Figure 4.25, Figure 4.24 (C) and (D). Core#2 was taken from a traverse joint at the slab which was located using a Micro-Covermeter, therefore, it showed that the dowel bar has diameter of 25 cm as shown in Figure 24 (B). The dowel bar was in good condition. Core#1, taken from cracked area, was cracked throughout its thickness as shown in Figure 4.24 (A).

4.5.3 Sikkhawithee 1(a) Road

At this site, three cores were extracted from which two were from cracked region and all the cores showed the presence of polyurethane foam (Figure 4.26 (A)). Core#12 was cracked till the half through the thickness of the slab whereas Core#13 was cracked throughout and even the core broke while extracting (Figure 4.26 (B)).

4.5.4 Sikkhawithee 1(b) Road

All the cores extracted from this site showed presences of thin layers of polyurethane foam beneath the pavement as showed in Figure 4.26 (C).

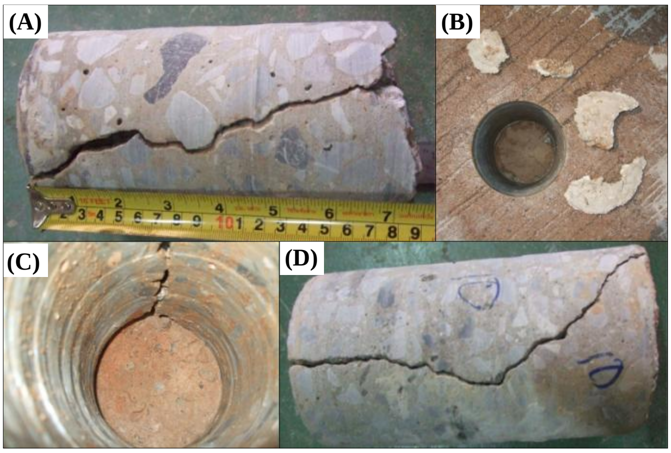

Figure 4.23 (A) and (D) Core#6 and Core#10, respectively showing crack through its depth. (B) Core#7 indicating the void on one side due to inability of polyurethane to penetrate through the fault. (C) Core#8 showing the presence of polyurethane foam.

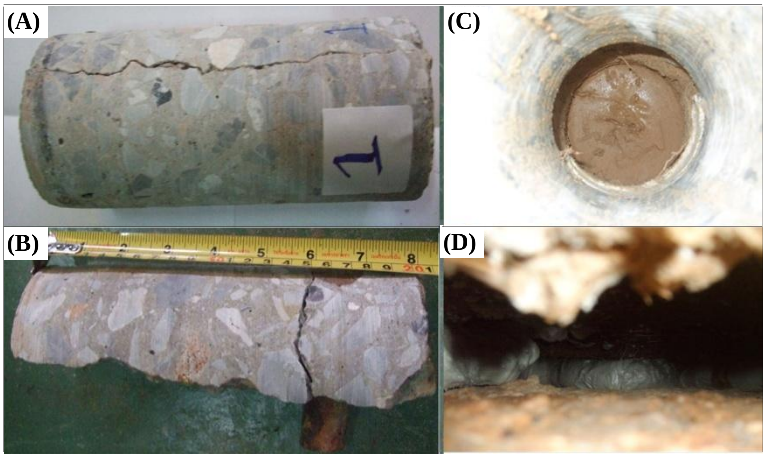

Figure 4.24 (A) Core#1 showing crack through it since it was taken out from a cracked area. (B) Dowel bar at Core#2 which was in good condition. (C) and (D) Presences of void beneath the slab which was due to its surrounding being lifted by PFI at Core #4.

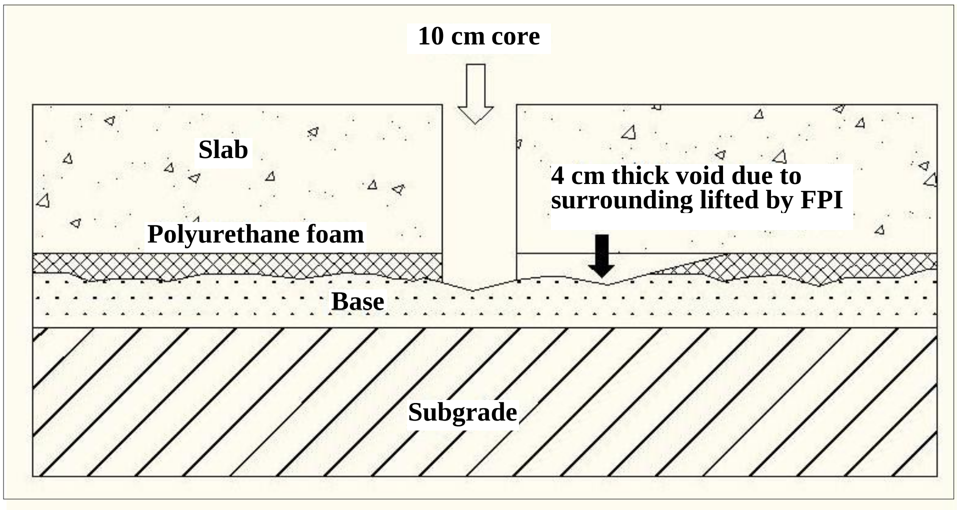

Figure 4.25 Core#4 showing the void beneath the slab which was due to surrounding being lifted by PFI.

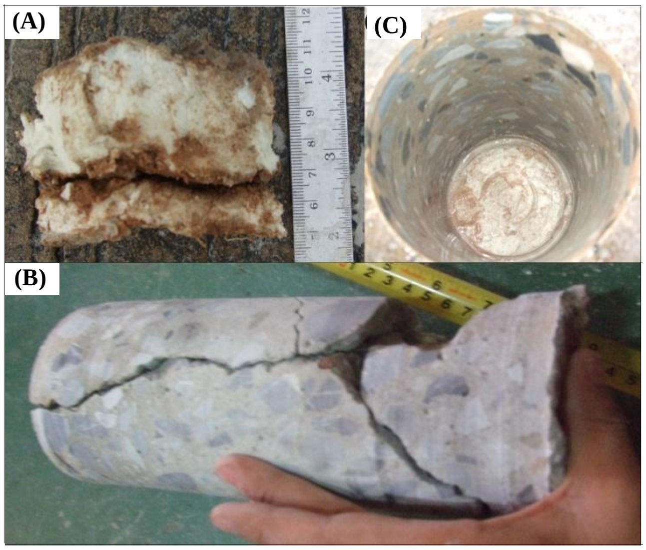

Figure 4.26 (A) Polyurethane layer thickness shown at Core#12. (B) Core#13 which was broken while extraction. (C) Core#16 showing the presence of polyurethane foam.

4.6 Dynamic Cone Penetrometer Testing

On March 20th and 21st, 2010 the subgrades of the four rehabilitated roads were examined using manual Dynamic Cone Penetrometer, as described in the previous chapter. The DCP testing was conducted on the exposed subgrade by the coring and the data of number of blows and penetrated depth for each blow were recorded. Out of 16 DCP tests couples of tests have extraneous data due to the existence of hard surface in the subgrade. The Penetration Index (PI) which indicates the support condition under the concrete slab was computed from the data recorded, and plots were constructed. The plots showed a non-uniformity of the subgrade soils, which was common throughout all section as shown in Figure 4.27 and Figure 4.28 showing Blow vs. depth plots for Core#1 at Withayawithee 3 Road and PI vs. depth for Core#9 at Withayathee 2 Road respectively.

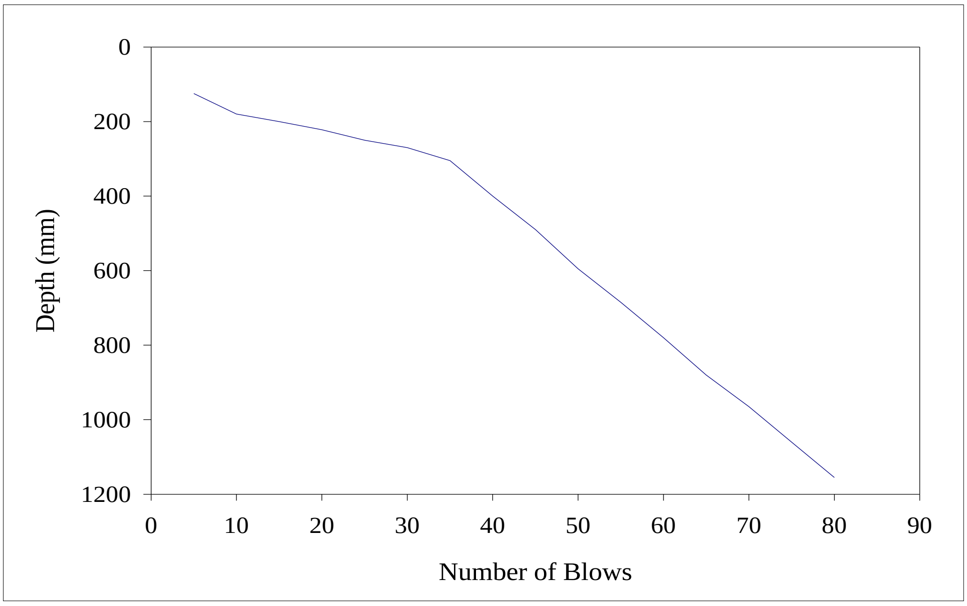

Figure 4.27 Blow vs. depth graph for Core #1 at Withayawithee 3 Road.

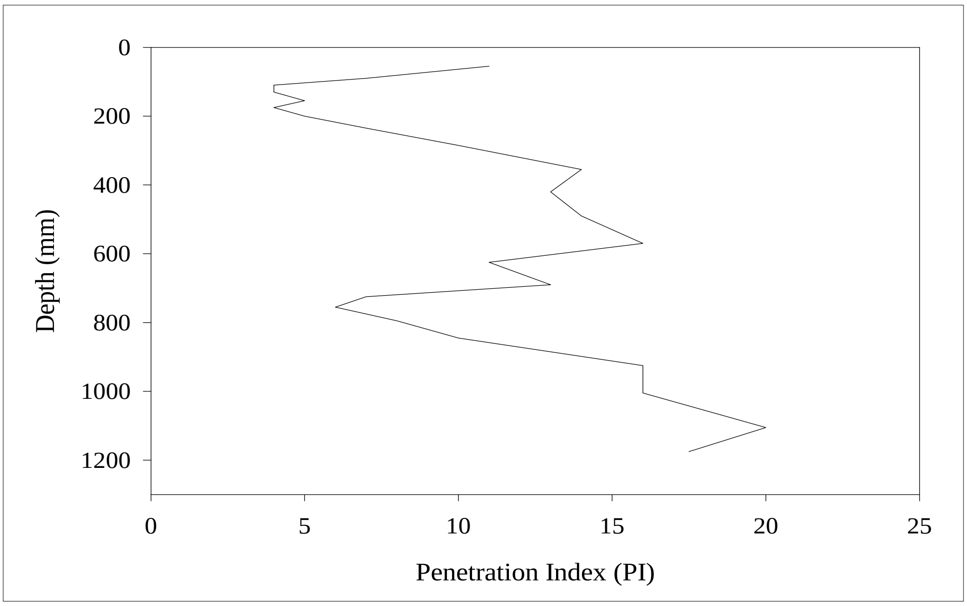

Figure 4.28 PI vs. depth graph for Core #9 at Withayawithee 2 Road.

The Data collected were used for deriving the Modulus of the subgrade by calculating the CBR using Equations 3.1 and 3.2 in the previous chapter. A penetration rate of 2.5 mm or less per blow represents a top class granular base with a California bearing ratio (CBR) value of approximately 100 and modulus of approximately 345 MPa when the above equations are used (Chen et al., 2005). In other words, a penetration rate exceeding 38 mm per blow indicates very weak foundation support, with a CBR value of 5 and modulus of 48 MPa (Chen and Won, 2007).

In addition, it was also observed that the base and subgrade at the cracked areas and noncracked areas have similar pattern of modulus. It is theorized that, with time, the weak support under the concrete slab will cause the cracks to propagate to the noncracked areas as traffic loads are applied (Chen and Won, 2007).

4.6.1 Withayawithee 2 Road

DCP tests from Withayawithee 2 Road contained some graphs displaying portions of extraneous data due to the presence of rocks in the subgrade as shown in Figure 4.29. However, it was observed that the modulus at upper portion of the subgrade was higher than the lower portion except for DCP test at Core#7 which was vice versa as shown in Figures 4.30 and 4.31. Core#7 was located at the faulted joint of the slab where more settlements were noticed. In addition, average modulus was only 98.8 MPa with modulus varying from 55 to 149 MPa at Core#7 whereas other areas have at least 120 MPa with each modulus more than 90 MPa. It might be because the subgrade at Core#7 areas was losing its stiffness at the upper portion due to scouring.

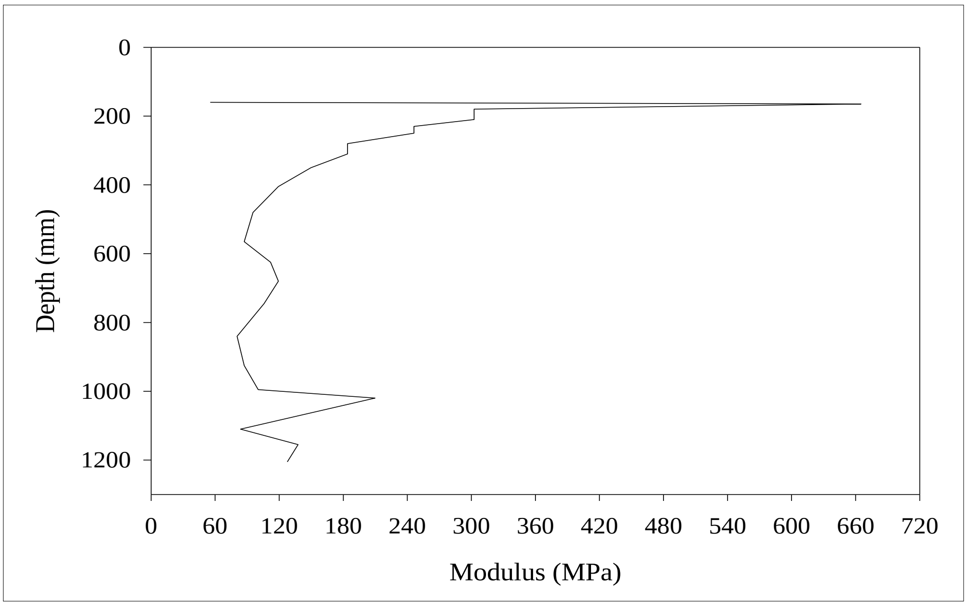

Figure 4.29 Modulus of subgrade reaction from Core#5 at Withayawithee 2 Road.

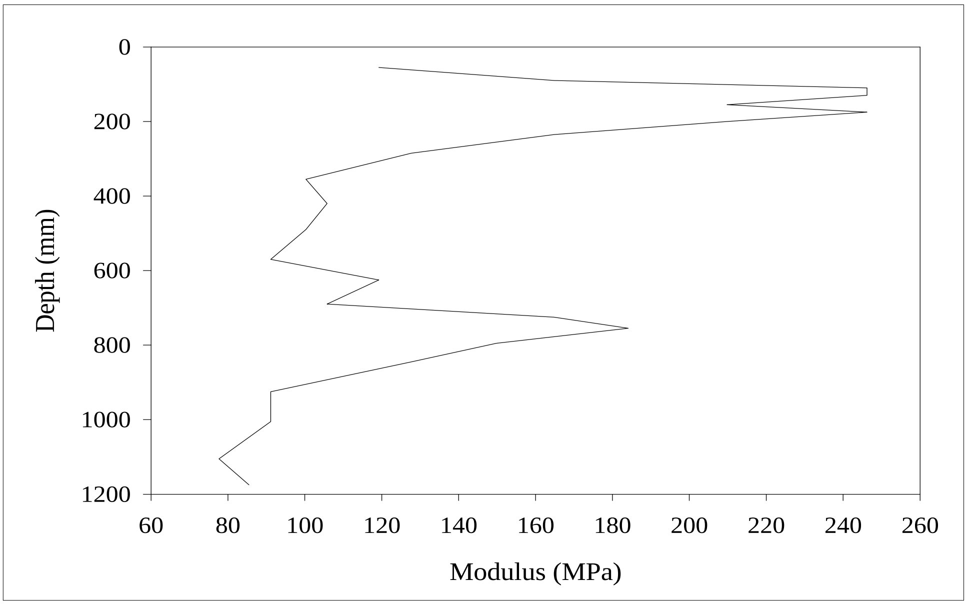

Figure 4.30 Modulus of subgrade reaction from Core#9 at Withayawithee 2 Road.

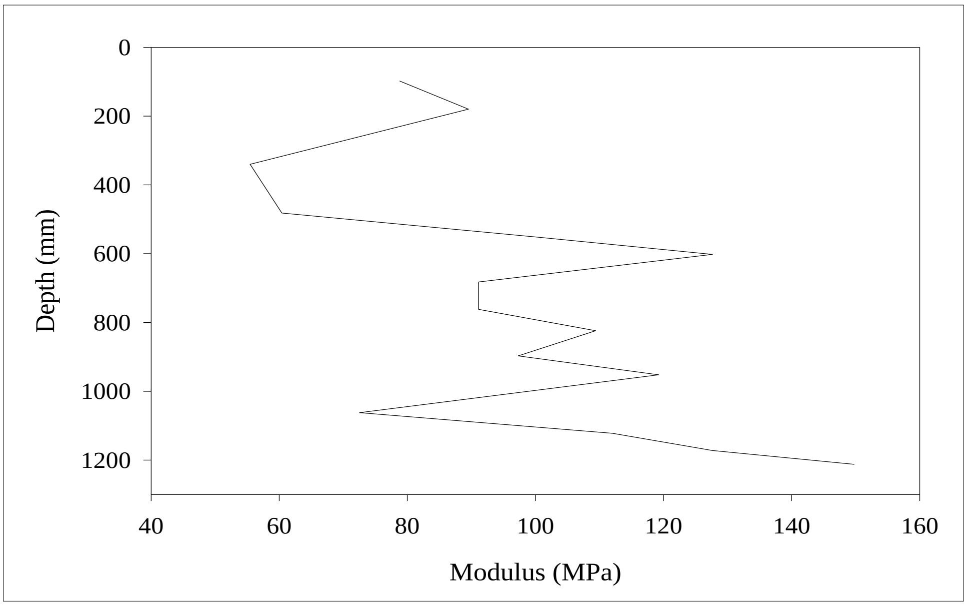

Figure 4.31 Modulus of subgrade reaction from Core#7 at Withayawithee 2 Road.

4.6.2 Withayawithee 3 Road

In Withayawithee 3 Road, similar patterns of weak layer just around 30 cm beneath the pavement as show in Figure 4.32 were observed in all the four core locations. Moduli from all the four areas were almost same with average around 100 MPa and CBR around 17%.

4.6.3 Sikkhawithee 1(a) Road

The profile monitoring test showed lesser settlements at this location compared to Withayawithee 2 and 3 Road, however DCP test showed that the subgrade at this location were very weak with average CBR 7.9% and modulus 64.15 MPa at Core#13 (Figure 4.33). Other DCP test at this road showed slightly higher parameters without much significance.

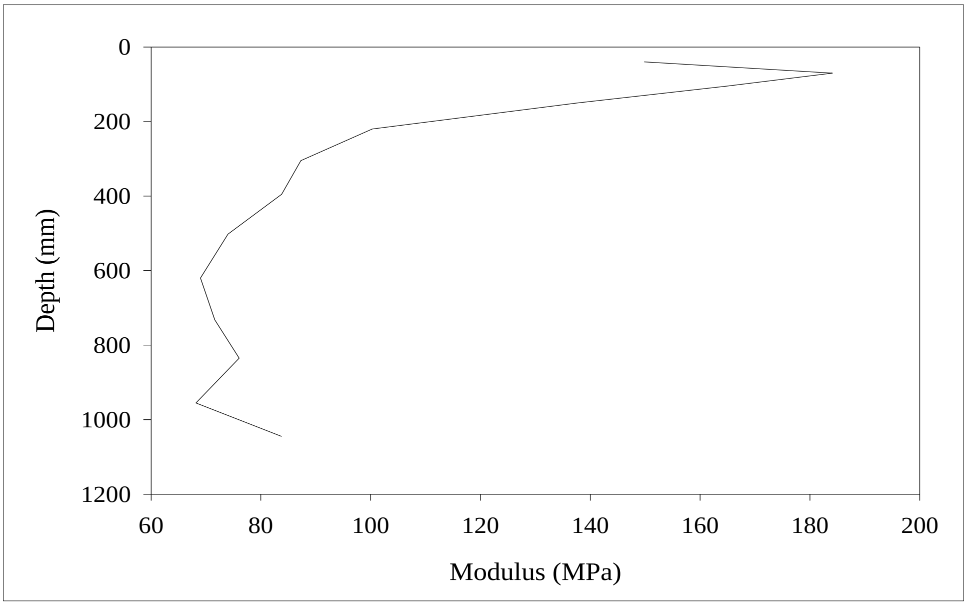

Figure 4.32 Modulus of subgrade reaction from Core#4 at Withayawithee 3 Road.

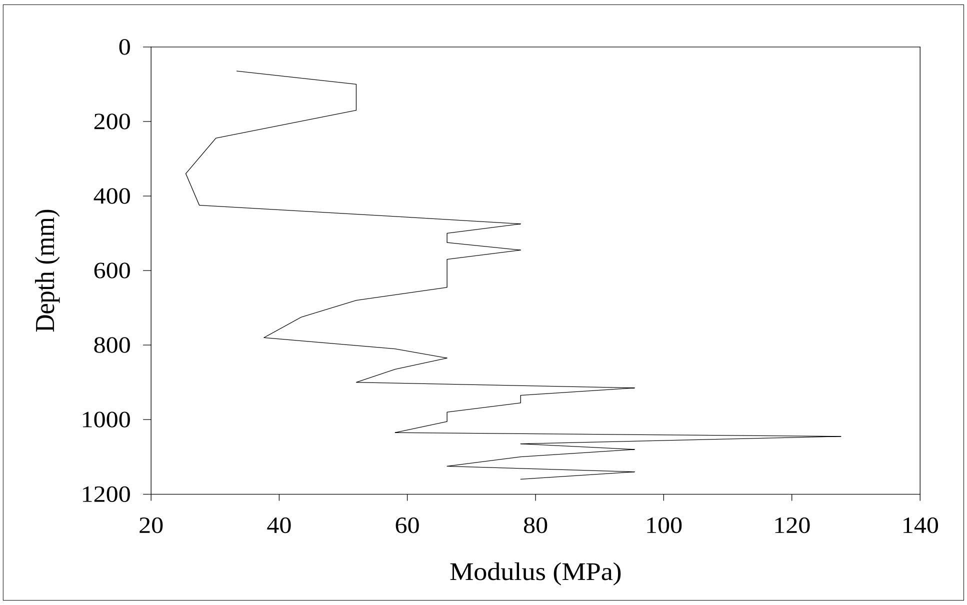

Figure 4.33 Modulus of subgrade reaction from Core#13 at Sikkhawithee 1(a) Road.

4.6.4 Sikkhawithee 1(b) Road

Modulus from this road segment contradicts the observations at other sites by having a near uniform throughout till the depth of the subgrade. However, there was not much deviation from other sites in case of average modulus and average CBR. These imply that the low base modulus is not the sole cause for settlement of the pavements. Other graphs related to DCP testing are shown in Appendix (D).

4.7 Soil Classification Testing Soil

In order to understand the observation from the pavement settlement and other tests conducted, it is necessary to know the type and characteristics of the subgrade and base. The soil samples from each site were taken and classified by Unconfined Soil Classification System (USCS) and the result is presented below. All of the sites confirmed that the approximate CBR value would be 10-40, Modulus of Subgrade Reaction (k) would be 200-300 pci. Furthermore, the soil would have excellent drainage condition.

Table 4.3 Soil Classification at the project site.

Sl. No.	Name of the Road	Corresponding soil sample core	Type of soil (cm)
1	Withayawithee 2 Road	Core#4	SW-SM
2	Withayawithee 3 Road	Core#6	SW-SM
3	Sikkhawithee 1(a) Road	Core#13	SP
4	Sikkhawithee 1(b) Road	Core#14	SW-SM

4.8 Pavement Condition Index Testing

ASTM D6433-03 was used for the calculation of the PCI index for each of the four test sections. These PCI results were associated to the rating system as that all sections were included within the “Fair” category. Table 4.4 summarizes the results for the PCI values for each section.

Table 4.4 Pavement Condition Index and rating for the study areas.

Name of the Road	PCI	Rating
Withayawithee 2 Road	46	Fair
Withayawithee 3 Road	48	Fair
Sikkhawithee 1(a) Road	52	Fair
Sikkhawithee 1(b) Road	52	Fair

The PCI for all sites were very similar with index 52% at Sikkhawithee 1 Road, 46% and 48% at Withayawithee 2 Road and Withayawithee 3 Road, respectively. All the sites suffered from low severity linear cracking and low severity joint sealing along with lane/shoulder drop-offs. Withayawithee 2 Road being the most effected as confirmed by distress survey and profile leveling, it showed high density (i.e. 37.5%) of medium severity divided slab. As mentioned in ASTM, no other distresses were counted when divided slab occurred with medium and high severity and thus the faulting and corner breaks at those sections were not accounted. Withayawithee 3 Road and Sikkhawithee 1(a) Road have similar distress with more numbers of linear cracking followed by lesser corner break. Sikkhawithee 1(b) Road has same density of divided slab and linear cracking with no signs of corner break.

With all the sites having PCI fair, not much can be concluded to compare the performance for the rehabilitation other than the need for the roads to be improve to facilitate quality and safety for the public.

CHAPTER V

CONCLUSIONS

5.1 Conclusion

The PFI rehabilitation method of rigid pavement has not been successful in providing the support to the slab for long run. However, it had lifted the pavement to the desired level and provided a good ride quality for a short span of time. The pavement lifted by PFI still experiences settlement and even new cracks have appeared on pavement. However, the settlement cannot be reasoned solely on PFI, increase traffic loading and sub-drainage can also be some of the contributing factors which can only be determined by further investigation. Information from the investigation of the pavement can be used to identify the causes of the problem and develop an optimal rehabilitation strategy to resolve the problem.

Even though all the site locations were built at same time and faced same condition of deterioration and maintenance, Withayawithee 2 Road showed the maximum distresses even after the repair. By the nature of the location of the road, Withayawithee 2 and 3 Roads facilitates traffic to more utility building than at the Sikkhawithee 1 Road. This might have contributed to large number of traffic leading to more deterioration of the pavement. All the tests conducted showed that Withayawithee Roads were weaker than the Sikkhawithee Roads.

Through coring it was confirmed that the typical pavement section of these roads is a 200-mm jointed reinforced concrete pavement (JRCP) over a soil base. Dowel bars, 25 mm in diameter by 450-mm long at 300-mm spacing, were placed at the transverse joints. Numerous types of distresses were observed throughout the length of the four sites. Linear cracking and divided slabs were most prominent where as joint sealing and lane/shoulder drop-offs were present throughout the length of the sections. It was evident from the visual survey that Withayawithee 2 Road was most affected. PCI though “FAIR” rated for all sites, this road has the lowest PCI.

Results from monitoring the elevation difference for the pavement to check the settlements of the pavement also single out Withayawithee 2 Road with maximum settlement over a period of 6 months. Even while polyurethane foam injection this road section had the maximum lift required to bring the pavement to desired level leading to more consumption of the material. Dynamic Cone Penetrometer testing define all the sites with almost similar penetration rates, however, Withayawithee 2 Road showed slight higher penetration then other roads.

Following were some of the findings drawn from the investigation conducted in this study:

1. A core extracted from a faulted joint found void beneath it which clearly indicated that polyurethane was unable to penetrate through the fault. Therefore, it would not have occurred if the injection was thoroughly done on both sides of the slabs which would have proven a better workmanship.
2. The presence of hard base material had caused settlements at some portion of the pavement while lifted the other portion. This might led to the

movement of the slab resulting in instability which may have been prevented by uniformly lifting the pavement.

3. The extraction of core found to be a practical tool to present clear picture of the polyurethane thickness and pavement distresses. DCP result showed that the base material were weak at all the sites, which had led to cracking. This emphasizes the need for a good base material to support the wheel loads applied.
4. The soil classification and DCP testing had confirmed that the type soil underneath the pavement was only sandy soil which falls in groups of either SW-SW or SP under Unified Soil Classification System (USCS).
5. As per the PCI testing, all sites were rated “FAIR” which imply that the road need immediate improvement to provide a better performance.

5.2 Recommendation

The distresses in concrete pavements are caused by more than single factor which is why indentifying the distress mechanisms requires a comprehensive understanding of the factors involved and their interaction was a great challenge (Chen et al., 2009). Therefore, more investigation can be carried out to find out the optimal rehabilitation strategy.

After drawing above conclusions it would be wise to recommend that Polyurethane Foam Injection (PFI) can be used as a supplementary to other rehabilitation methods or as a temporary method to realign pavement. The limitation of Polyurethane material at penetrating the base would initiate more investigation on the material. Polyurethane foam is used in many applications such as insulation and void filling; therefore, it has become necessary to develop the material to suit the condition as required to pavement repair where repeated loads are applied. The procedure to inject polyurethane foam was explained in chapter II and chapter III. To distribute the material uniformly beneath and filling all the voids, the use DCP or Hyper optic void detection system to locate void must be used to improve the workmanship of the rehabilitation method. The sagging of one portion due to lifting of another portion could have been prevented if monitored properly.

From visual observation at the site location, it was observed that road has minimum maintenance. Routine maintenances are need for all type of road whether it is designed and constructed with scientific bias or not. The longitudinal and cross drains would need attention under the routine maintenance work which includes removal or silt, rubbish and weed (Khanna and Justo, 1971). The site locations were also observed to have undergone crack sealing at the same time as PFI but the crack sealing did not serve it purpose as it had widen the cracks more. Entire area was suffering from distress lane/shoulder drop-offs and drainage blockage but it was not given enough attention.

From this study, the rehabilitation method for repairing the pavement had not been successful enough which imply that a new rehabilitation method must be studied. In addition, it implies the importance of identifying the defects that may cause pavement failure and adopt measures to maintain the pavement. From this study, it could be suggested to have retaining walls through the length of Withayawithee 2 Road to protect losing soil from the base by scouring. This may be a contribution factor since the road runs through a slope area. As for the other sites, mud jacking or undersealing might work but a full depth repair would be suggested if the base is strengthen by using aggregate base or stabilized aggregate and soil base or dense graded HMA or lean concrete.

From the data gathered by this study can be used as a basis for any forth coming rehabilitation procedure studies. The entire distressed are mapped along with PCI calculation allowing the comparisons to be drawn between the existing pavement and the eventually the rehabilitated pavement.

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APPENDIX A

PROFILE LEVELING

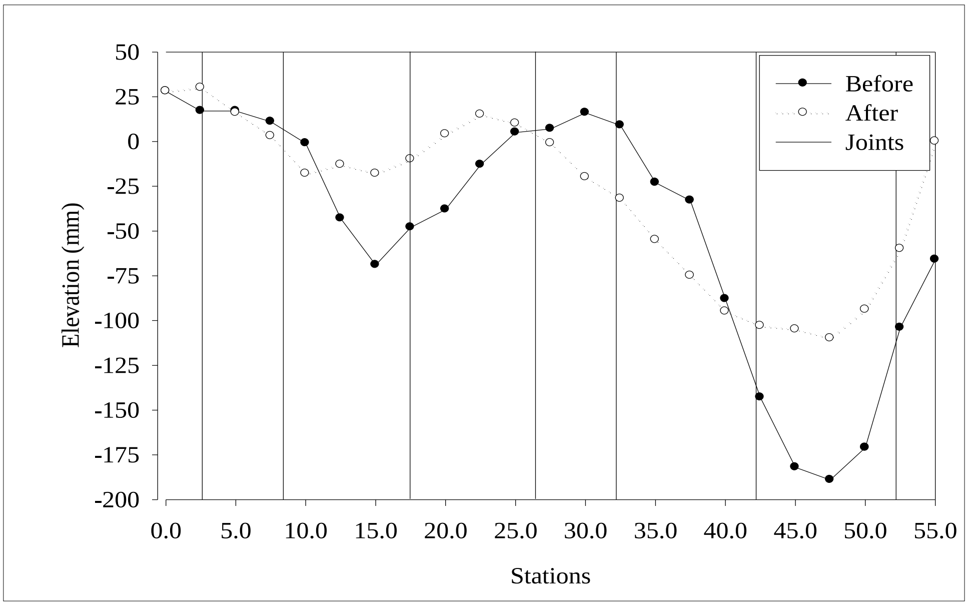

Figure A.1 Profile leveling before and after polyurethane foam injection on left lane of Withayawithee 2 Road.

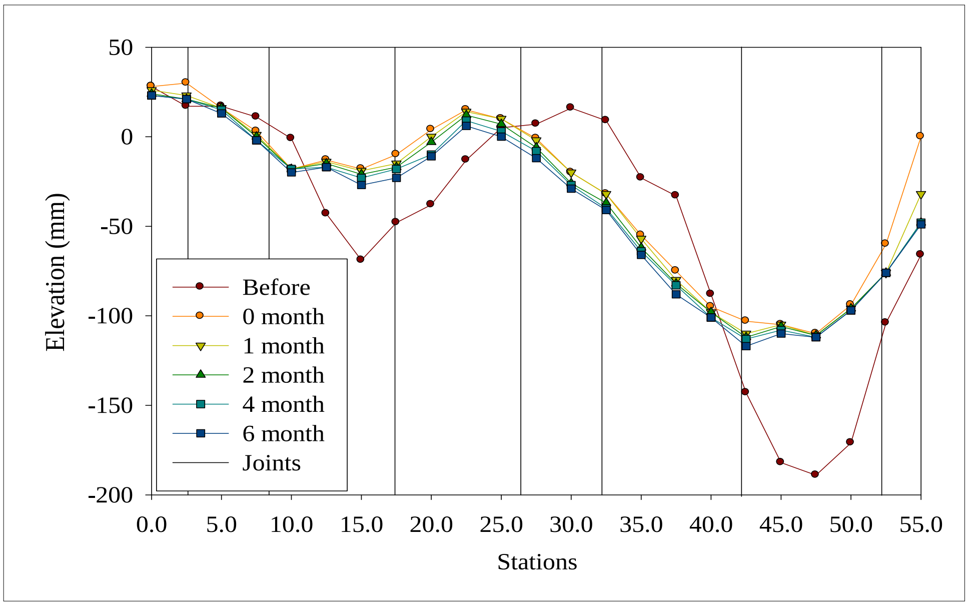

Figure A.2 Settlements on left lane of Withayawithee 2 Road during the study period.

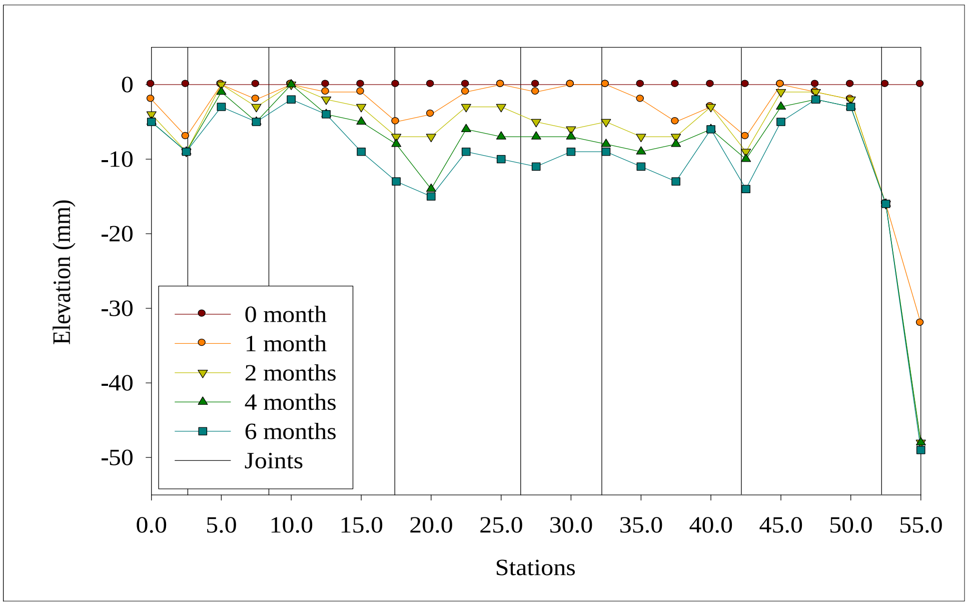

Figure A.3 Differential settlement on left lane of Withayawithee 2 Road during the study period.

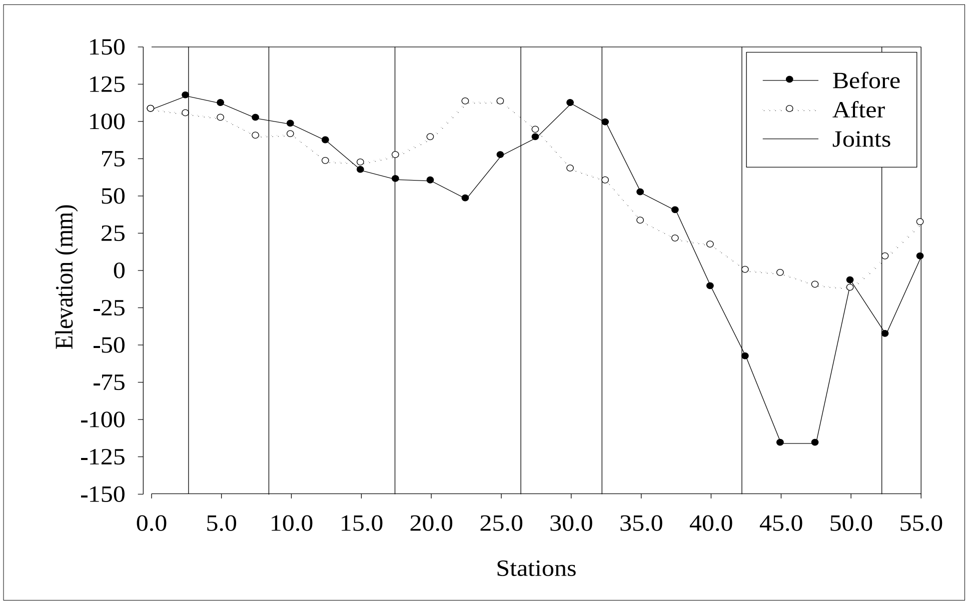

Figure A.4 Profile leveling before and after polyurethane foam injection at the middle of lanes of Withayawithee 2 Road.

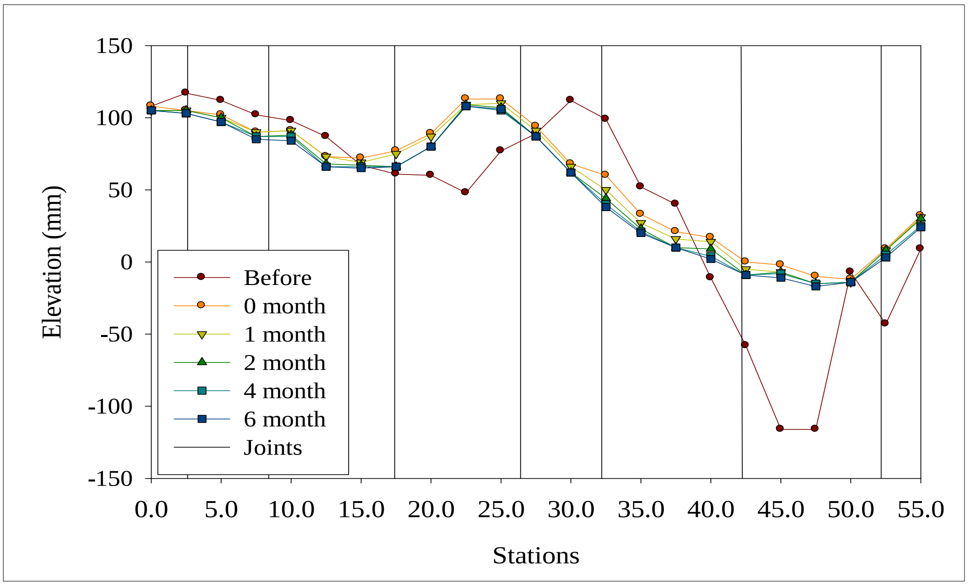

Figure A.5 Settlements at the middle of lanes of Withayawithee 2 Road during the study period.

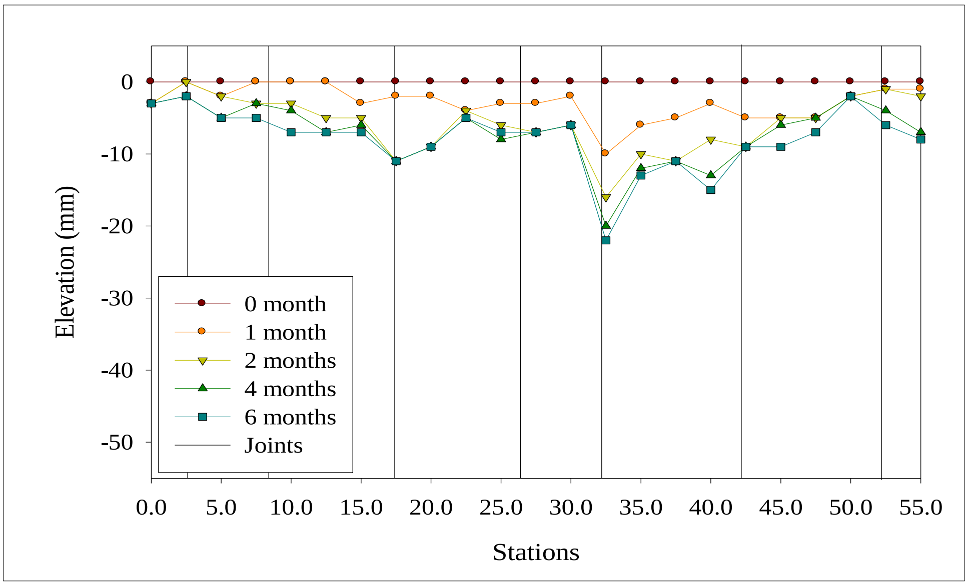

Figure A.6 Differential settlement at the middle of lanes of Withayawithee 2 Road during the study period.

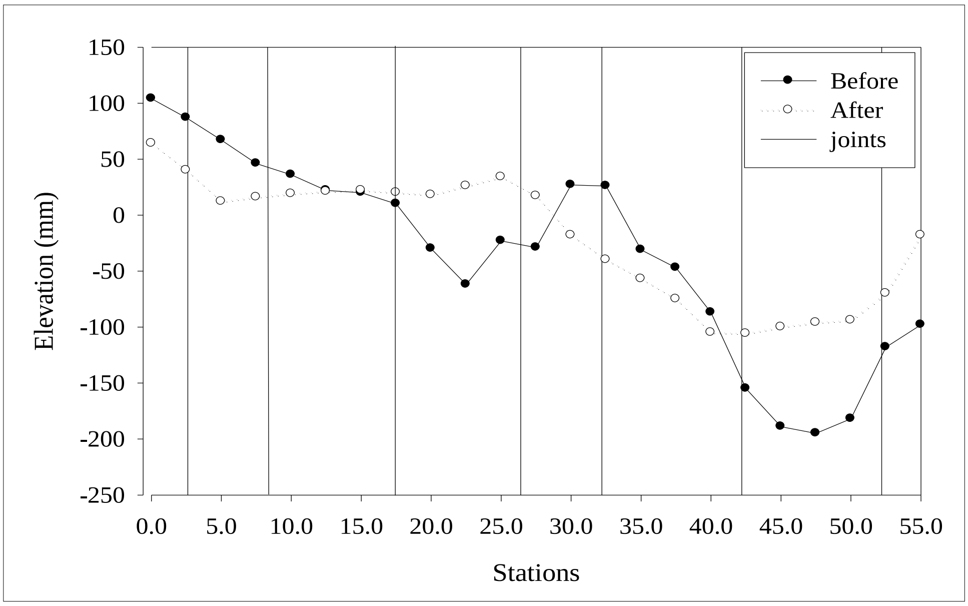

Figure A.7 Profile leveling before and after polyurethane foam injection on right lane of Withayawithee 2 Road.

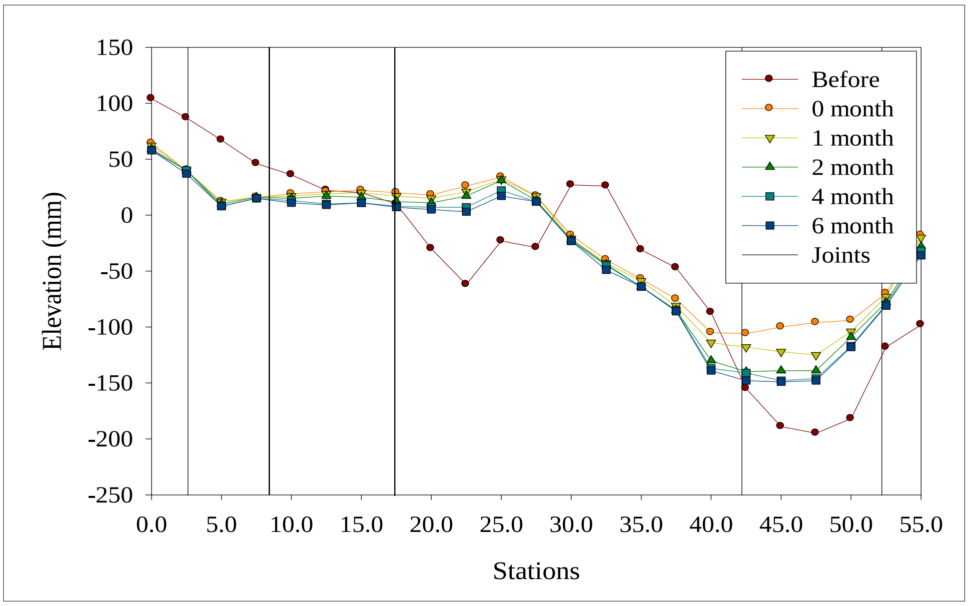

Figure A.8 Settlements on right lane of Withayawithee 2 Road during the study period.

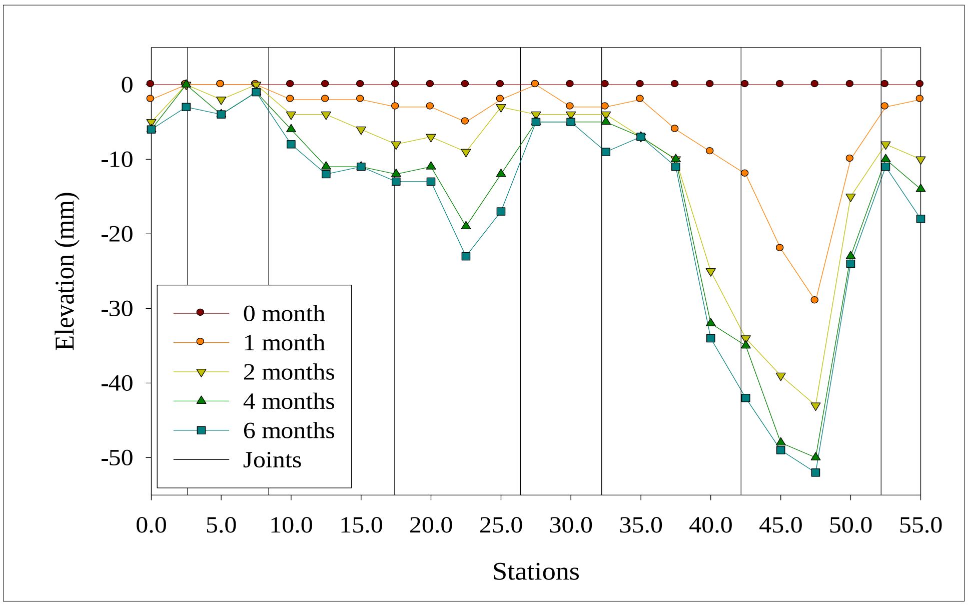

Figure A.9 Differential settlement on right lane of Withayawithee 2 Road during the study period.

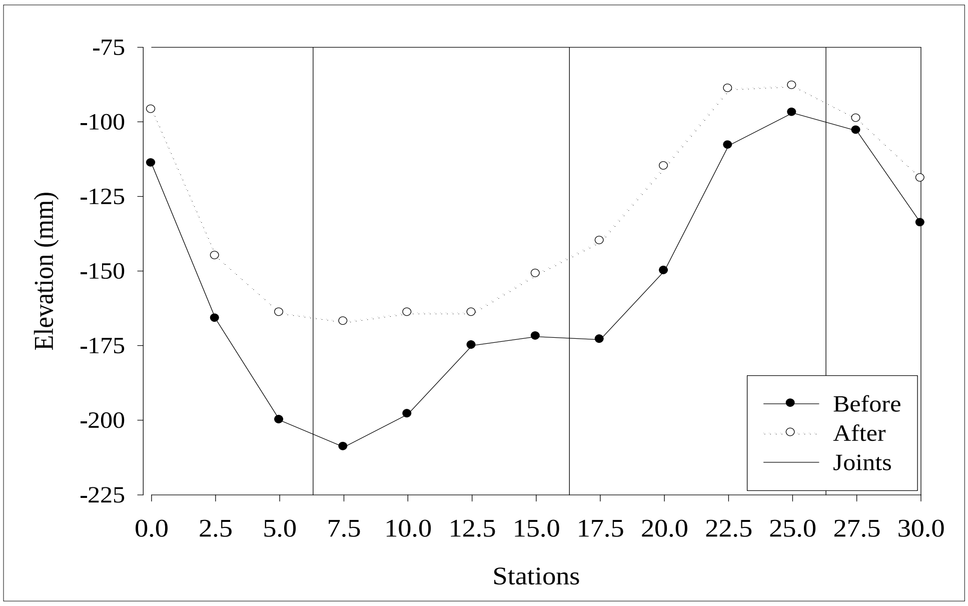

Figure A.10 Profile leveling before and after polyurethane foam injection on left lane of Withayawithee 3 Road.

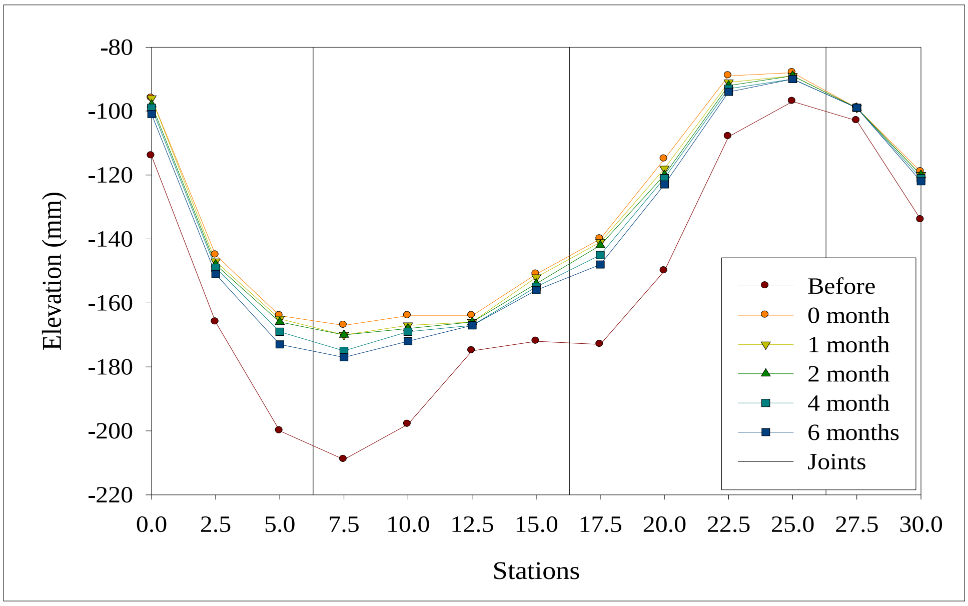

Figure A.11 Settlements on left lane of Withayawithee 3 Road during the study period.

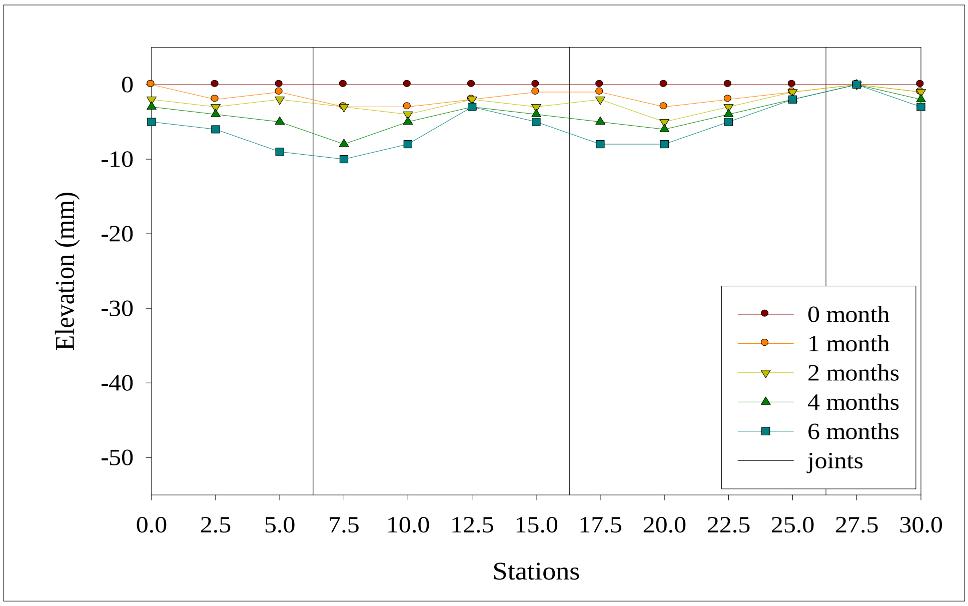

Figure A.12 Differential settlement on left lane of Withayawithee 3 Road during the study period.

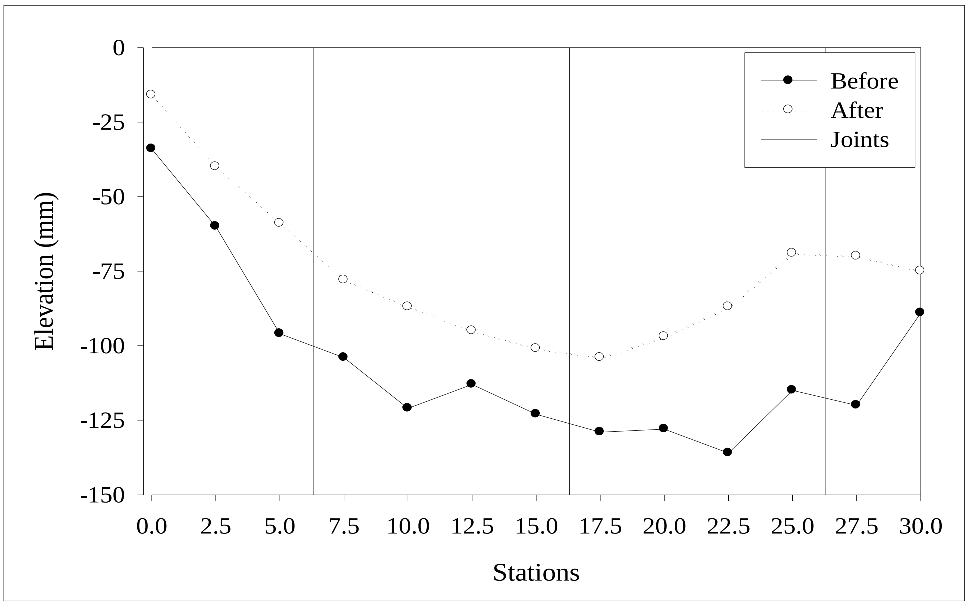

Figure A.13 Profile leveling before and after polyurethane foam injection at the middle of lanes of Withayawithee 3 Road.

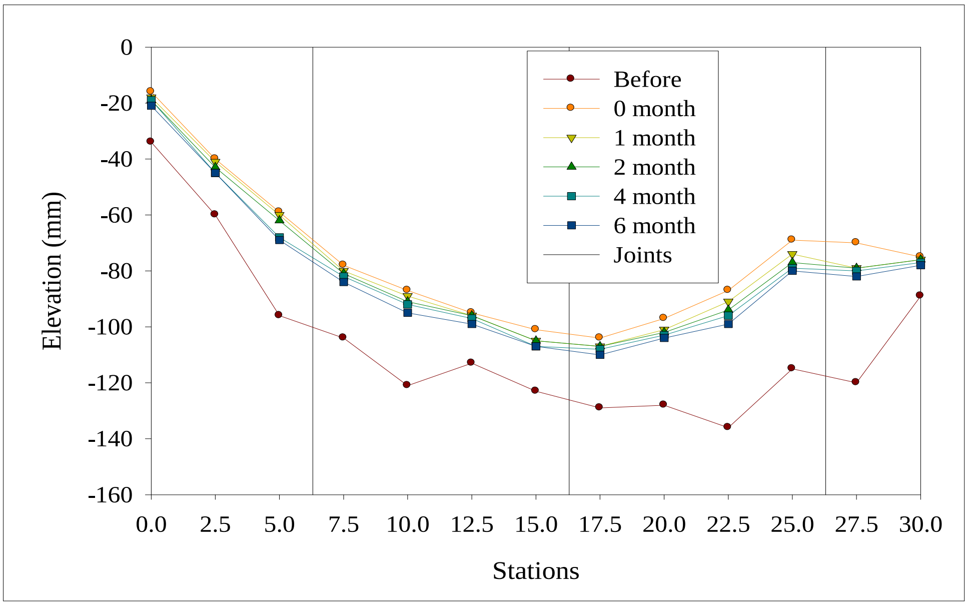

Figure A.14 Settlements at the middle of lanes of Withayawithee 3 Road during the study period.

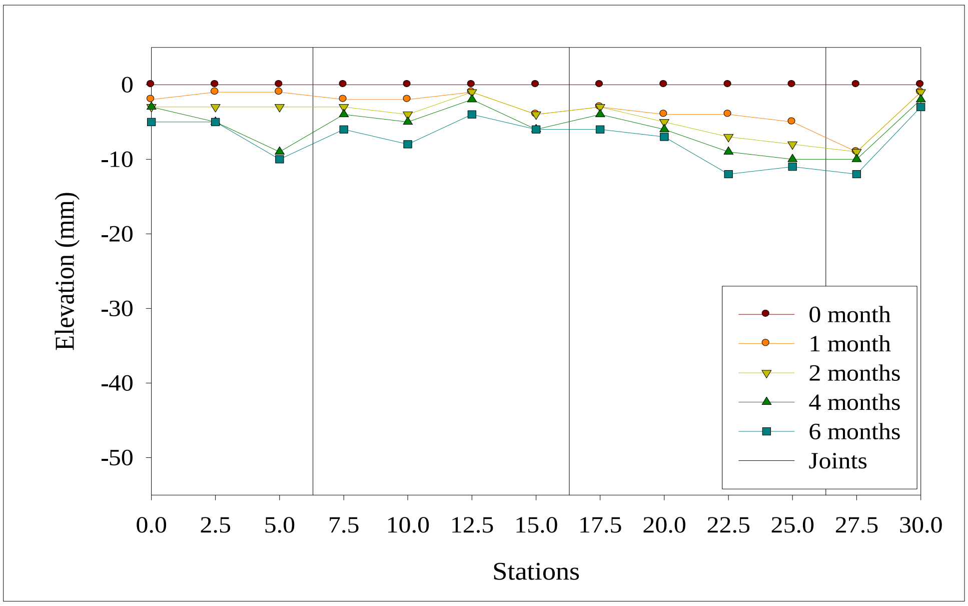

Figure A.15 Differential settlement at the middle of lanes of Withayawithee 3 Road during the study period.

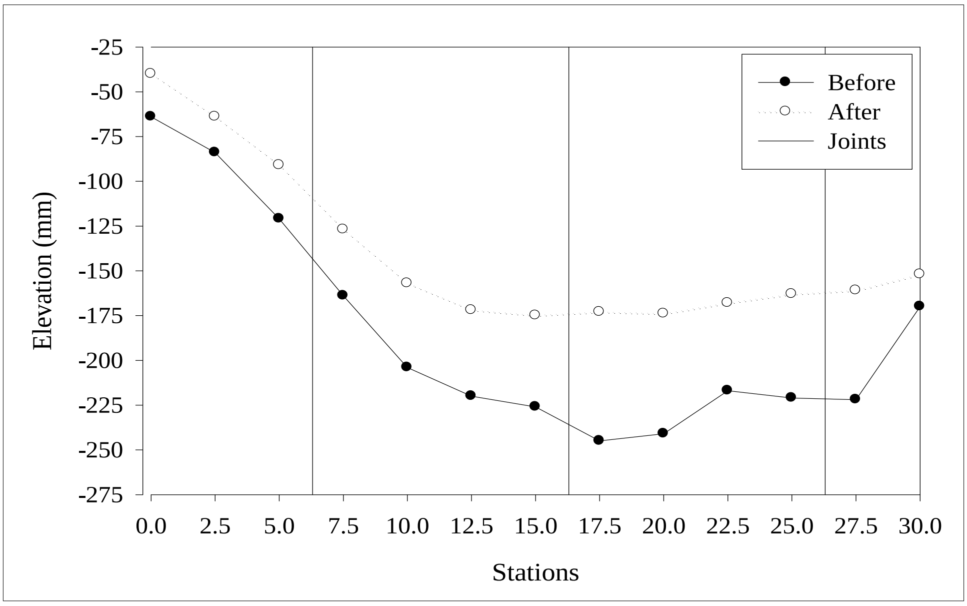

Figure A.16 Profile leveling before and after polyurethane foam injection on right lane of Withayawithee 3 Road.

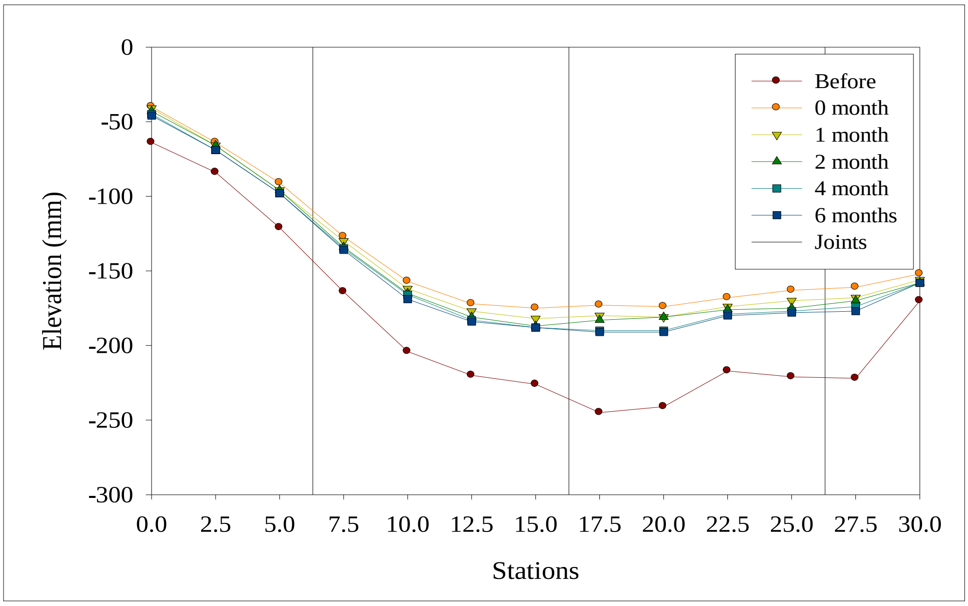

Figure A.17 Settlements on right lane of Withayawithee 3 Road during the study period.

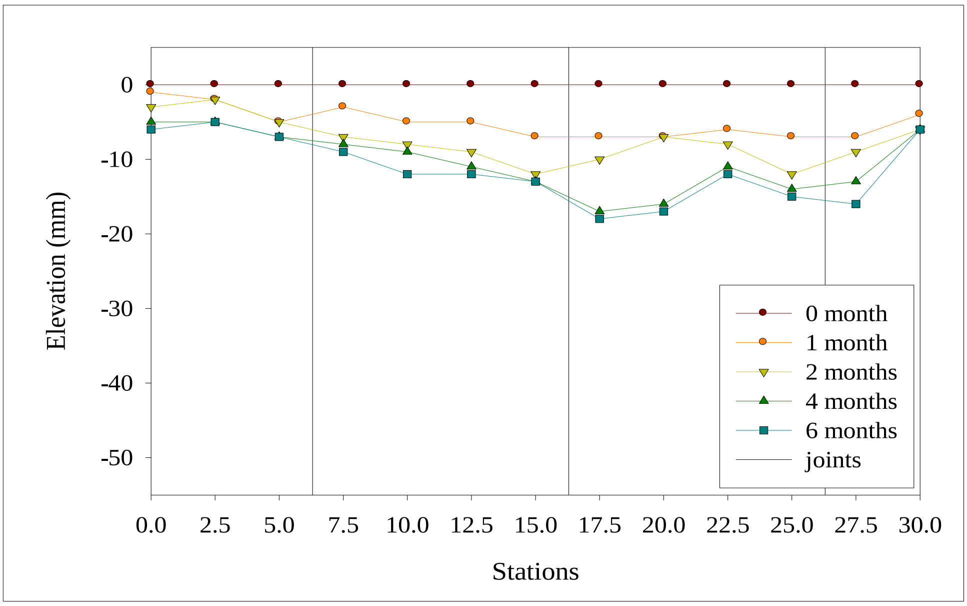

Figure A.18 Differential settlement on right lane of Withayawithee 3 Road during the study period.

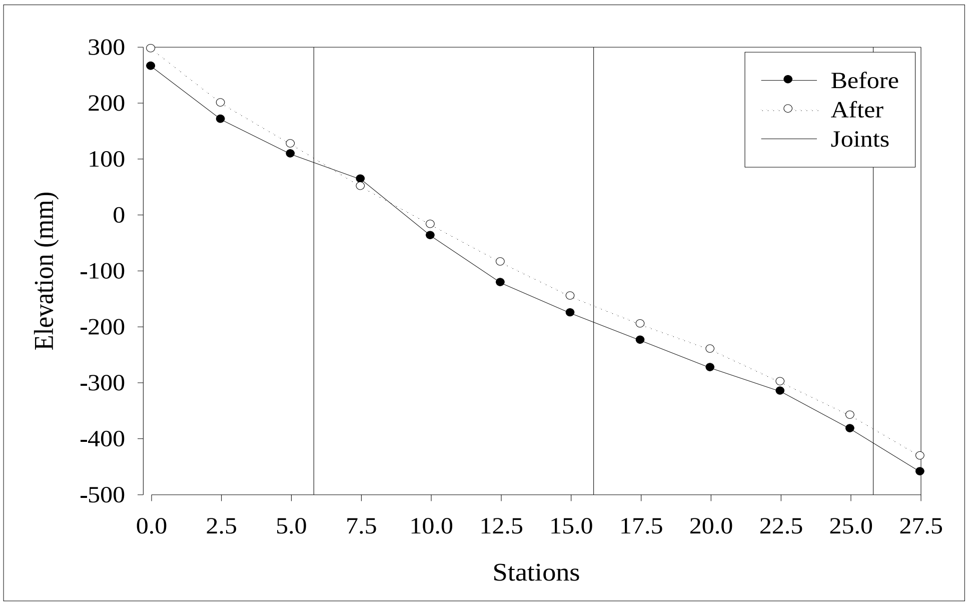

Figure A.19 Profile leveling before and after polyurethane foam injection on left lane of Sikkhawithee 1(a) Road.

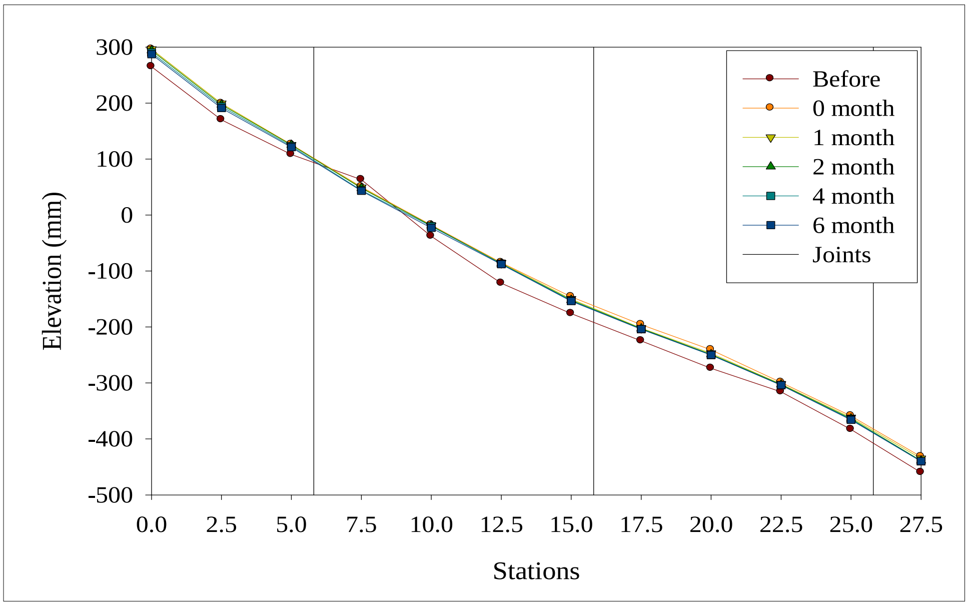

Figure A.20 Settlements on left lane of Sikkhawithee 1(a) Road during the study period.

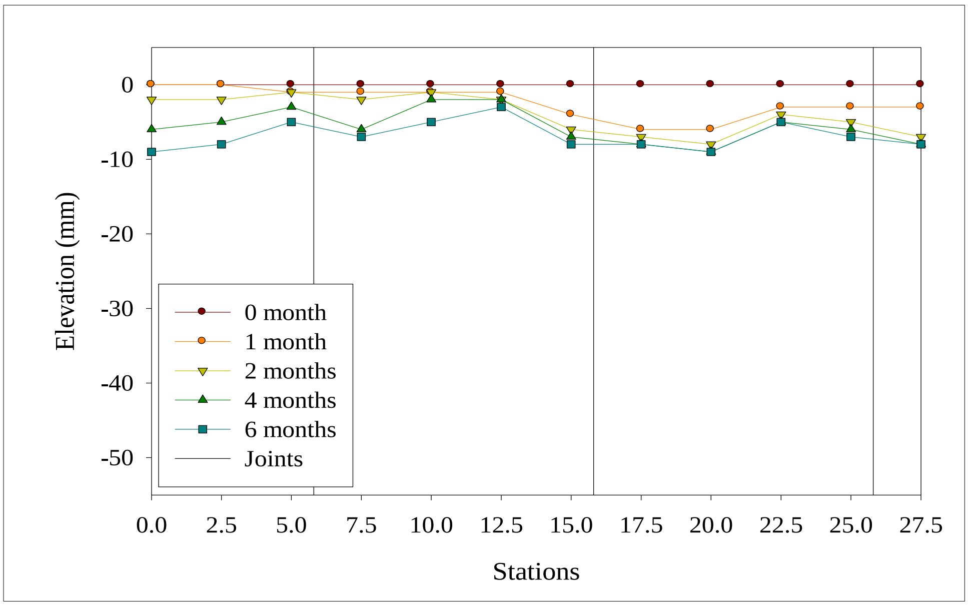

Figure A.21 Differential settlement on left lane of Sikkhawithee 1(a) Road during the study period.

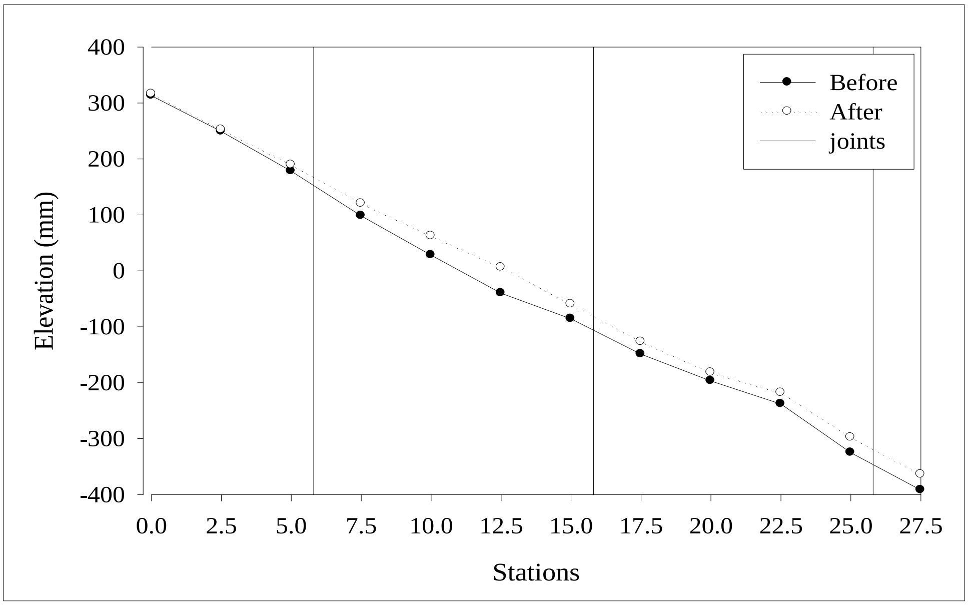

Figure A.22 Profile leveling before and after polyurethane foam injection at the middle of lanes of Sikkhawithee 1(a) Road.

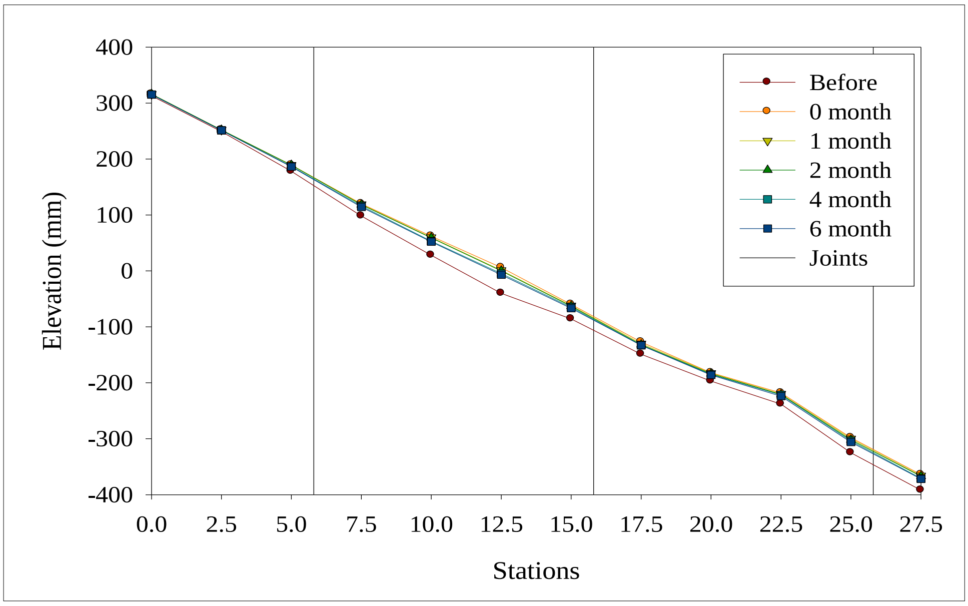

Figure A.23 Settlements at the middle of lanes of Sikkhawithee 1(a) Road during the study period.

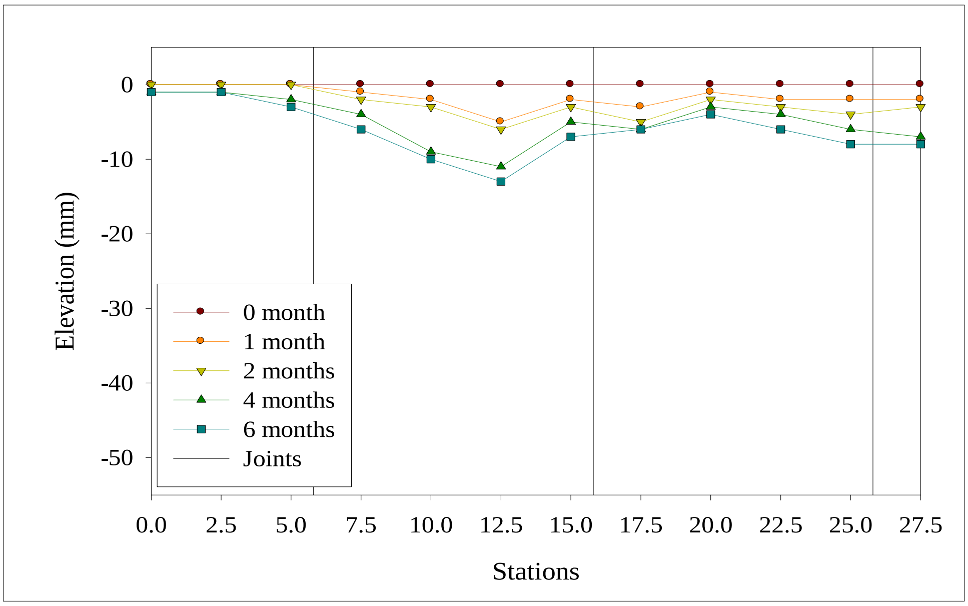

Figure A.24 Differential settlement at the middle of lanes at Sikkhawithee 1(a) Road during the study period.

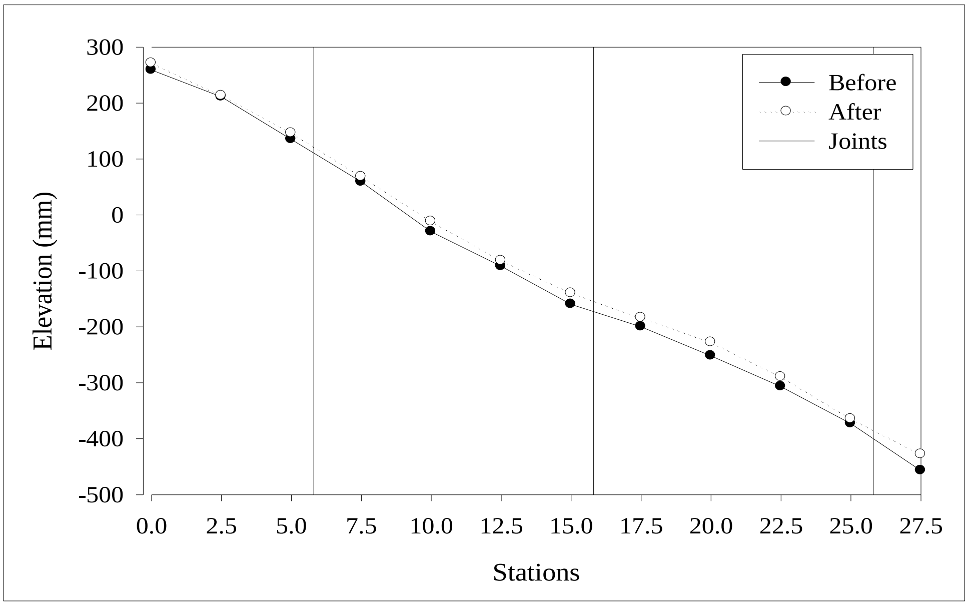

Figure A.25 Profile leveling before and after polyurethane foam injection on right lane of Sikkhawithee 1(a) Road.

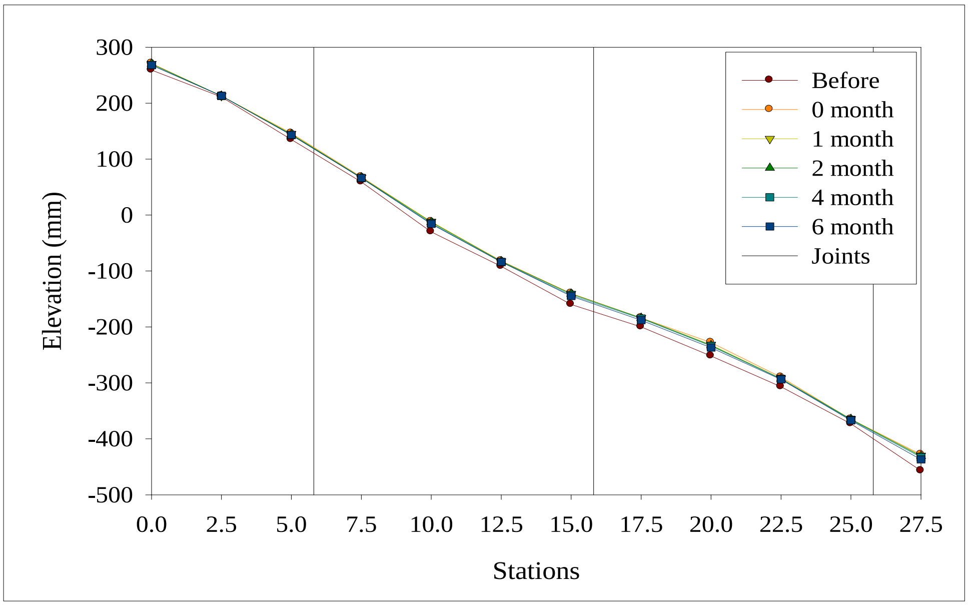

Figure A.26 Settlements on right lane of Sikkhawithee 1(a) Road during the study period.

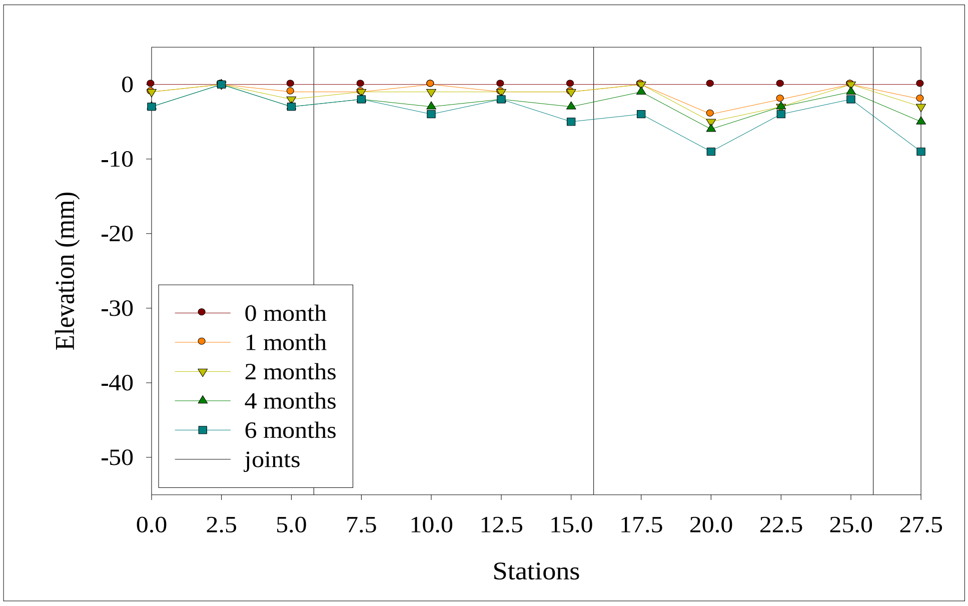

Figure A.27 Differential settlement on right lane of Sikkhawithee 1(a) Road during the study period.

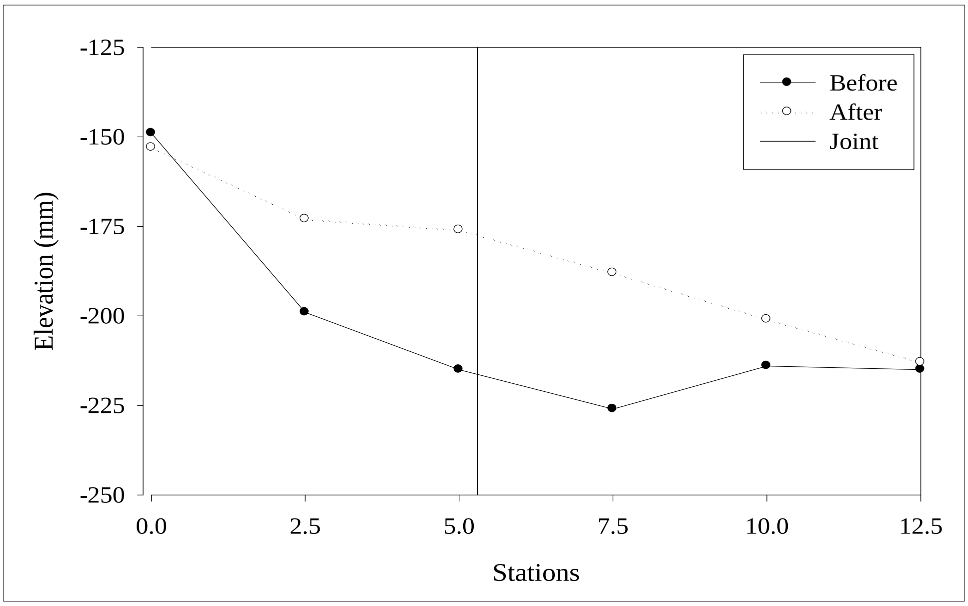

Figure A.28 Profile leveling before and after polyurethane foam injection on left lane of Sikkhawithee 1(b) Road.

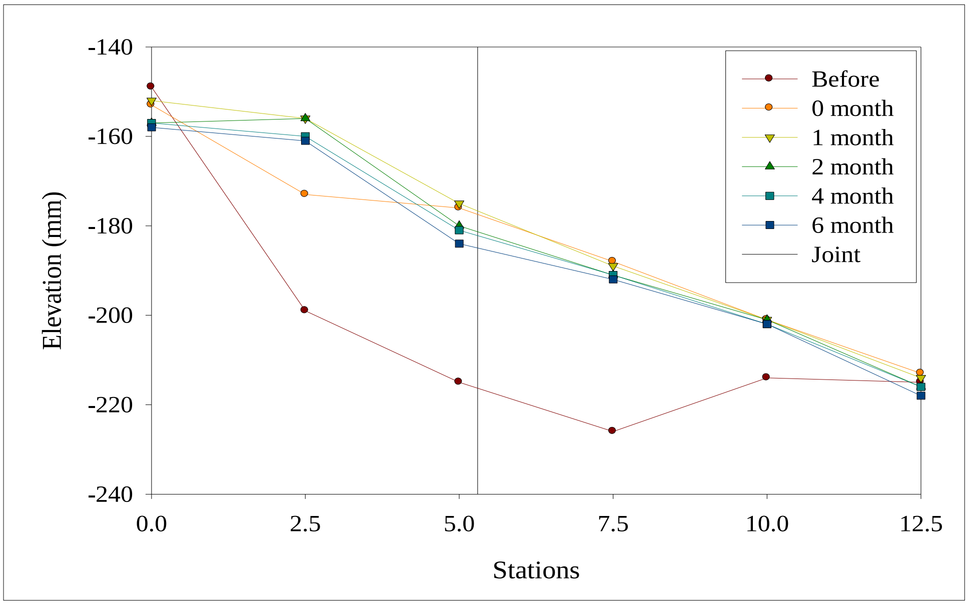

Figure A.29 Settlements on left lane of Sikkhawithee 1(b) Road during the study period.

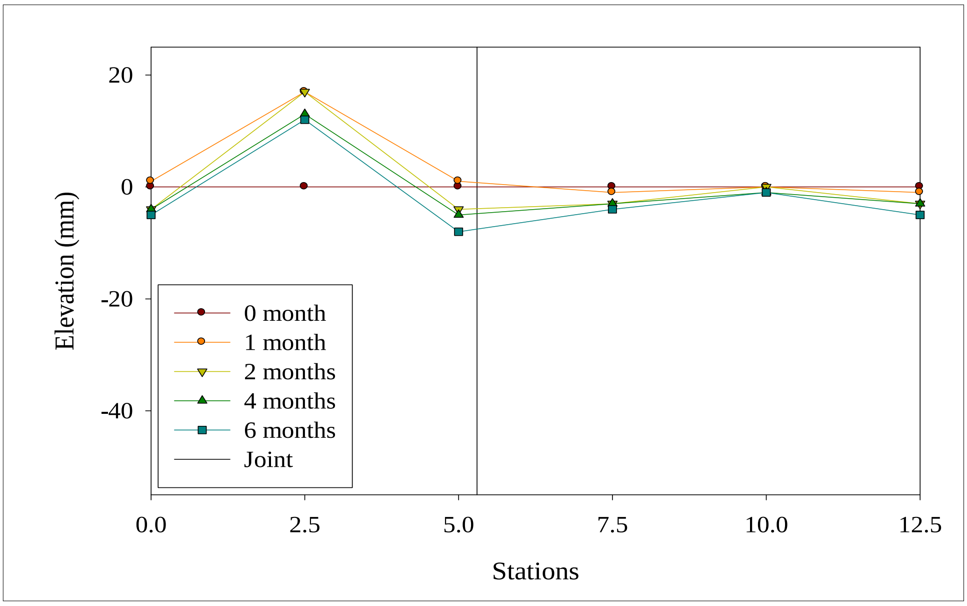

Figure A.30 Differential settlement on left lane of Sikkhawithee 1(b) Road during the study period.

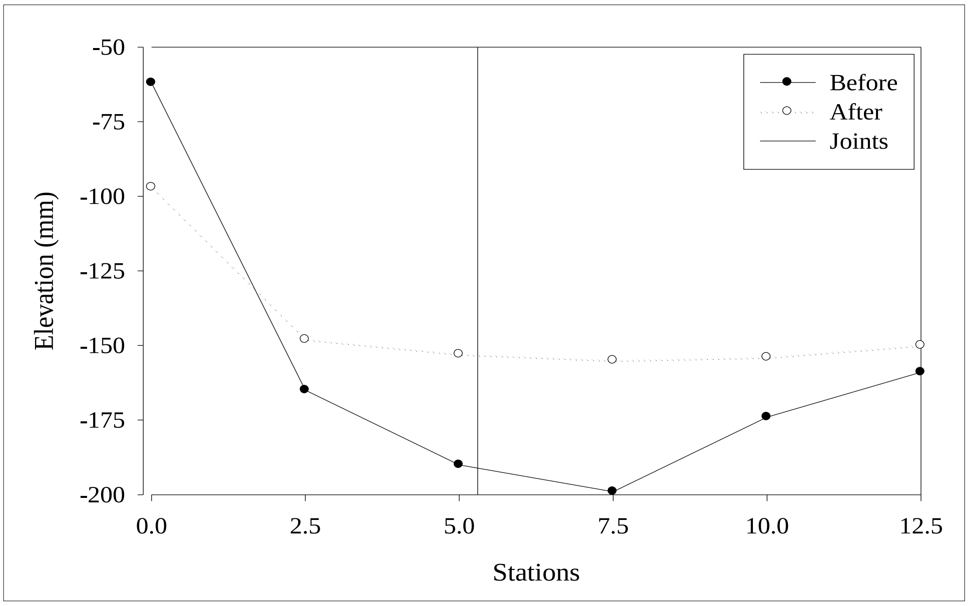

Figure A.31 Profile leveling before and after polyurethane foam injection at the middle of lanes of Sikkhawithee 1(b) Road.

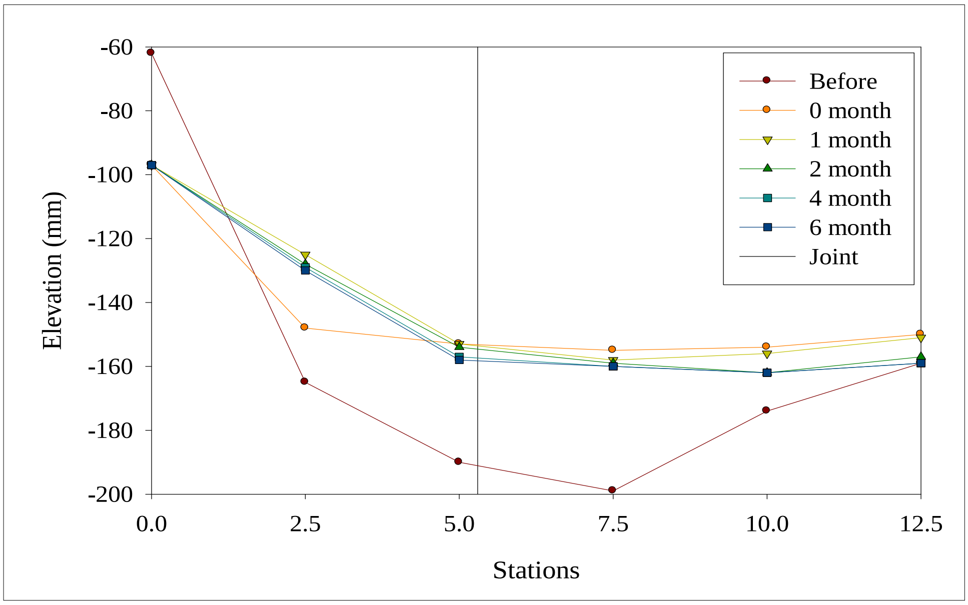

Figure A.32 Settlements at the middle of lanes at Sikkhawithee 1(b) Road during the study period.

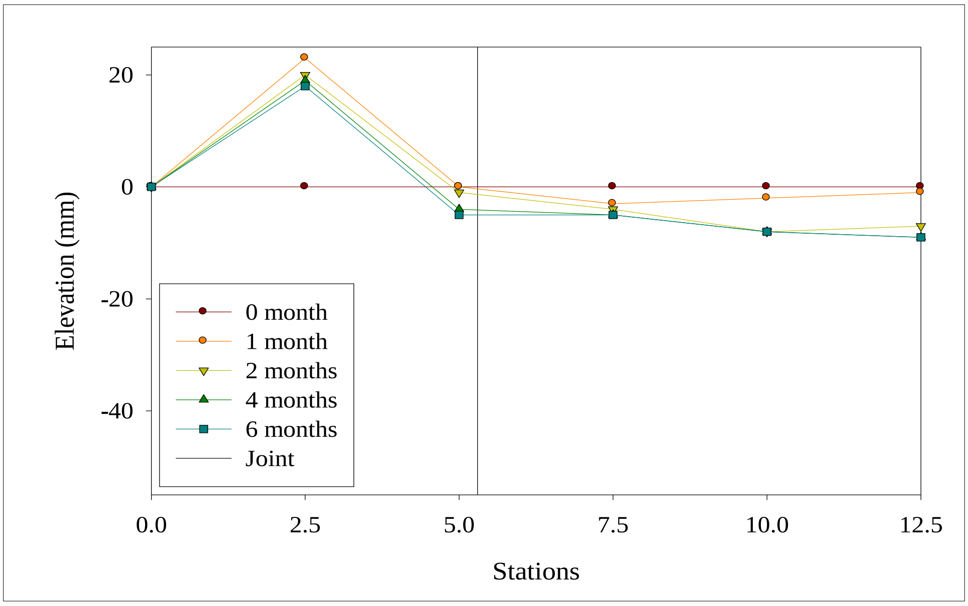

Figure A.33 Differential settlement at the middle of lanes of Sikkhawithee 1(b) Road during the study period.

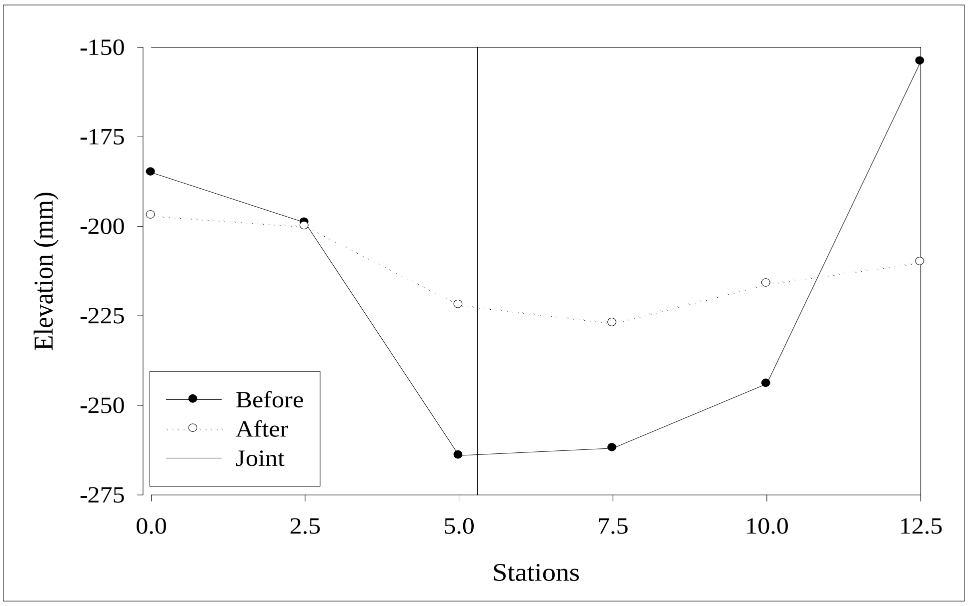

Figure A.34 Profile leveling before and after polyurethane foam injection on right lane of Sikkhawithee 1(b) Road.

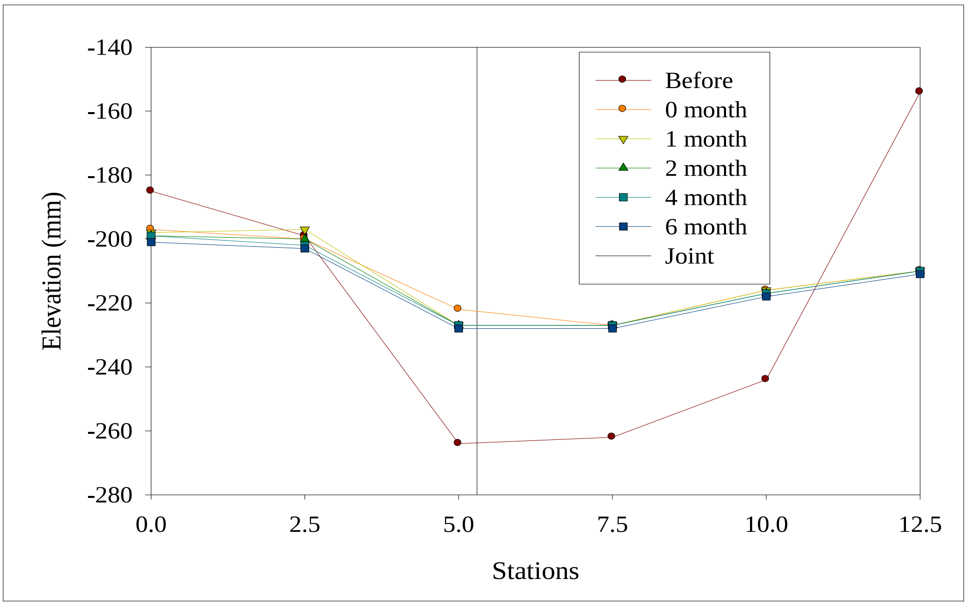

Figure A.35 Settlements on right lane of Sikkhawithee 1(b) Road during the study period.

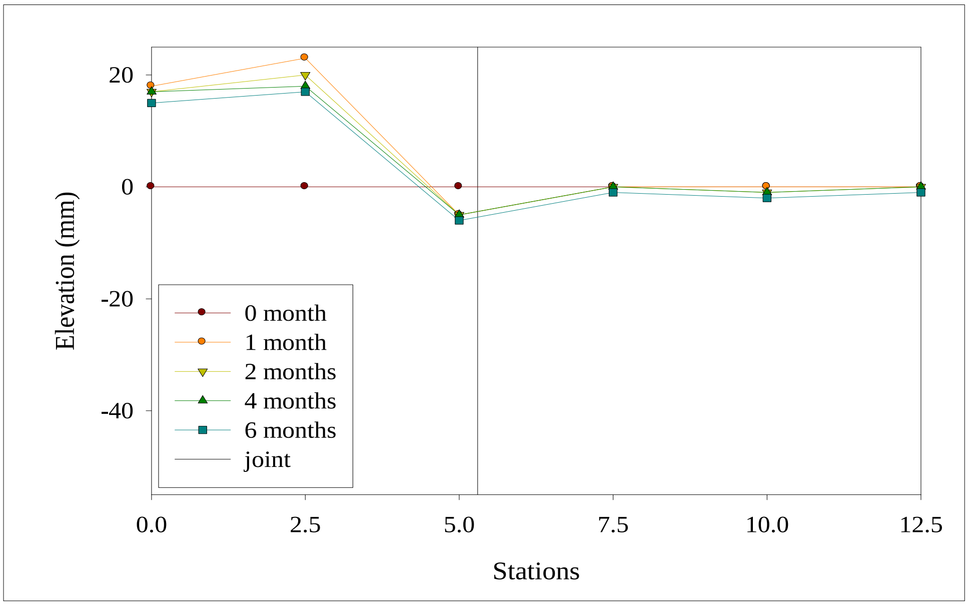

Figure A.36 Differential settlement on right lane of Sikkhawithee 1(b) Road during the study period.

APPENDIX B

DISTRESS MAPS

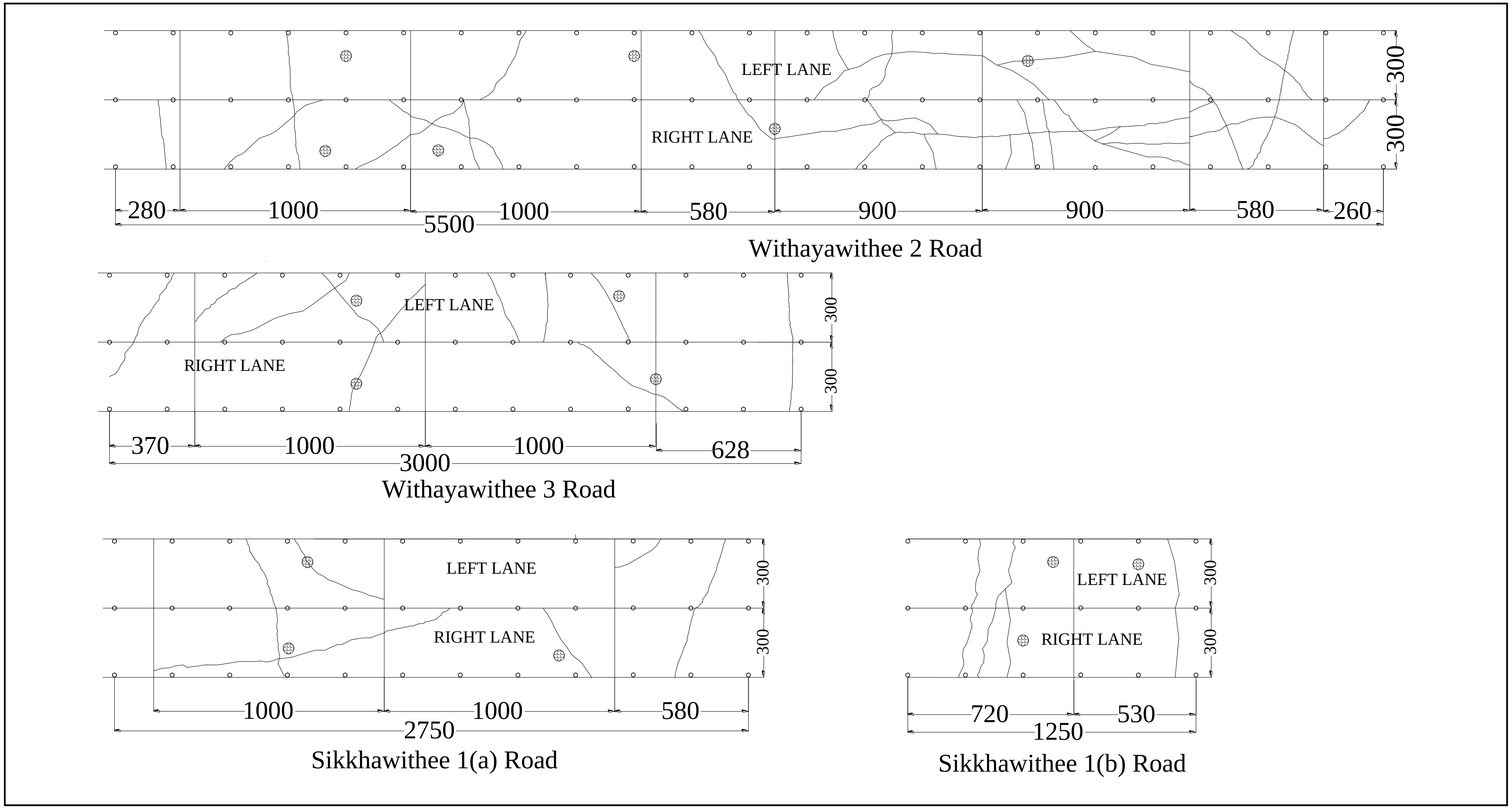

Figure B.1 Distress map for all the four section (showing linear cracks).

APPENDIX C

CORING GRAPH

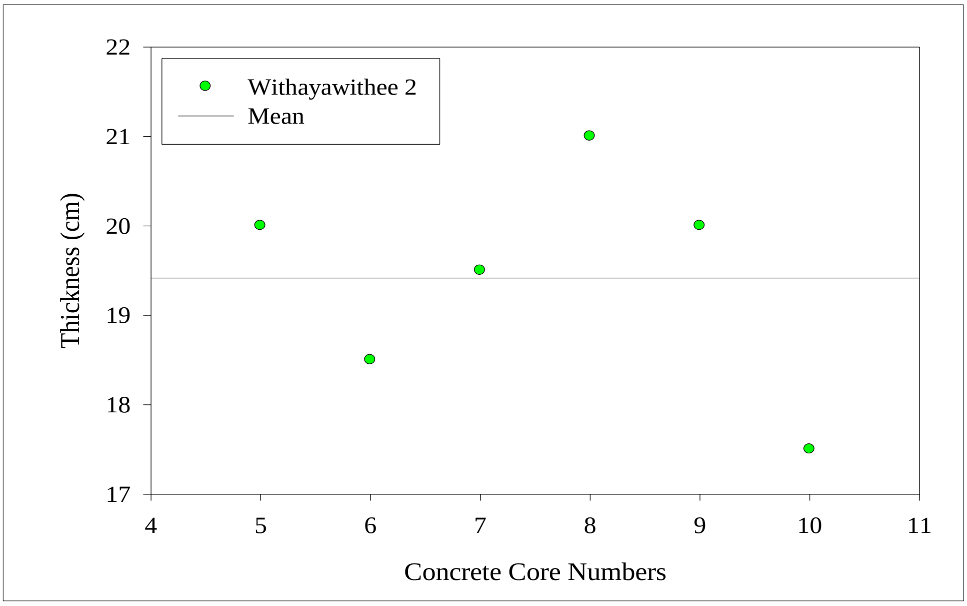

Figure C.1 Variation of concrete core thickness and its mean at Withayawithee 2 Road.

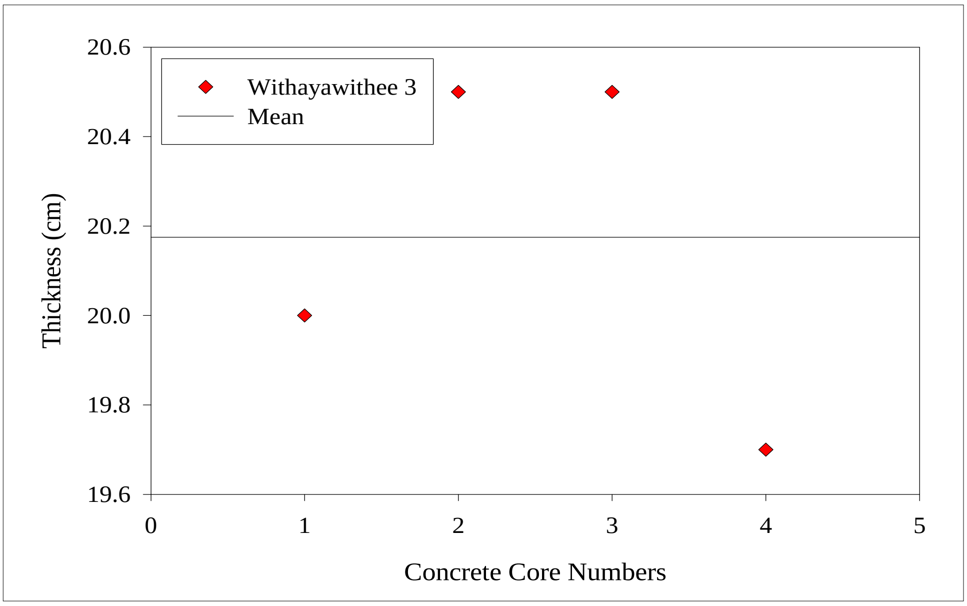

Figure C.2 Variation of concrete core thickness and its mean at Withayawithee 3 Road.

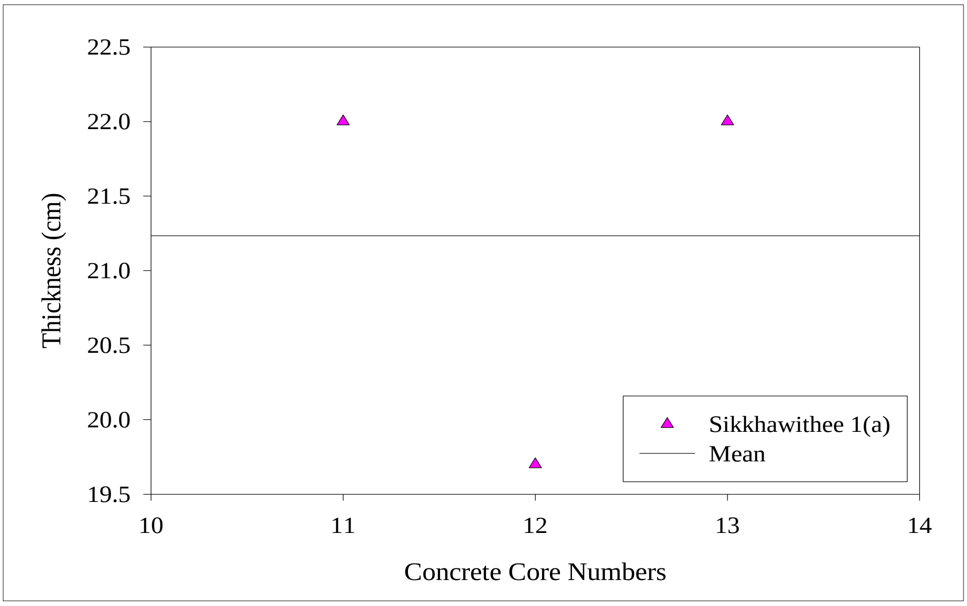

Figure C.3 Variation of concrete core thickness and its mean at Sikkhawithee 1(a) Road.

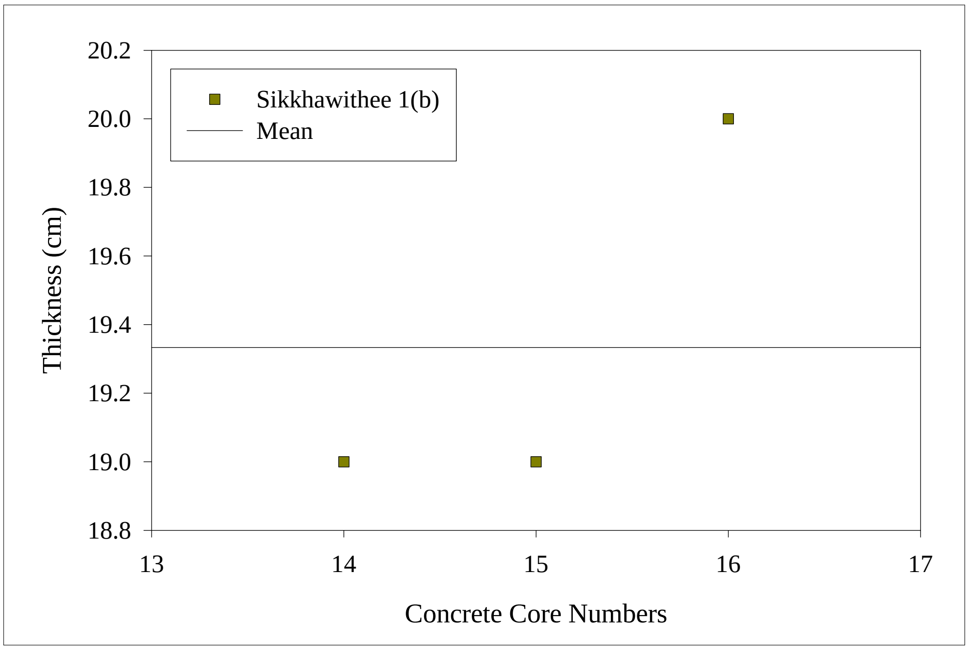

Figure C.4 Variation of concrete core thickness and its mean at Sikkhawithee 1(b) Road.

APPENDIX D

DYNAMIC CONE PENETROMETER GRAPH

1. Blow vs. Depth

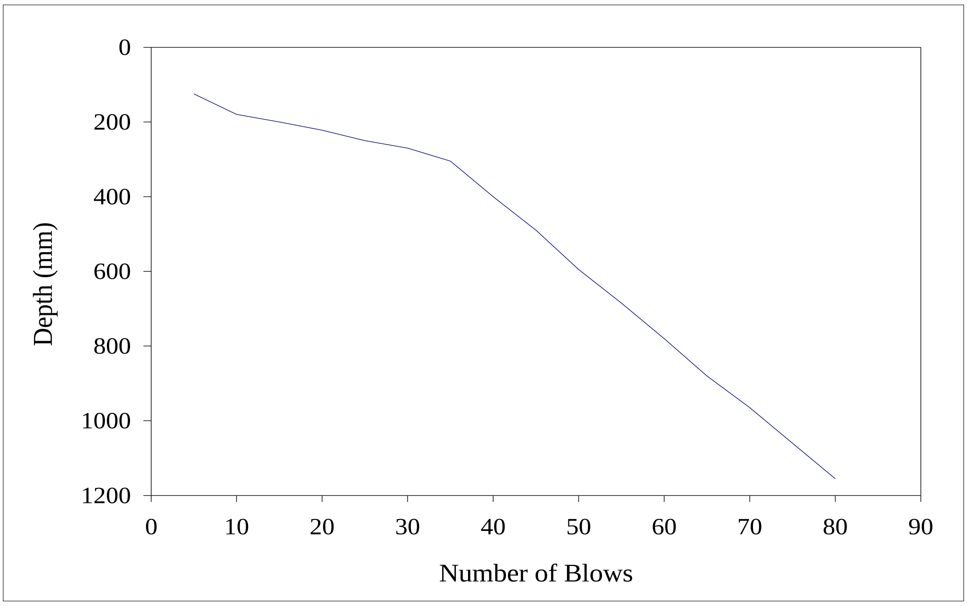

Figure D.1 Blow vs. depth graph for Core#1 at Withayawithee 3 Road.

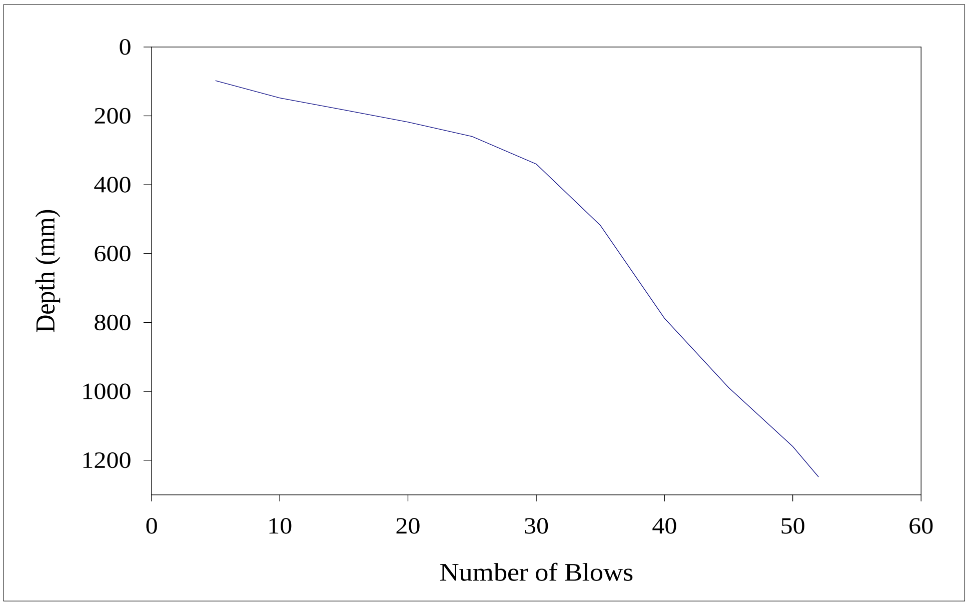

Figure D.2 Blow vs. depth graph for Core#2 at Withayawithee 3 Road.

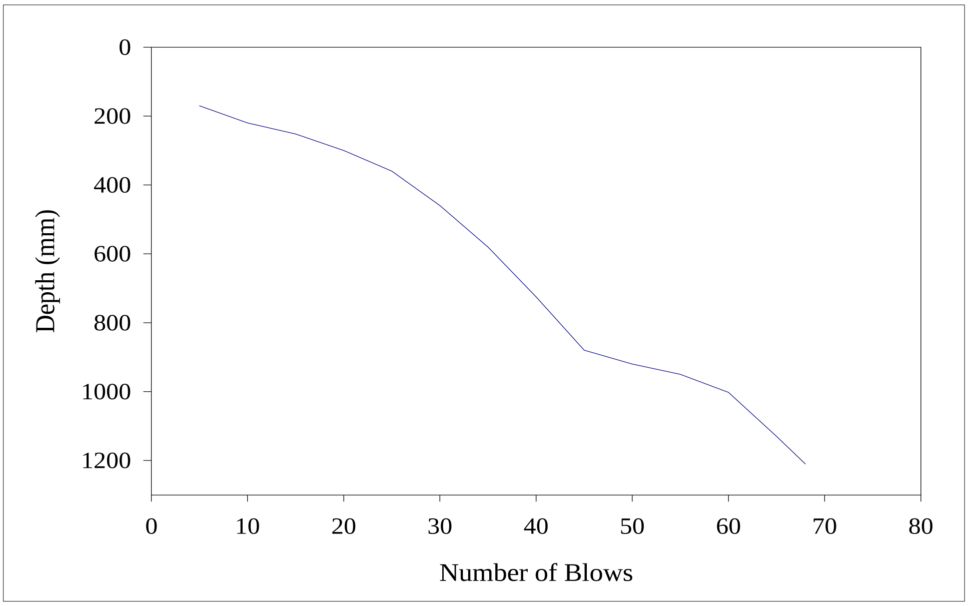

Figure D.3 Blow vs. depth graph for Core#3 at Withayawithee 3 Road.

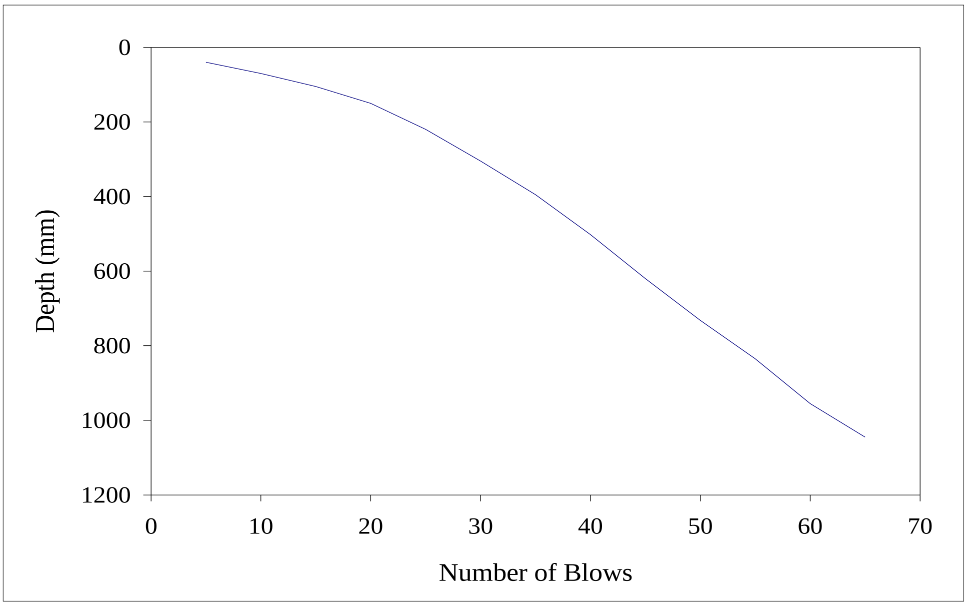

Figure D.4 Blow vs. depth graph for Core#4 at Withayawithee 3 Road.

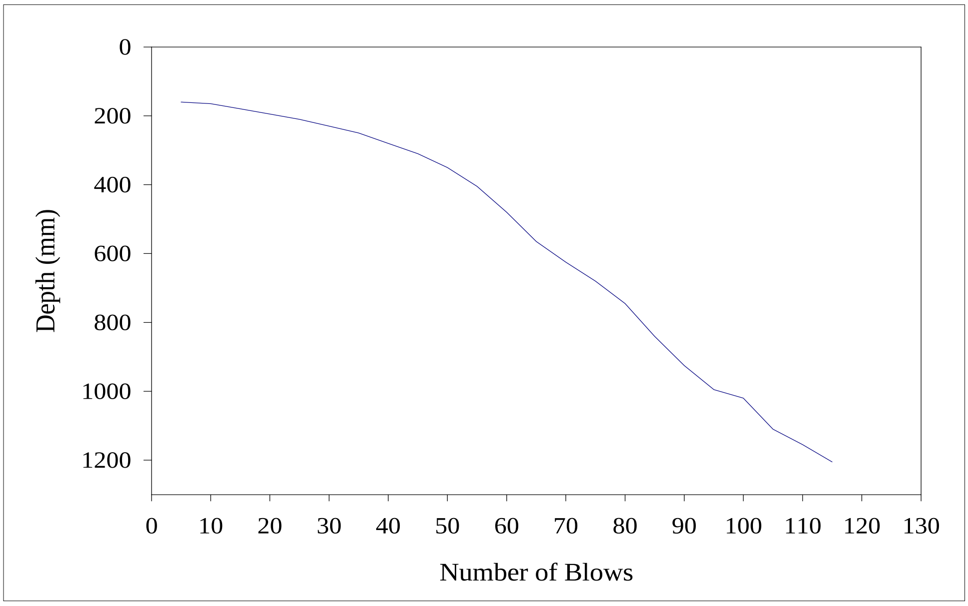

Figure D.5 Blow vs. depth graph for Core#5 at Withayawithee 2 Road.

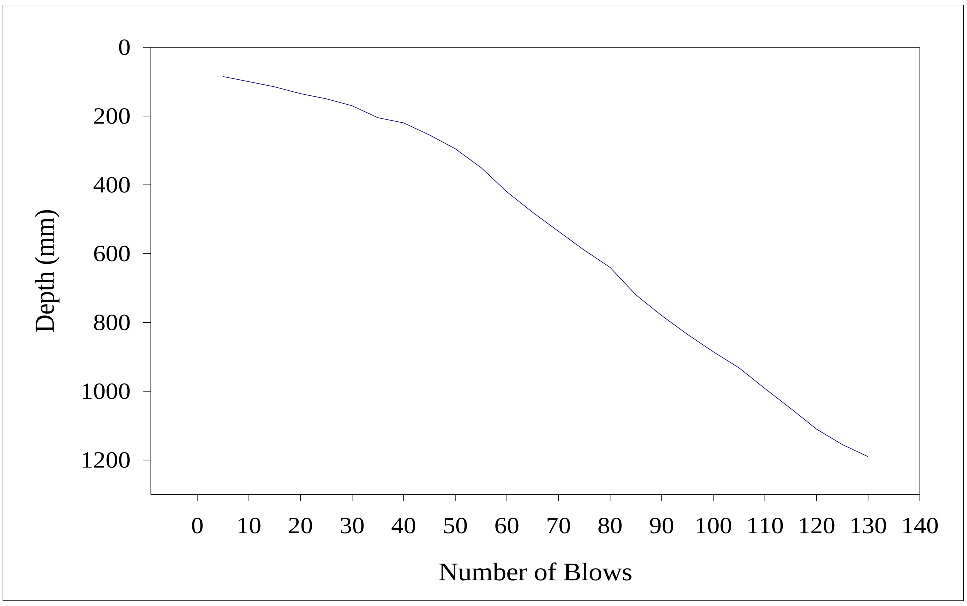

Figure D.6 Blow vs. depth graph for Core#6 at Withayawithee 2 Road.

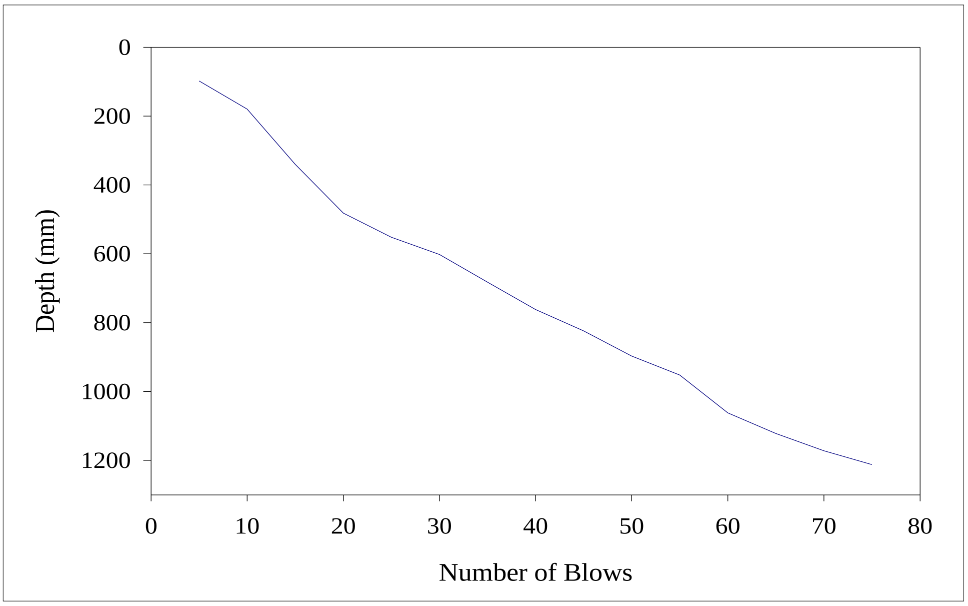

Figure D.7 Blow vs. depth graph for Core#7 at Withayawithee 2 Road.

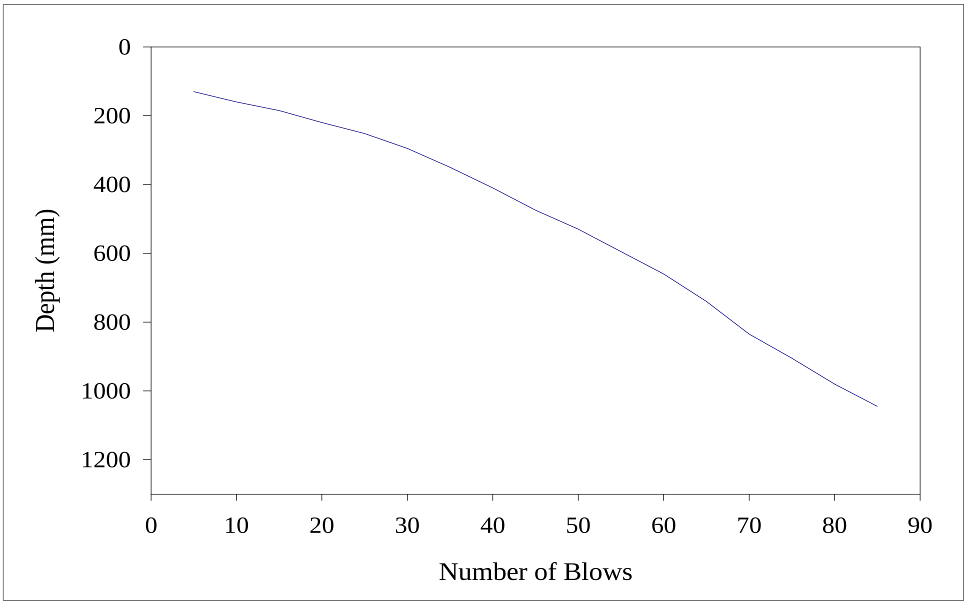

Figure D.8 Blow vs. depth graph for Core#8 at Withayawithee 2 Road.

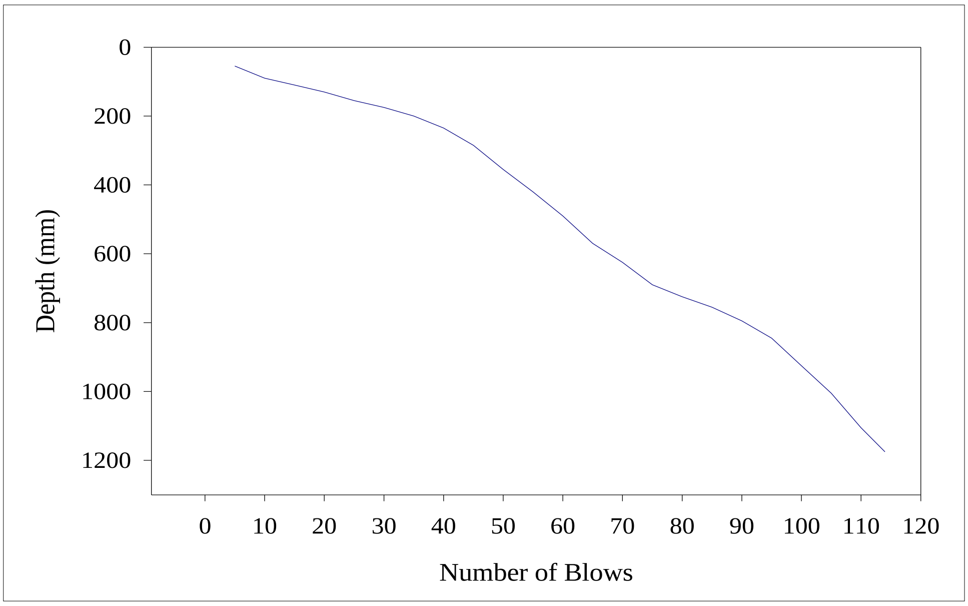

Figure D.9 Blow vs. depth graph for Core#9 at Withayawithee 2 Road.

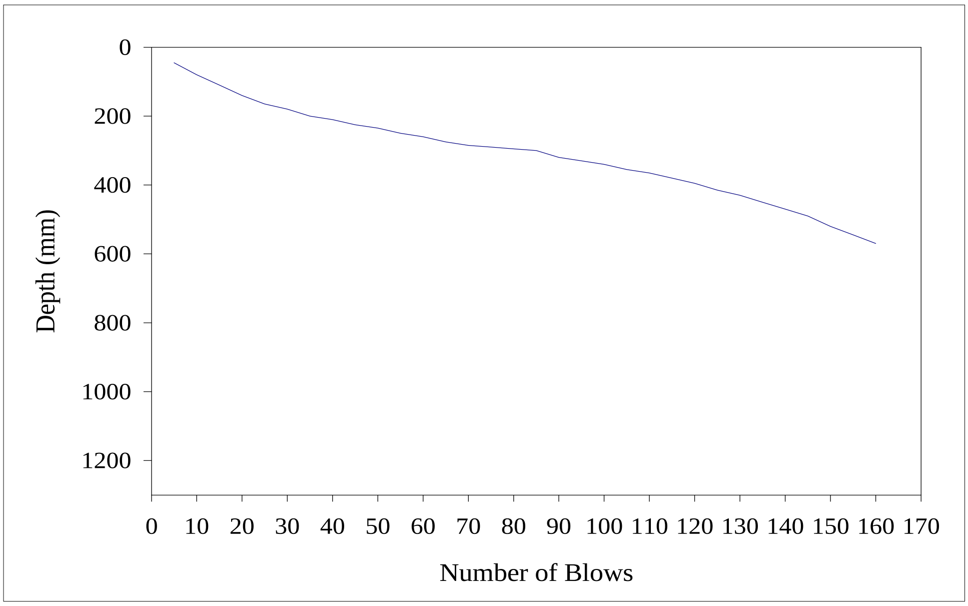

Figure D.10 Blow vs. depth graph for Core#10 at Withayawithee 2 Road.

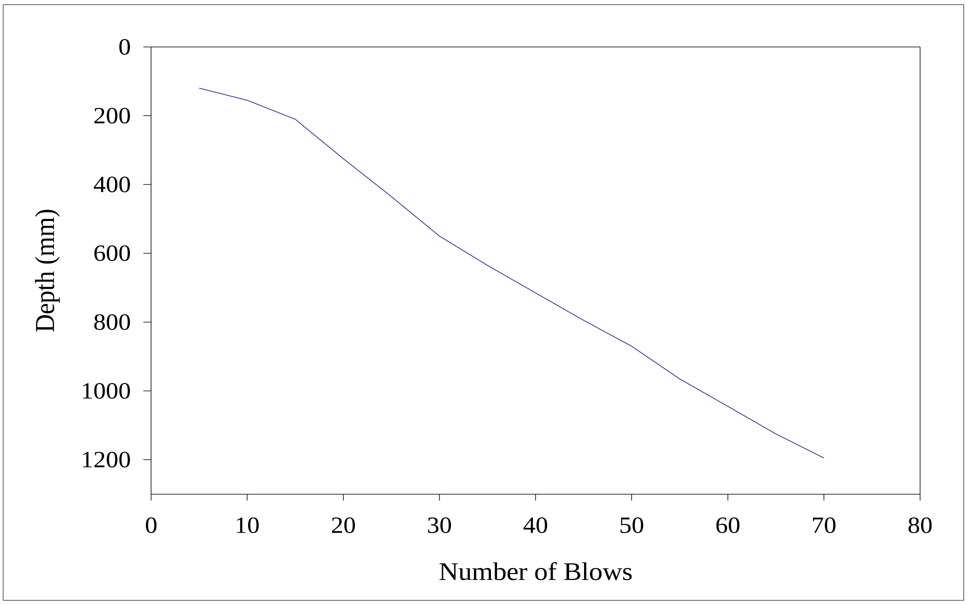

Figure D.11 Blow vs. depth graph for Core#11 at Sikkhawithee 1(a) Road.

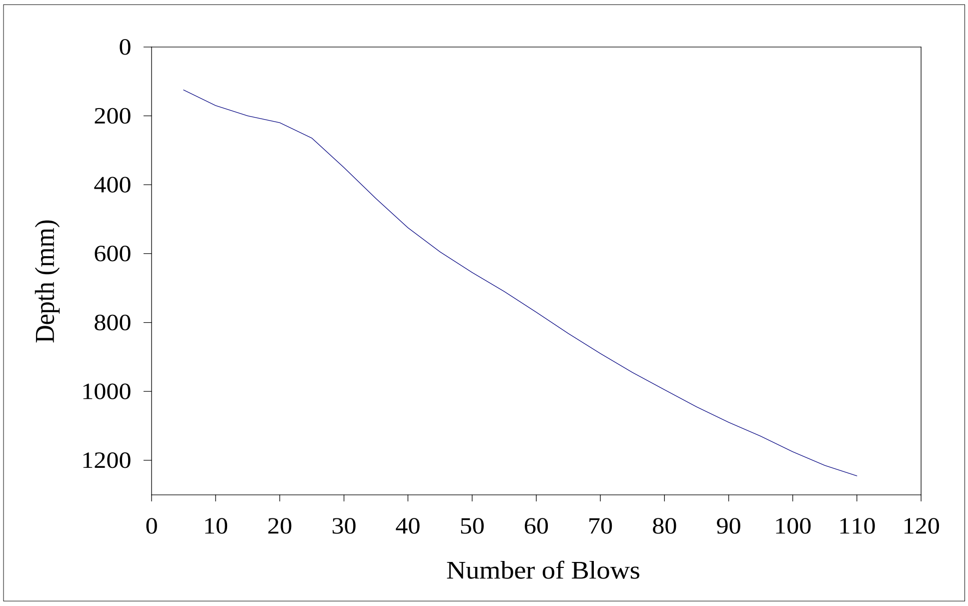

Figure D.12 Blow vs. depth graph for Core#12 at Sikkhawithee 1(a) Road.

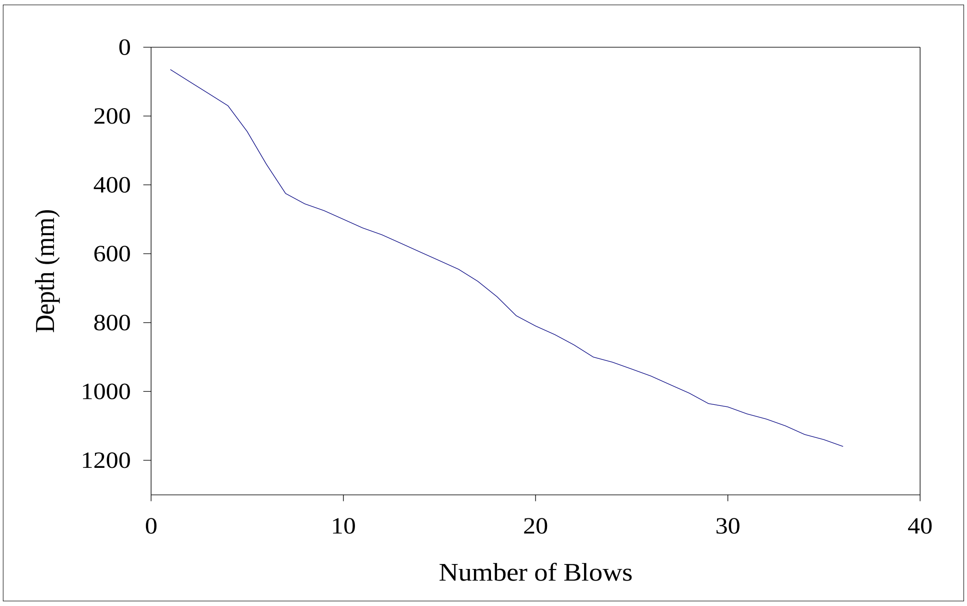

Figure D.13 Blow vs. depth graph for Core#13 at Sikkhawithee 1(a) Road.

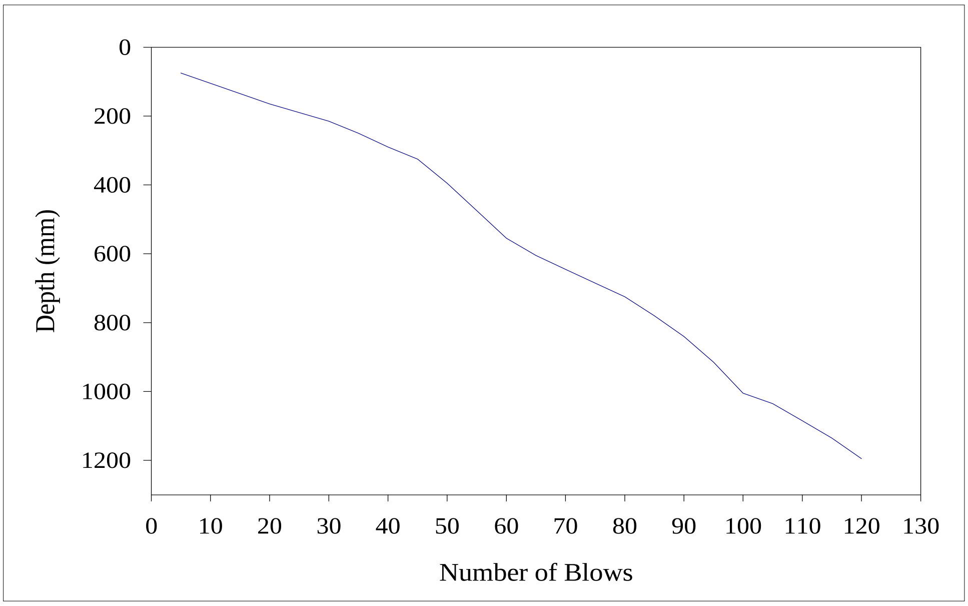

Figure D.14 Blow vs. depth graph for Core#14 at Sikkhawithee 1(b) Road.

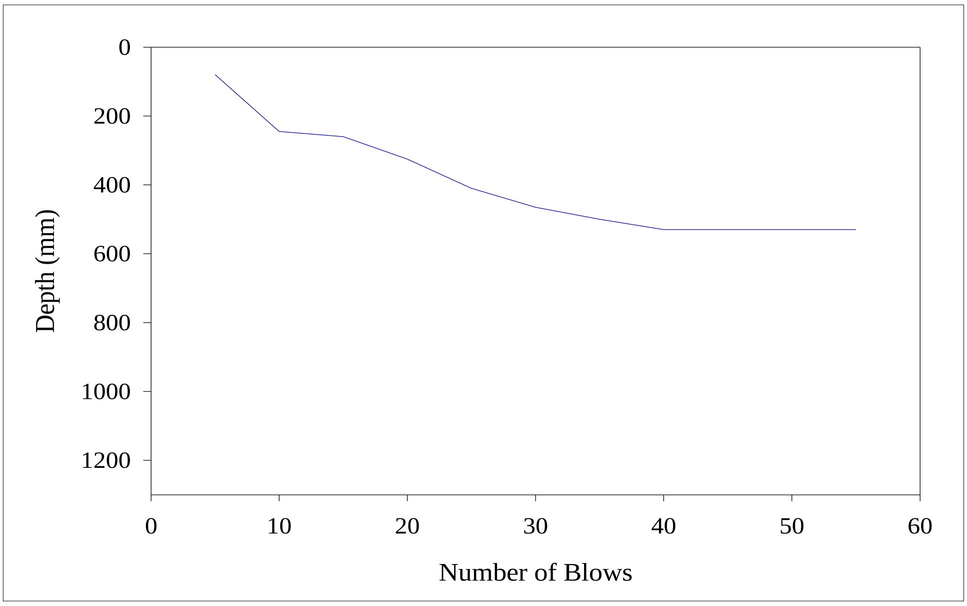

Figure D.15 Blow vs. depth graph for Core#15 at Sikkhawithee 1(b) Road.

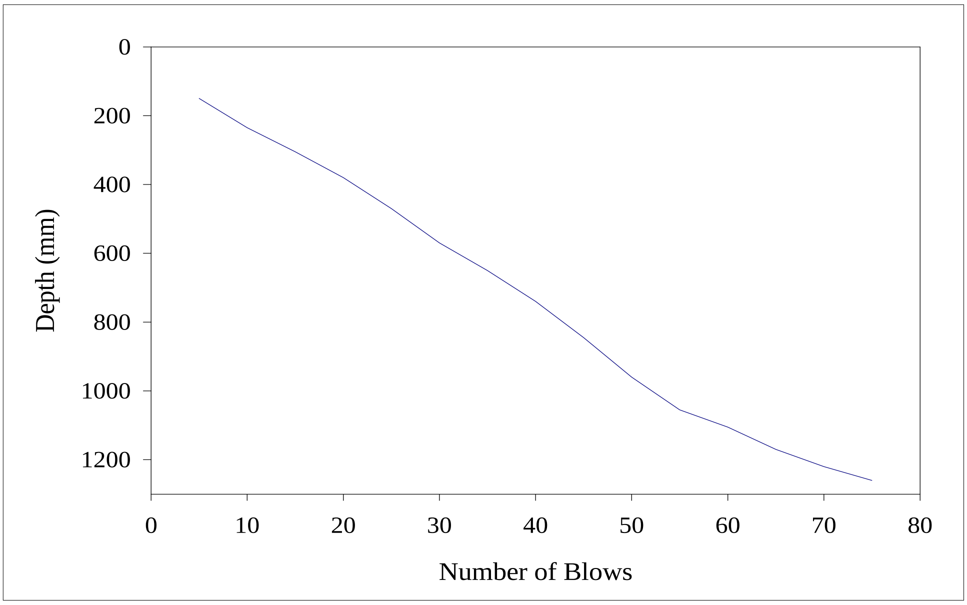

Figure D.16 Blow vs. depth graph for Core#16 at Sikkhawithee 1(b) Road.

2. Penetration Index vs. Depth

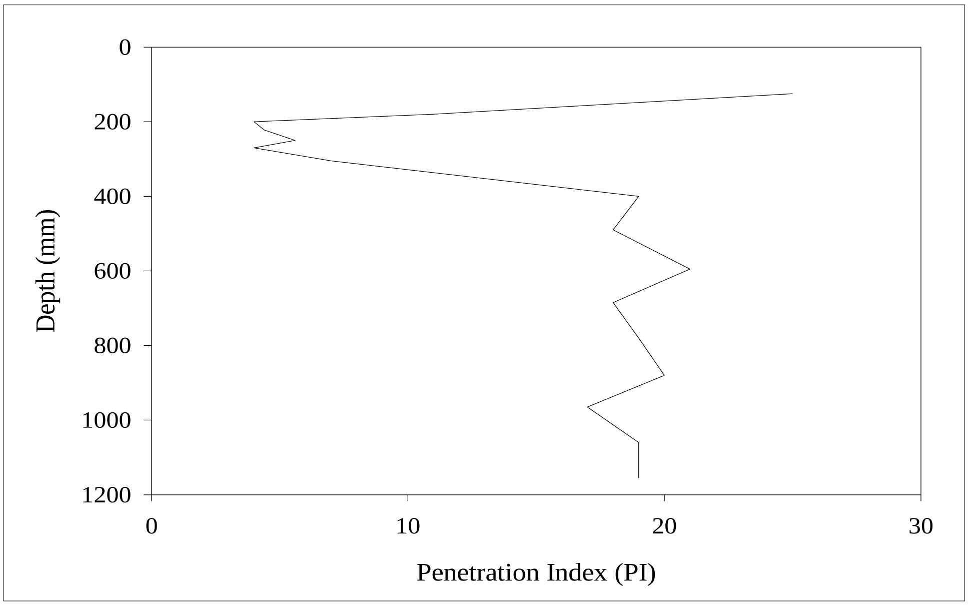

Figure D.17 PI vs. depth graph for Core#1 at Withayawithee 3 Road.

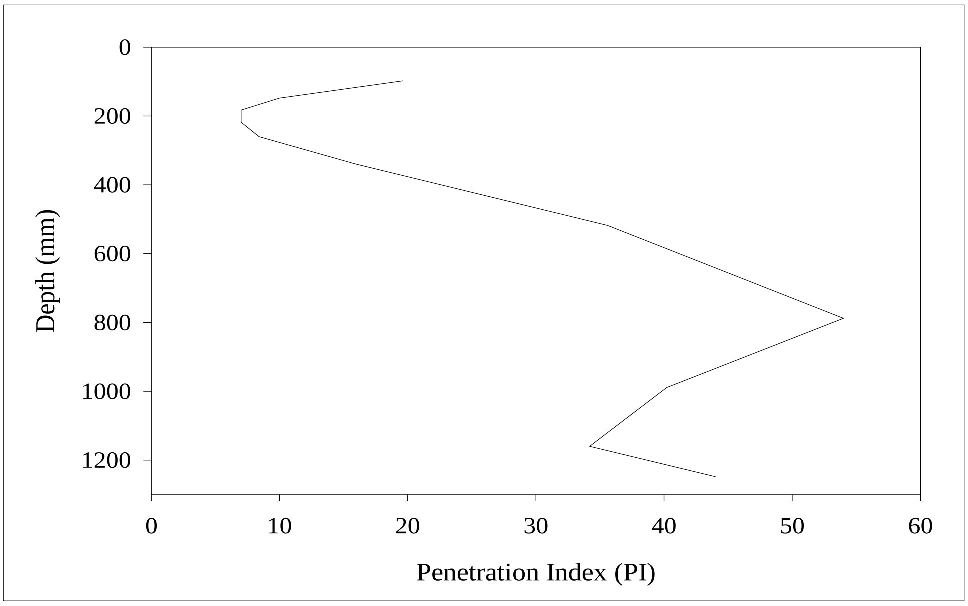

Figure D.18 PI vs. depth graph for Core#2 at Withayawithee 3 Road.

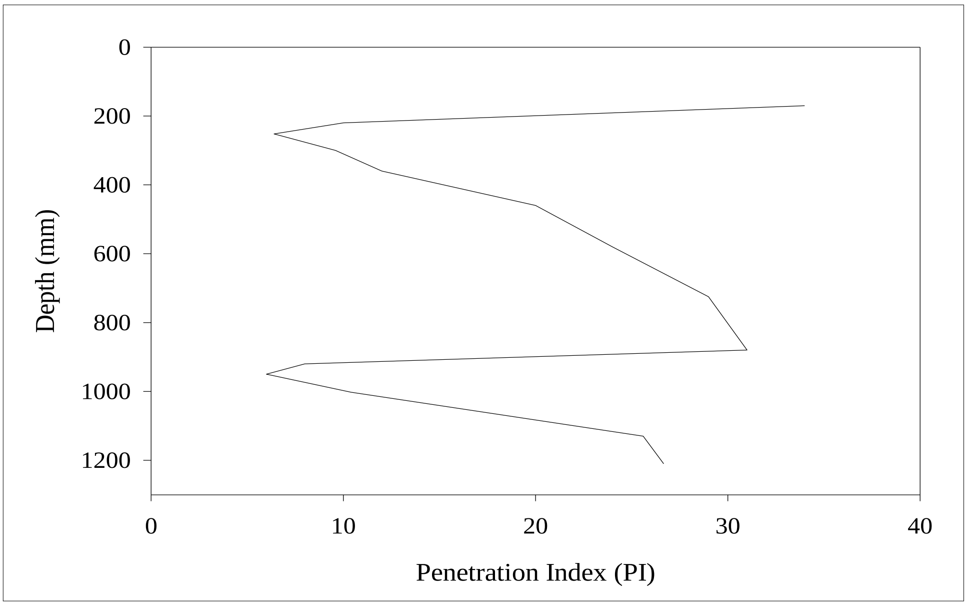

Figure D.19 PI vs. depth graph for Core#3 at Withayawithee 3 Road.

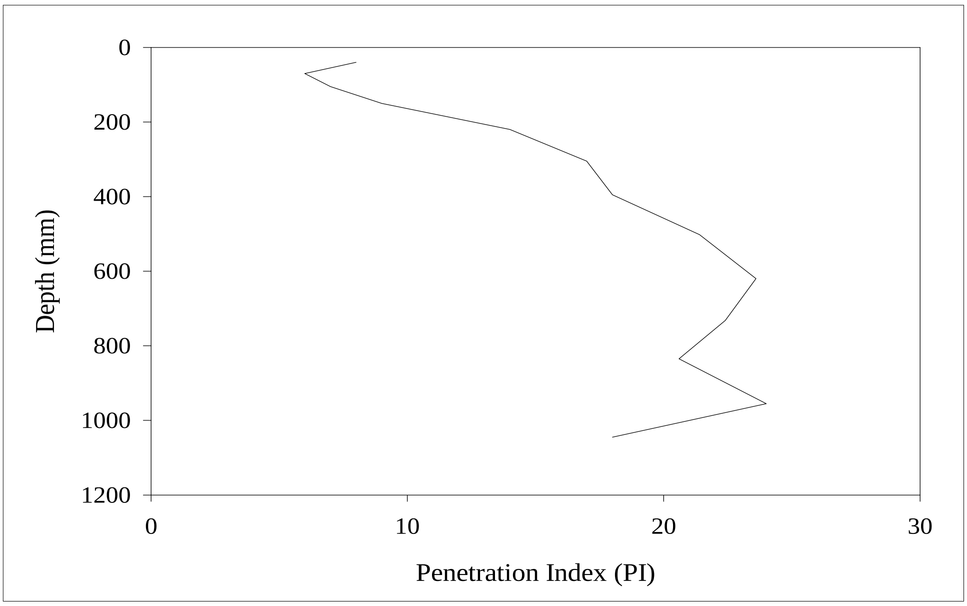

Figure D.20 PI vs. depth graph for Core#4 at Withayawithee 3 Road.

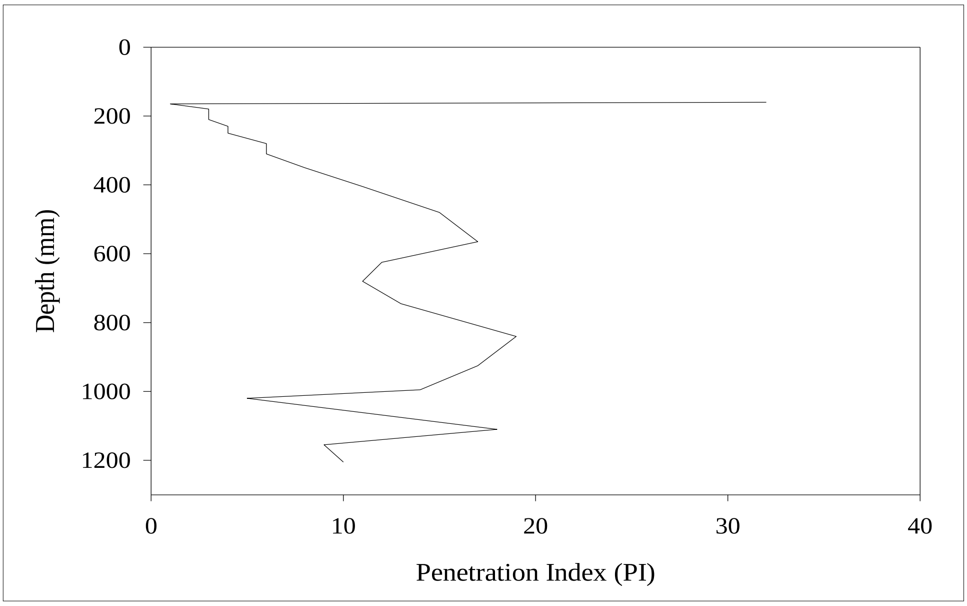

Figure D.21 PI vs. depth graph for Core#5 at Withayawithee 2 Road.

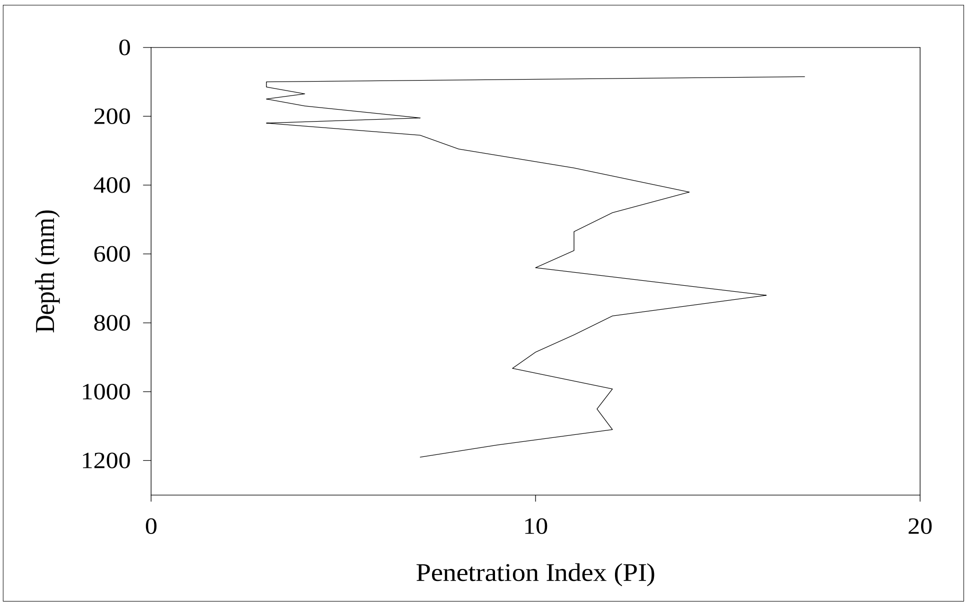

Figure D.22 PI vs. depth graph for Core#6 at Withayawithee 2 Road.

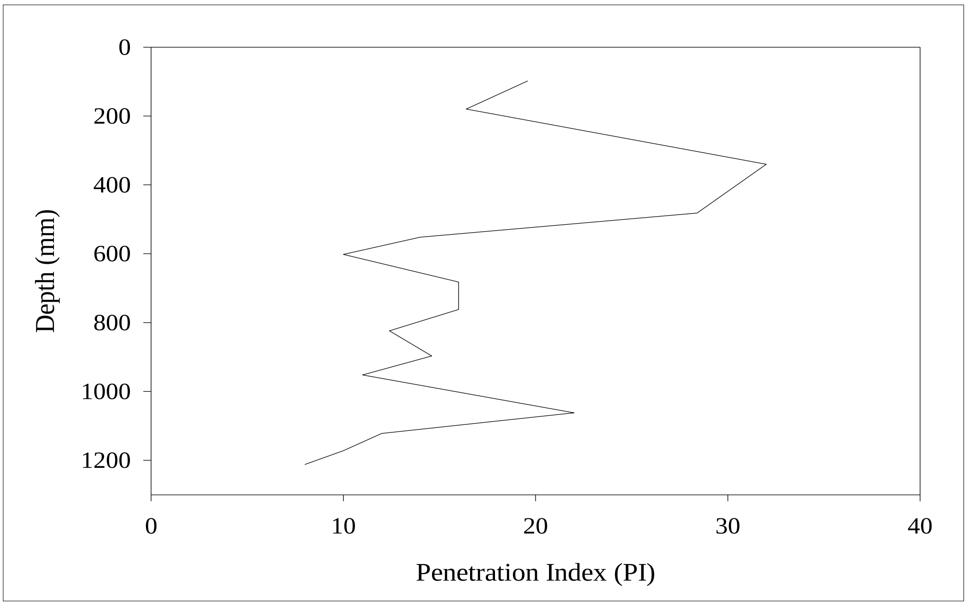

Figure D.23 PI vs. depth graph for Core#7 at Withayawithee 2 Road.

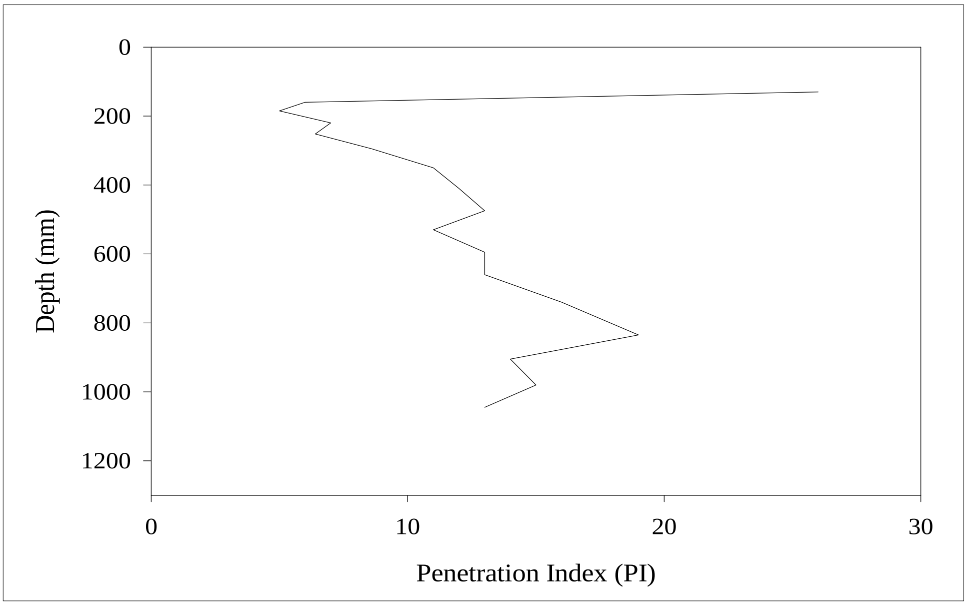

Figure D.24 PI vs. depth graph for Core#8 at Withayawithee 2 Road.

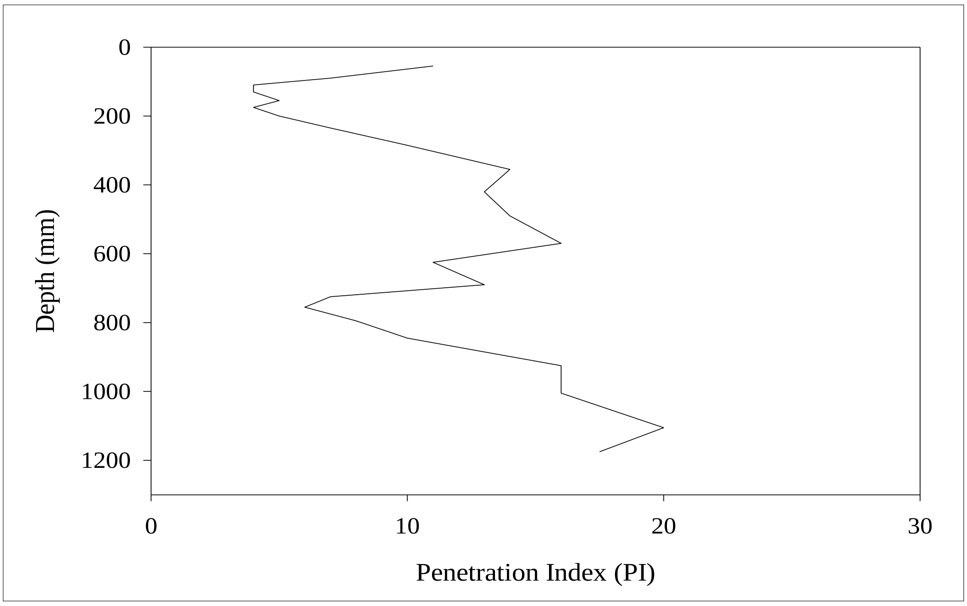

Figure D.25 PI vs. depth graph for Core#9 at Withayawithee 2 Road.

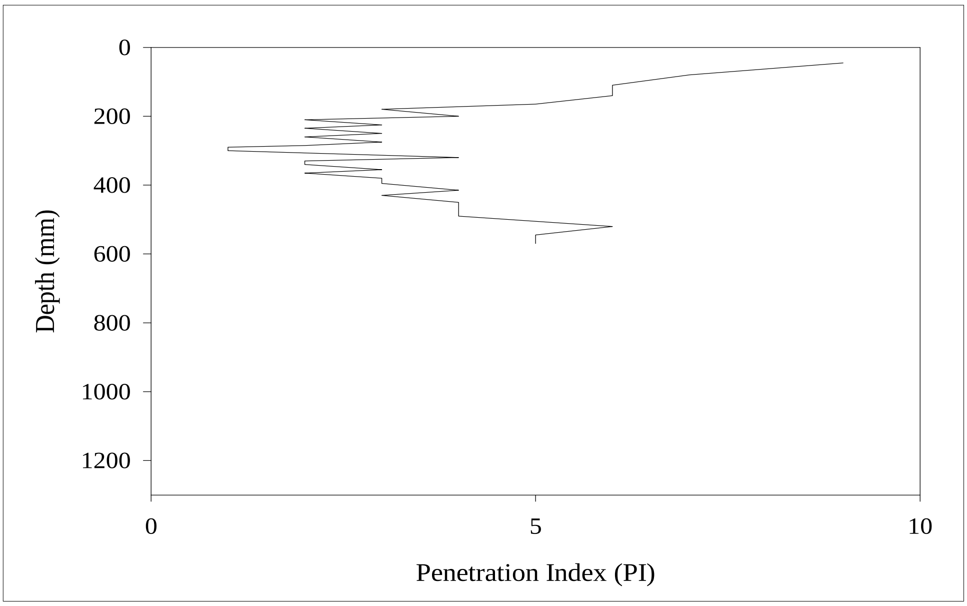

Figure D.26 PI vs. depth graph for Core#10 at Withayawithee 2 Road.

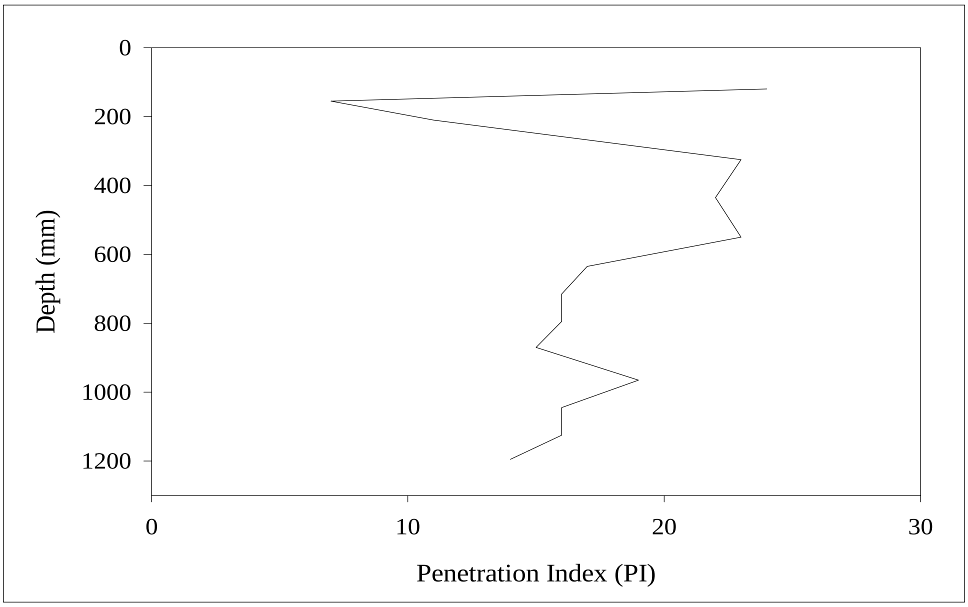

Figure D.27 PI vs. depth graph for Core#11 at Sikkhawithee 1(a) Road.

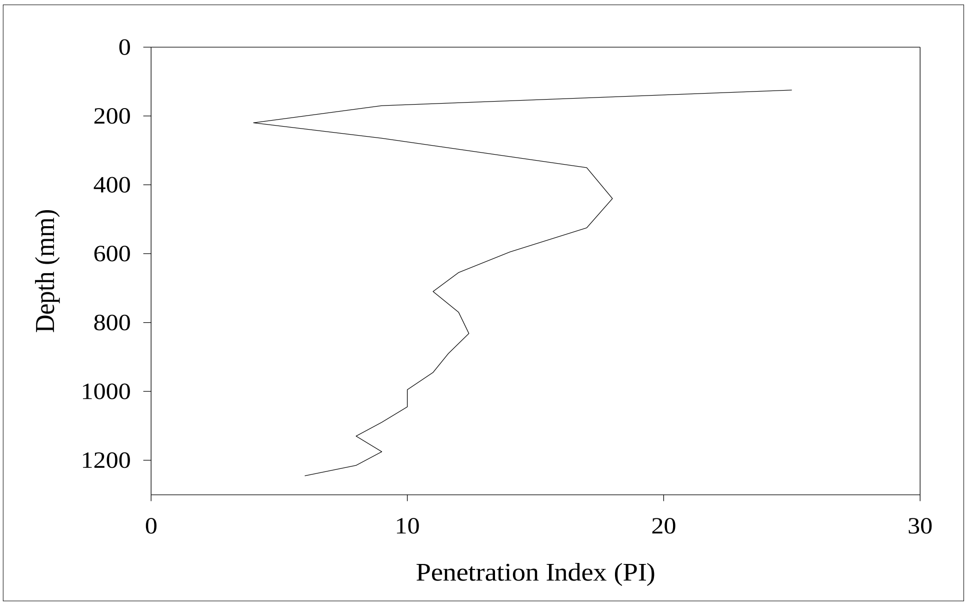

Figure D.28 PI vs. depth graph for Core#12 at Sikkhawithee 1(a) Road.

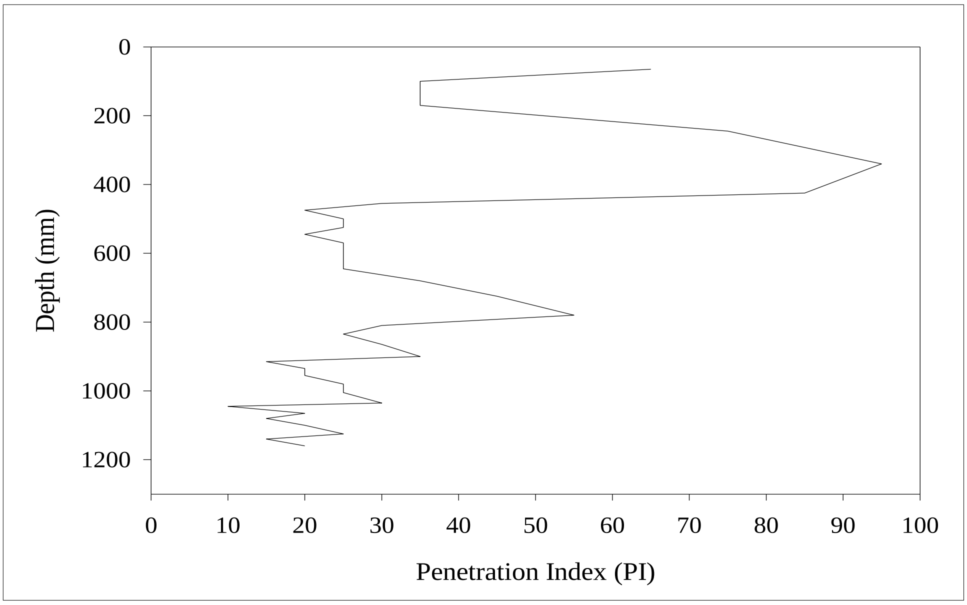

Figure D.29 PI vs. depth graph for Core#13 at Sikkhawithee 1(a) Road.

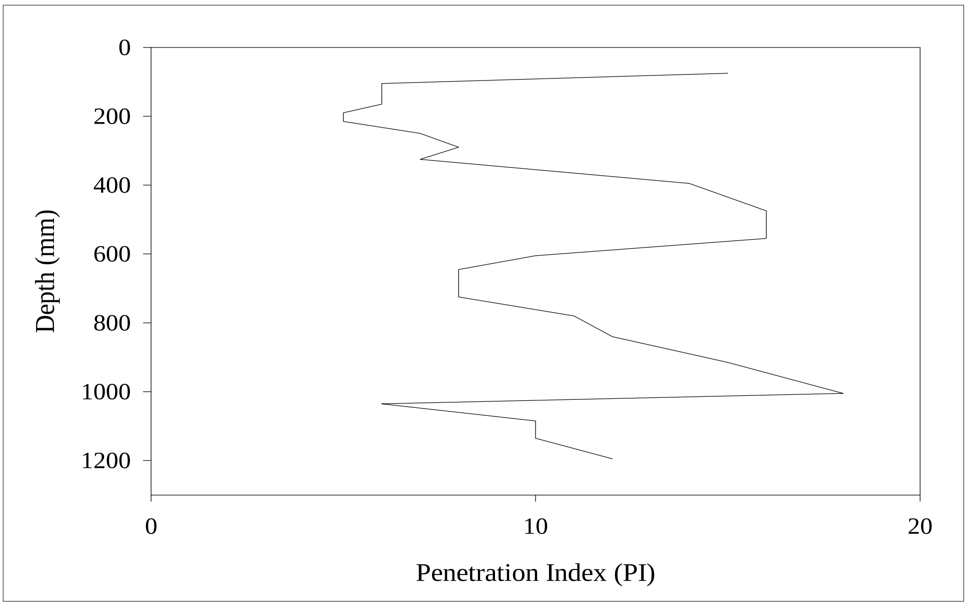

Figure D.30 PI vs. depth graph for Core#14 at Sikkhawithee 1(b) Road.

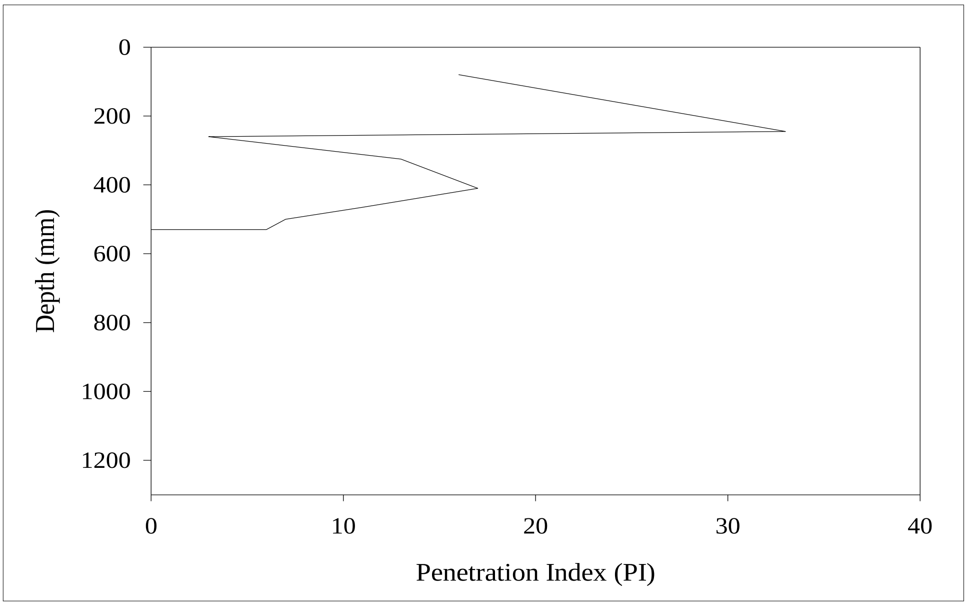

Figure D. 31 PI vs. depth graph for Core#15 at Sikkhawithee 1(a) Road.

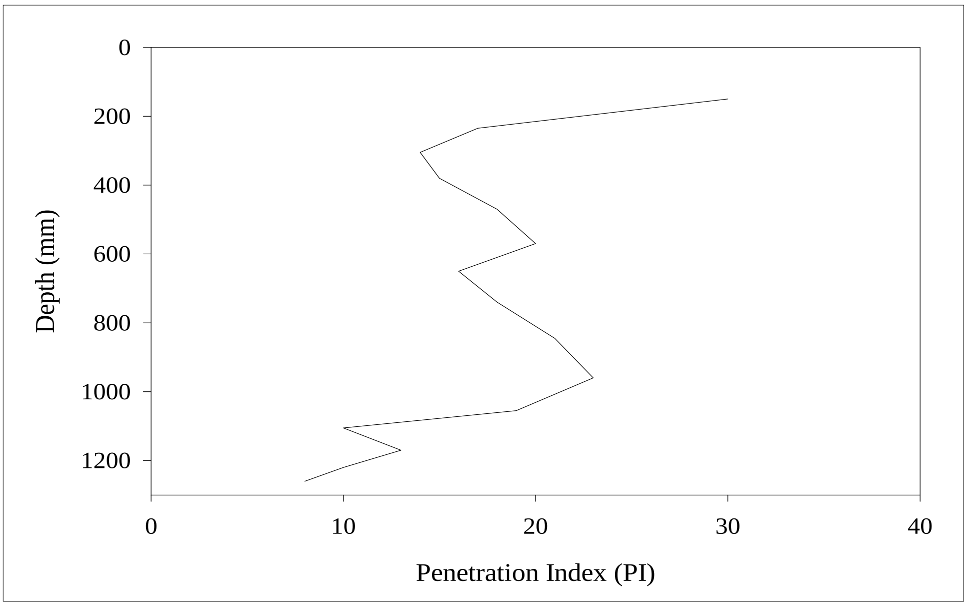

Figure D.32 PI vs. depth graph for Core#16 at Sikkhawithee 1(b) Road.

3. Modulus Vs Depth

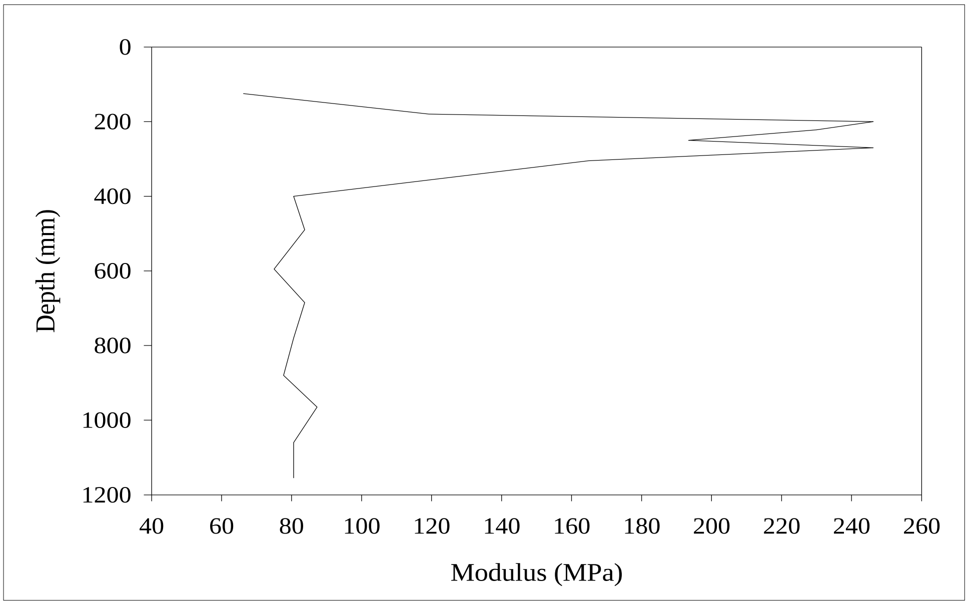

Figure D.33 Modulus of subgrade reaction from Core#1 at Withayawithee 3 Road.

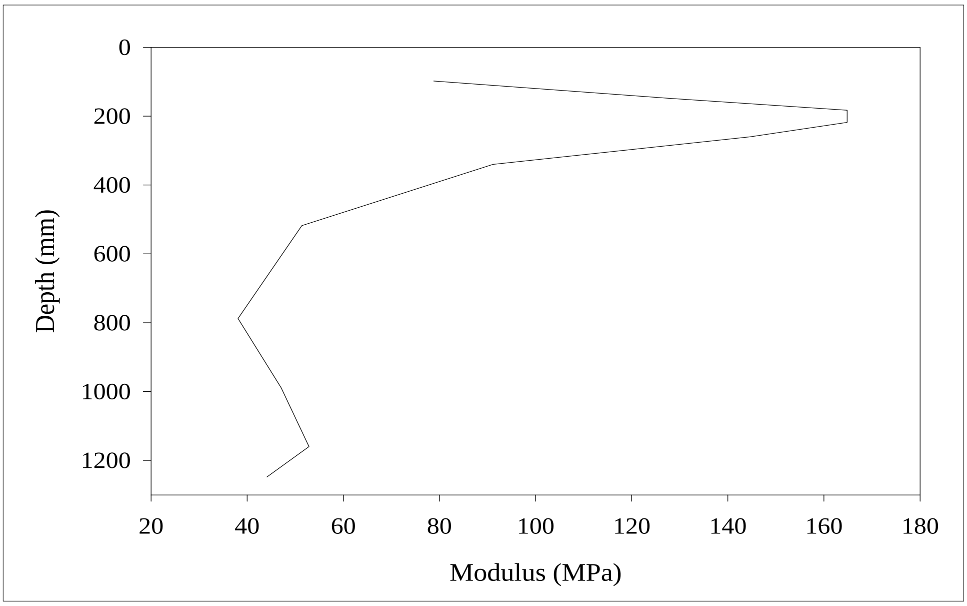

Figure D.34 Modulus of subgrade reaction from Core#2 at Withayawithee 3 Road.

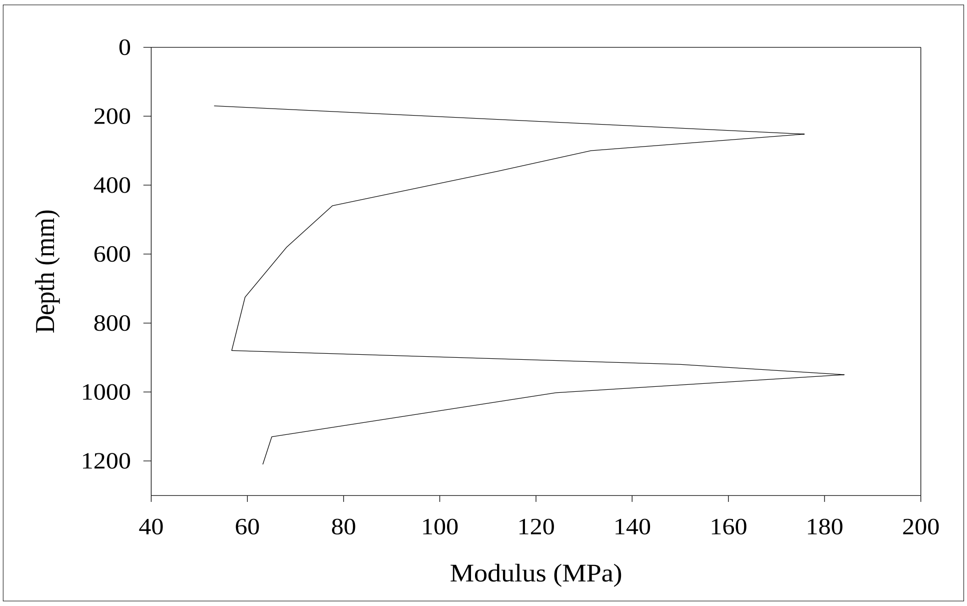

Figure D.35 Modulus of subgrade reaction from Core#3 at Withayawithee 3 Road.

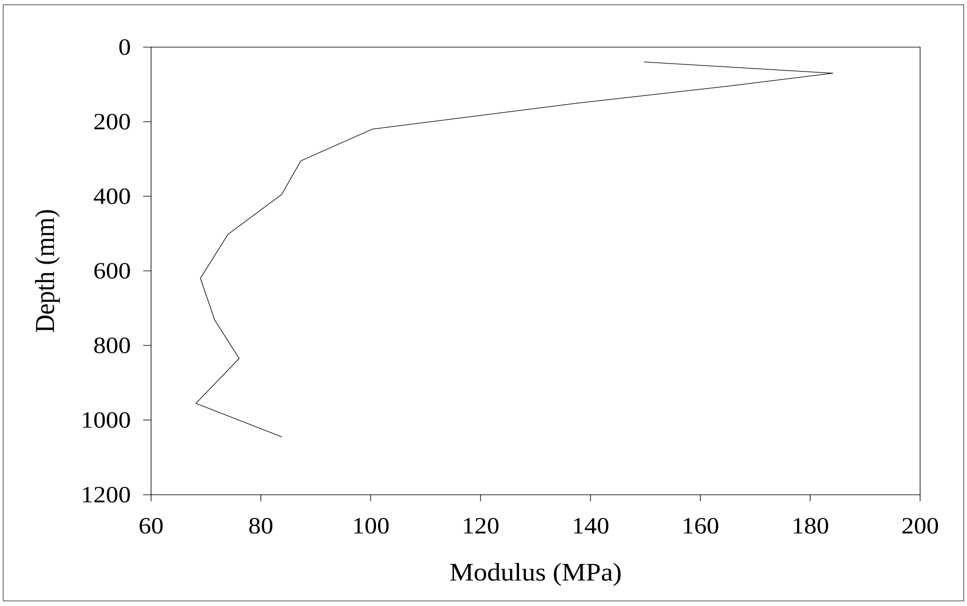

Figure D.36 Modulus of subgrade reaction from Core#4 at Withayawithee 3 Road.

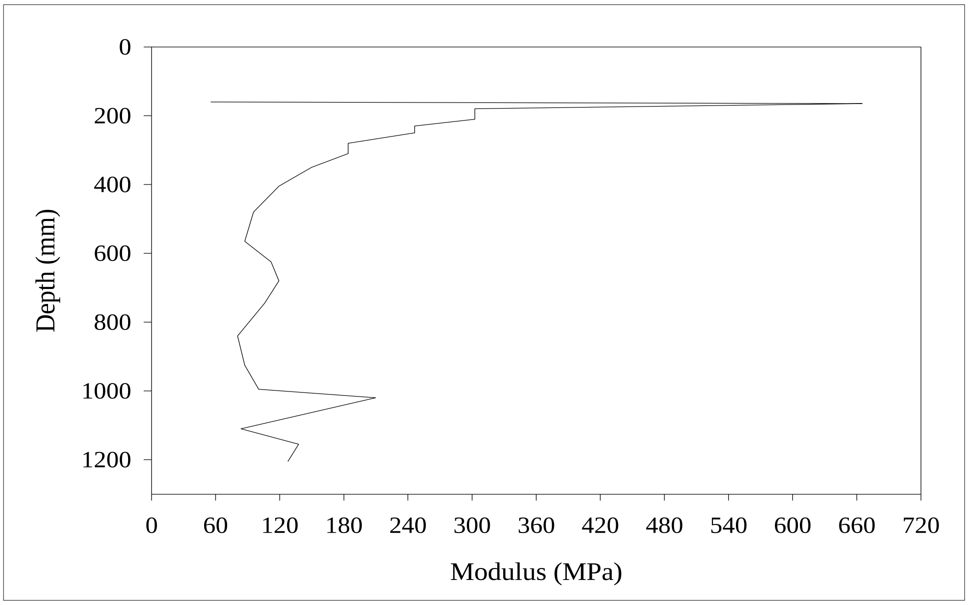

Figure D.37 Modulus of subgrade reaction from Core#5 at Withayawithee 2 Road.

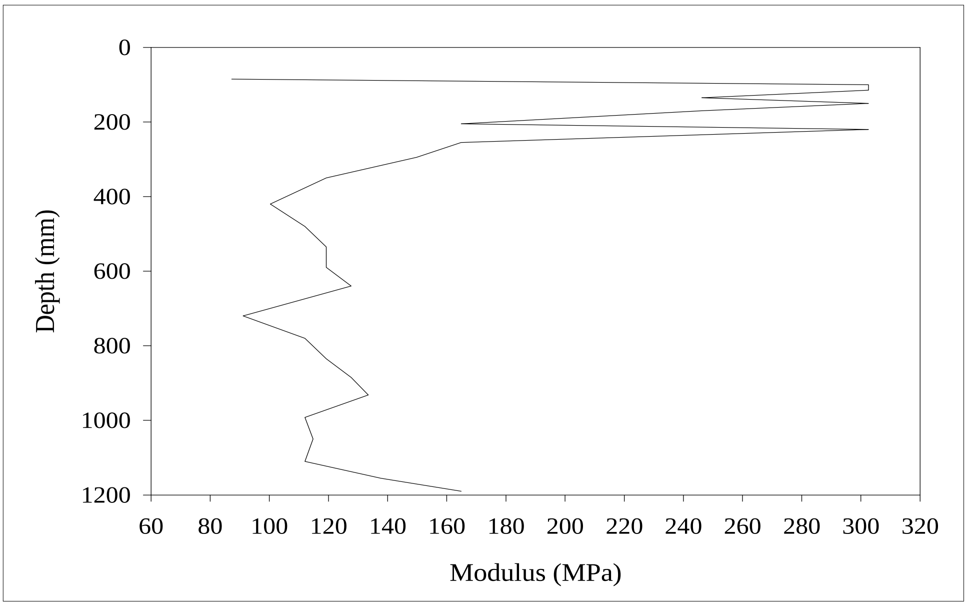

Figure D.38 Modulus of subgrade reaction from Core#6 at Withayawithee 2 Road.

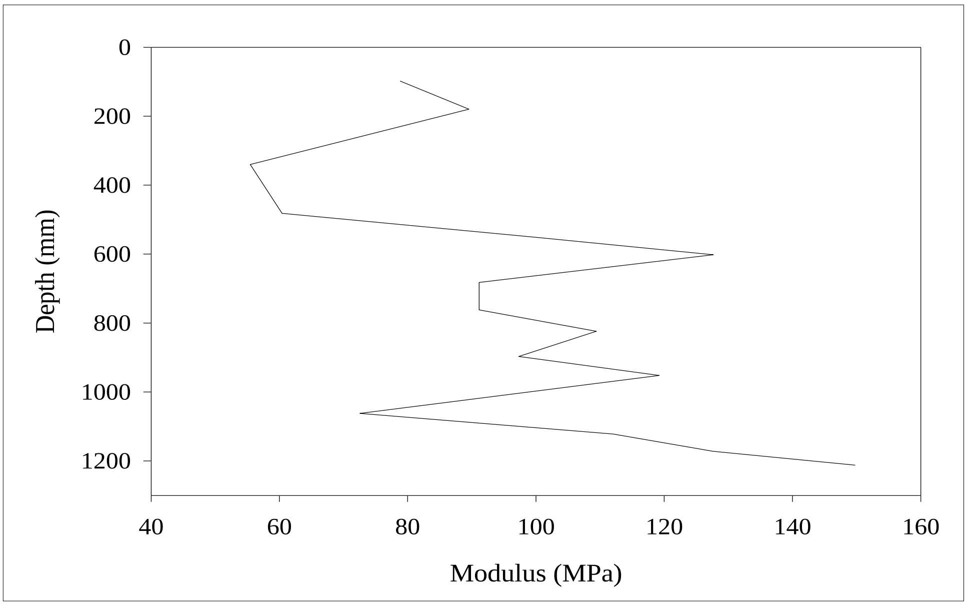

Figure D.39 Modulus of subgrade reaction from Core#7 at Withayawithee 2 Road.

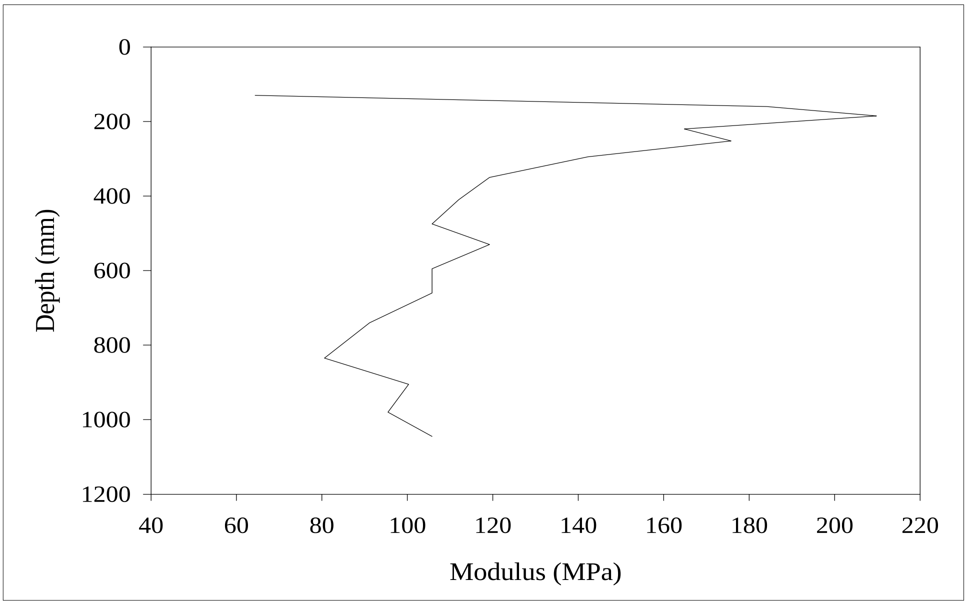

Figure D.40 Modulus of subgrade reaction from Core#8 at Withayawithee 2 Road.

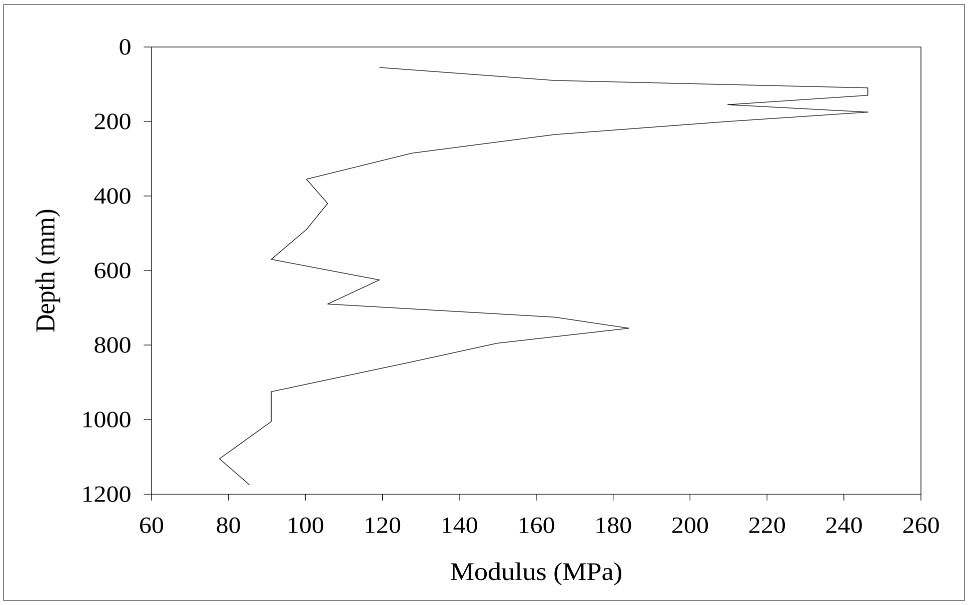

Figure D.41 Modulus of subgrade reaction from Core#9 at Withayawithee 2 Road.

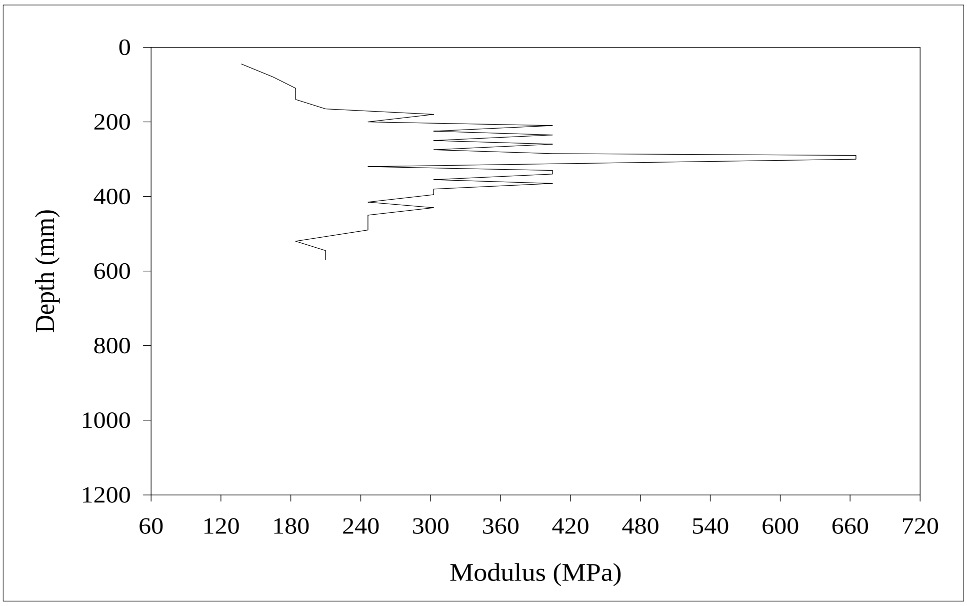

Figure D.42 Modulus of subgrade reaction from Core#10 at Withayawithee 2 Road.

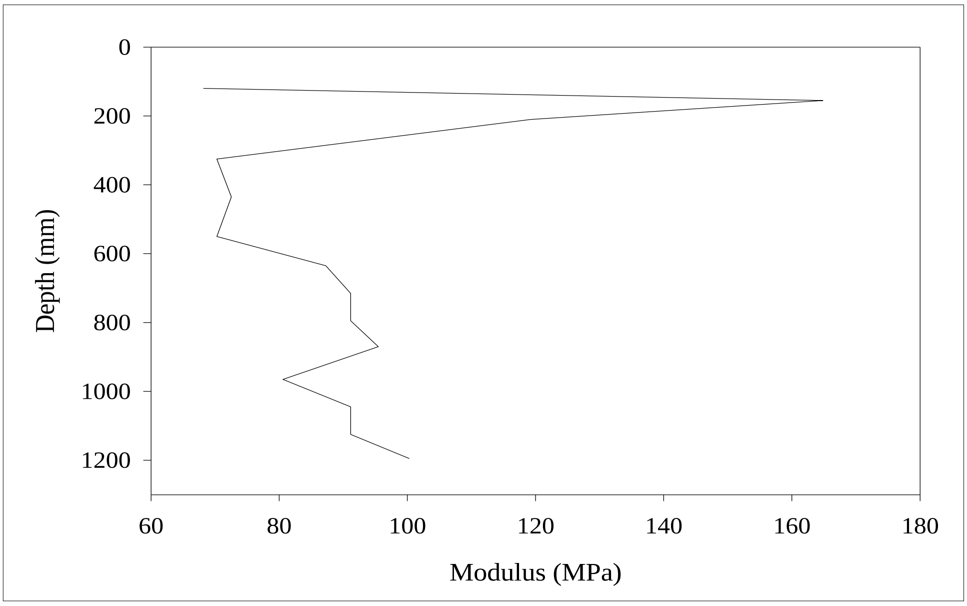

Figure D.43 Modulus of subgrade reaction from Core#11 at Sikkhawithee 1(a) Road.

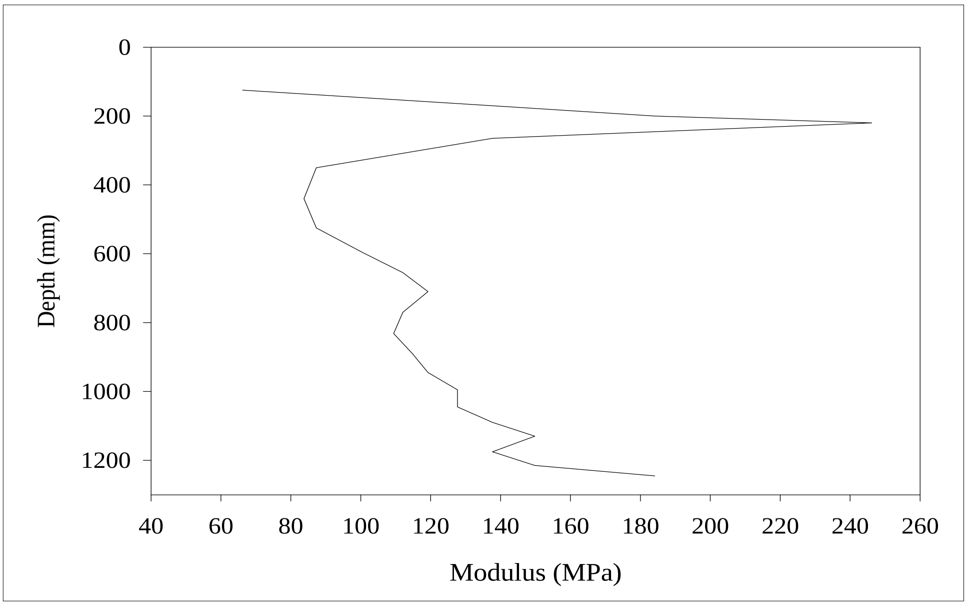

Figure D.44 Modulus of subgrade reaction from Core#12 at Sikkhawithee 1(a) Road.

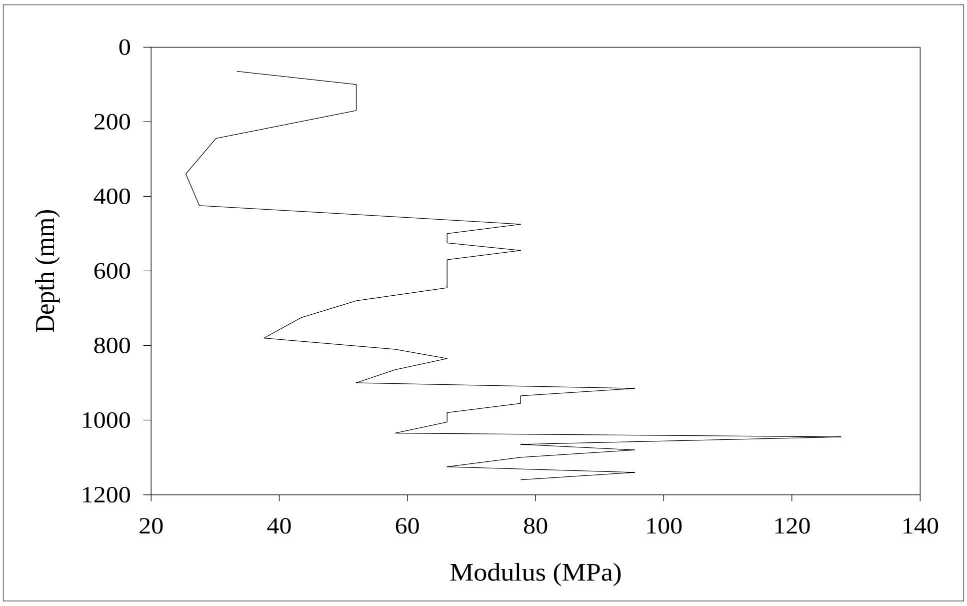

Figure D.45 Modulus of subgrade reaction from Core#13 at Sikkhawithee 1(a) Road.

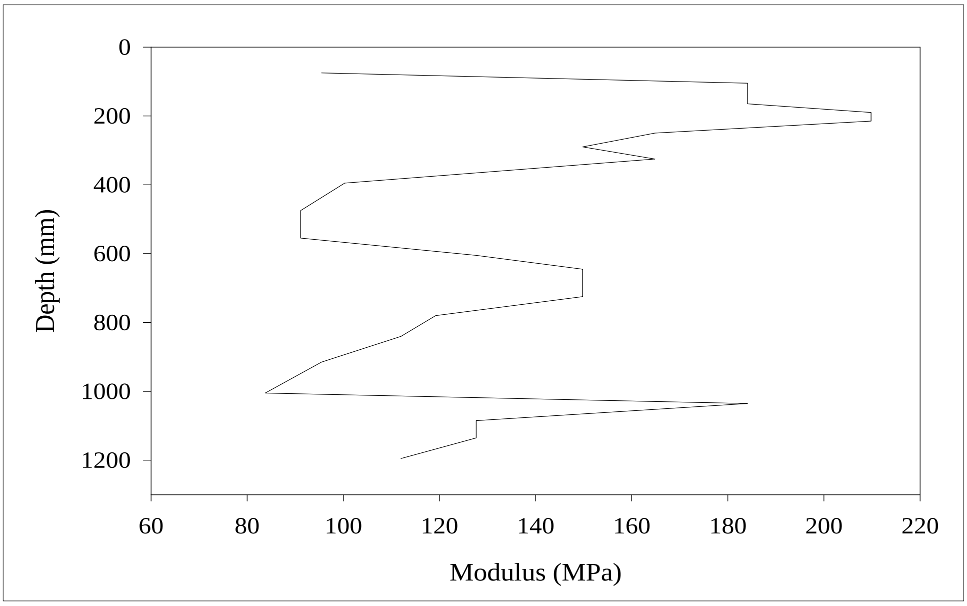

Figure D.46 Modulus of subgrade reaction from Core#14 at Sikkhawithee 1(b) Road.

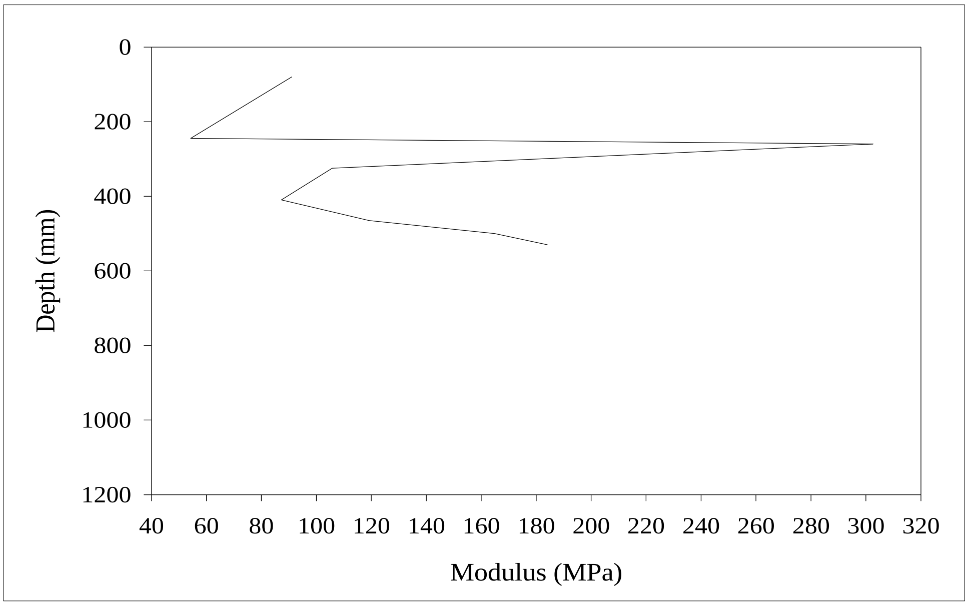

Figure D.47 Modulus of subgrade reaction from Core#15 at Sikkhawithee 1(b) Road.

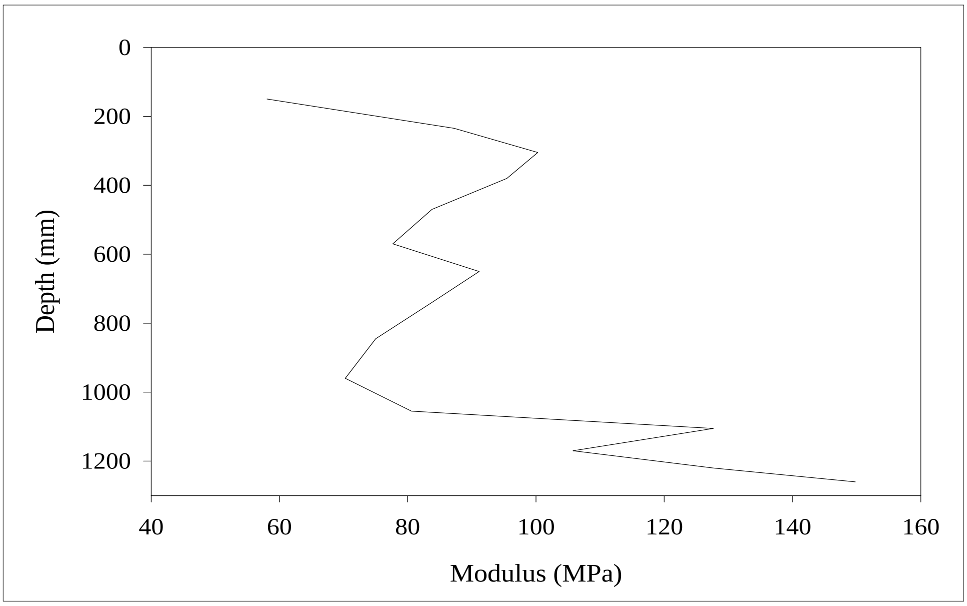

Figure D.48 Modulus of subgrade reaction from Core#16 at Sikkhawithee 1(b) Road.

APPENDIX E

TECHNICAL PUBLICATION

Pasang, S., Limanond, T. and Tanseng, P. (2010). **Evaluation of Polyurethane Foam Injection for Rigid Pavement Rehabilitation**. In Proceeding of the 4th Ubon Ratchathani University Research Conference, 9th-10th August 2010.

การประเมินประสิทธิภาพของการฉีดพอลียูรีเทนโฟมเพื่อซ่อมบำรุง ควรใช้ ผิวทางคอนกรีต

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Evaluation of Polyurethane Foam Injection for Rigid Pavement Rehabilitation

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บทคัดย่อ

การศึกษานี้มีวัตถุประสงค์เพื่อตรวจสอบประสิทธิภาพของการฉีดพอลิยูรีเทนโฟม (PFI) เพื่อซ่อมบำรุงถนนคอนกรีตในบริเวณที่ทรุดตัว โดยใช้กรณีศึกษาการซ่อมแซมถนนคอนกรีตสี่ช่วงถนนในมหาวิทยาลัยเทคโนโลยีสุรนารี จังหวัดนครราชสีมา ผู้วิจัยได้ประเมินประสิทธิภาพและความเหมาะสมของวิธีการซ่อมบำรุง โดยตรวจสอบการยุบตัวของถนนภายหลังฉีดพอลิยูรีเทนโฟมแล้ว หกเดือน พบว่ากระบวนการ PFI นี้ สามารถช่วยยกระดับและปรับปรุงคุณภาพของถนนได้ ใช้เวลาดำเนินการน้อยกว่าวิธีดั้งเดิม ช่วยลดระยะเวลาการปิดช่องจราจรเพื่อการซ่อมบำรุง อย่างไรก็ตามการตรวจสอบอย่างต่อเนื่องพบว่าบริเวณที่ฉีดพอลียูรีเทนบางช่วงถนนยุบตัวเนื่องจากพื้นดินใต้ถนนไม่แข็งแรงและถูกชะล้าง นอกจากนี้ยังสังเกตุเห็นรอยแตกใหม่ ๆ และความไม่สม่ำเสมอของผิวถนนซึ่งอาจเกิดเนื่องจากการฉีดพอลิยูรีเทนโฟมไม่สม่ำเสมอ

คำสำคัญ: การฉีดพอลิยูรีเทนโฟม การซ่อมบำรุง

Abstract

The objective of this study is to evaluate the performance of polyurethane foam injection (PFI) to rehabilitate distressed rigid pavements using a case study at four project sites in Suranaree University of Technology (SUT), Nakhon Ratchasima. The study evaluated the performance and suitability of the rehabilitation method by monitoring the differential settlement at the rehabilitated pavement sections for a period of six months. It was observed that the PFI process has been successfully lifted and aligned the pavement at the desired level, and improved the ride quality. Moreover, the process was less time consuming than conventional method of slab jacking, which minimized the disruption to the traffic flow. However, the continuous monitoring indicated some settlements at the polyurethane injected pavement sections, which was possibly caused by the weaker subgrade and scouring of the base material. In addition, several new cracks and uneven slabs settlements were observed at those sections, which could be because of uneven support created while injecting polyurethane foam beneath the pavement sections.

Keywords: Polyurethane Foam Injection, Pavement Rehabilitation

Introduction

A rigid pavement road of 125 m length in four different roads at SUT had undergone a repair treatment of polyurethane foam injection to restore the condition of concrete. The pavements in these locations were showing significant distresses due to uneven slab settlements caused by loss of subgrade which resulted to faulting, slab cracking and poor ride quality. SUT opted for PFI method, a technique relatively new in Thailand, to slab jacking the distressed pavements.

The PFI used to fill voids and level the CRCP and bridge approach slabs proved that injecting polyurethane foam into the pavement structure was an effective method of leveling CRCP and bridge approach slabs which were still in good condition after four years, and no additional slab faulting has occurred (Gaspard & Morvant, 2004).

PFI was tried in many places for correcting the pavement settlement and alignment problems with minimal disruption or downtime. It was able to raise the slabs to the target levels, however, in some projects they were not able to maintain the target elevation or prevent slabs from further settlement. New cracks formed in some slabs that had previously received PFI (Chen et al., 2008). Polyurethane foam injection worked on most projects in raising slabs to the target levels; however, the long-term effectiveness in keeping slabs at target levels is questionable (Chen et al., 2008). Polyurethane foam gave better result on pavement with dense-graded aggregate bases than those with open-graded bases (Opland & Barnhart, 1995).

Polyurethane foam did improve the base support where the pavements were cracked, but it raised the pavement and provided a temporary increase in base stability where the pavement was severely faulted at Monroe County. In addition, an initial improvement in ride quality was also seen.

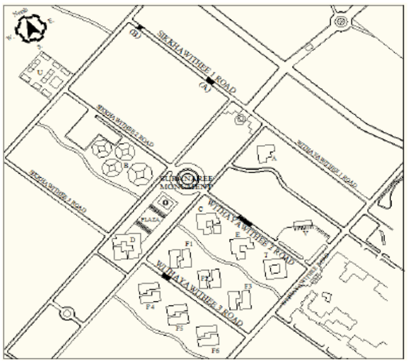

Figure 1: Site locations of PFI repaired roads

This study would evaluate the road injected with polyurethane foam by monitoring the settlement and visually. It would also assess the pavement condition through checking the profile levels, structure and the ride quality and ultimately recommending on the repair treatment. The following are the road pavements locations within SUT which have undergone polyurethane injection in August of 2009 (Figure 1).

1. Road Sikkhawithee 1(a) Road- 27.5m of the road
2. Road Sikkhawithee 1(b) Road- 12.5m of the road
3. Road withayawithee 2 Road- 55m of the road
4. Road withayawithee 3 Road- 30m of the road

Methodology

2.1 Instrumentation

A survey leveling equipment was used to determine the variation of profile level of the pavement surface. The entire four polyurethane foam injected pavements were surveyed; the readings were taken from the same bench mark and on the same control points on pavement throughout the study period of six months.

Sixteen cores were extracted to evaluate the condition of the concrete and tie bars, and further to see the thickness and distribution of the polyurethane foam beneath the pavement. At the cored areas, DCP test was conducted to evaluate the condition of base and subgrade.

2.2 Data Collection

Survey control points were marked at both edges and the middle of the road at 2.5 m interval. The road profile levels were taken before and after polyurethane foam injection to determine the lift provided by the foam. To monitor the levels, profile levels were also taken after one, two, four and six months while all the data were complied for analysis.

A wet coring machine was used to extract concrete cores from different locations depending on the degree of settlements. Three, each cores from Sikkhawithee 1 (a) and (b) Road, Four cores from Withayawithee 3 Road and six cores from Withayawithee 2 Road were extracted.

DCP showed the blow counts against the penetration data. The penetration index (PI) is defined as the difference between each blow and the one preceding it. This PI will show the uniformity of subgrade soil and the difference of California Bearing Ratio (CBR) at each site. At the core holes, DCP was conducted to determine the CBR and base modulus by relations in equation (1) and (2) respectively.

$$\text{Log CBR} = 2.465 - 1.12 (\log \text{PR}) \quad \text{or} \quad \text{CBR} = 292/\text{PR}^{1.12} \qquad (1)$$

$$E\ (\text{psi}) = 2{,}550\text{CBR}^{0.64} \quad \text{or} \quad E\ (\text{MPa}) = 17.58\text{CBR}^{0.64} \qquad (2)$$

Where CBR=California bearing ratio; PR=DCP's penetration rate through the layer (mm/blow) and E= Base Modulus

Result

3.1 Profile leveling

The PFI proved to be an effective method to lift and align the pavement to a desired level. Figure 2 showed the lift provided by the foam at Withayawithee 2 Road where the maximum lift was 114 mm on the middle of the lanes at 45 m from the beginning of the pavement section (i.e. station 0). In some part of the road, settlements were seen at some portions of the road while injecting polyurethane foam which may be due to presences of hard support acting as pivot which transfers lifts at one section to settlement in another.

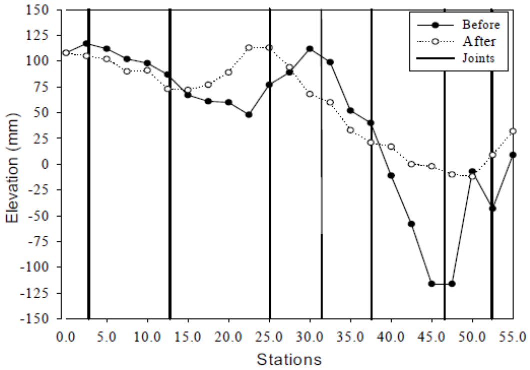

Figure 2: Profile leveling before and after polyurethane foam injection on the middle of lanes of Withayawithee 2 Road.

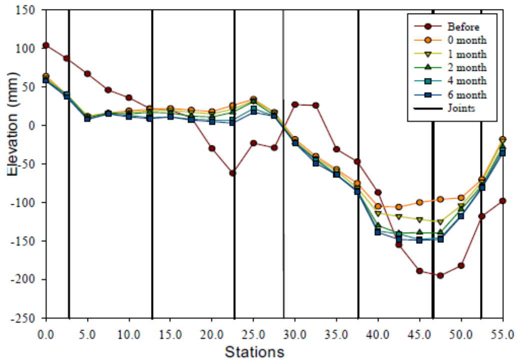

Figure 3: Settlements on right lane of Withayawithee 2 Road during the study period.

After six months of profile survey, maximum settlement was observed on the right lane of Withayawithee 2 Road with 52 mm as highest at station 47.5 m (Figure 3). In the other locations also settlements were noticed varying from 2-10 mm.

3.2 Coring Results

The cores extracted also confirmed the diameters of the tie and dowel bar along with the diameter of the steel reinforcements. Moreover, the cores represented the presence of the polyurethane foam beneath the pavements.

Even though designed thickness for the existing concrete pavement was 20 cm and the extracted cores deviated considerably from this value, the thickness of the cores across all four sites varied from 17.5 cm to 22 cm. However a mean thickness of 19.9331 cm was found throughout the sections.

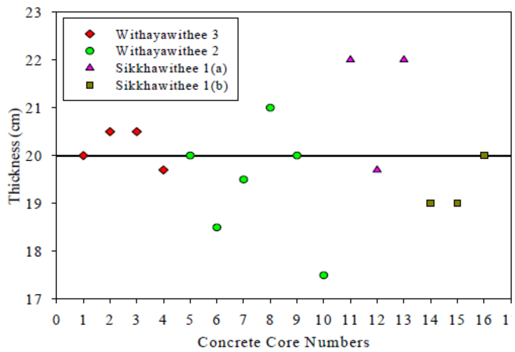

Figure 4: The pavement thickness variation as per the cores from all the sites.

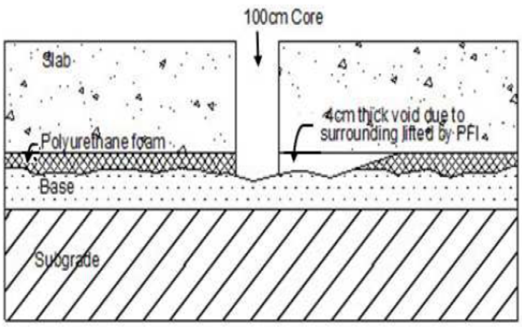

Figure 5: A 4 cm thick void formed due to surrounding being lifted by PFI beneath core #4

It was noted that the cores from Withayawithee 2 Road had lesser number of cores, thinner than the designed thickness while the cores from Sikkhawithee 1 (a) Road had more cores thicker than the designed thickness. The cores from Withayawithee 3 Road and Sikkhawithee 1 (b) Road had more core equivalent to the designed value. The Figure 4 illustrated the variation of thickness for all the cores from the designed value.

Core #7 extracted from a longitudinal faulted joint at Withayawithee 2 Road showed that the polyurethane was unable to penetrate through the joint, forming void of 2-3 cm on one side of the fault (Figure 6 (c) and (d)).

Core #6 and core #10 were taken from cracked areas which showed that the crack was throughout the thickness of the slab was core #12 is cracked till half way only (Figure 6 (e) and (f)).

Figure 6: (a), (b) void formed by surrounding being lifted by PFI at core #4; **(c), (d)** PFI was not able pass through the fault forming void at one side at core #7; **(e)** core #10 cracked throughout the core thickness; **(f)** core #12 crack half way through the thickness.

Even though core #4 at Wittayawithee 3 Road was 50 cm away from nearest injection hole, a void of 3-4 cm was formed below the slap surrounded by polyurethane foam (Figure 5 and Figure 6 (a) and (b)).

All the other cores were noticed with a layer of polyurethane, some as much as 10 cm to 20 cm polyurethane layer beneath the slab. Coring also confirmed that the tie bar installed was 25 mm in diameter and with the help of a Micro-Covermeter the spacing was found to be 30 cm.

3.3 Dynamic Cone Penetrometer testing (DCP) Results

The DCP testing was conducted on the exposed subgrade by the coring and the data from number of blows and penetration depth for each blow were recorded. A penetration rate exceeding 38 mm per blow indicates very weak foundation support, with a CBR value of 5 and modulus of 48 MPa (Chen & Won, 2007).

Figures 7 and 8 showed the non-uniformity of the subgrade soils, which was common throughout all section. The required number blow and the PI increased as the DCP went dipper.

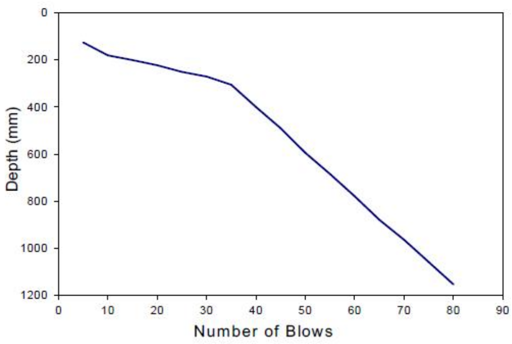

Figure 7: Blow vs. depth graph for core #1 at Withayawithee 3 Road.

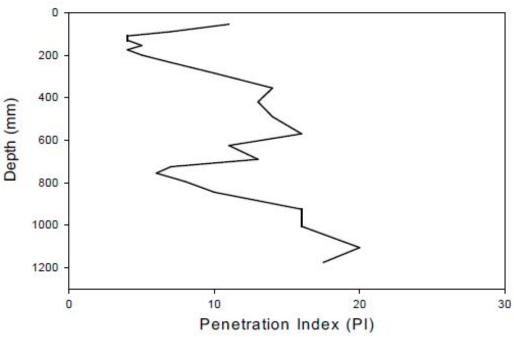

Figure 8: PI vs. depth graph for core #9 at Withayawithee 2 Road.

In Withayawithee 3 Road, a similar pattern of weak layer just beneath the pavement were observed in all the four core locations, as shown in Figure 9 while in Withayawithee 2 Road, some graphs displayed portions of extraneous data due to the presence of rocks in the subgrade as shown in Figure 10.

However, the DCP result showed that the base and subgrade at the cracked areas were slightly weaker than in non-cracked areas. The average modulus exceeds 100 MPa in non-cracked area while Modulus is below 100 MPa in cracked area as shown in Figure 11 and 12 respectively.

At Sikkhawithee (a) and (b) Road, the DCP result also showed weak base and subgrade with average Modulus below 100 MPa through some tests.

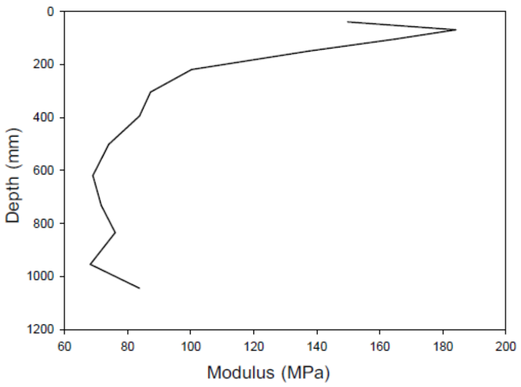

Figure 9: Modulus of subgrade reaction from core #4 at Withayawithee 3 Road.

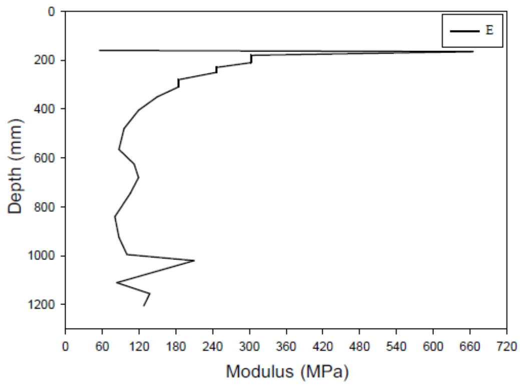

Figure 10: Modulus of subgrade reaction from core #5 at Withayawithee 2 Road.

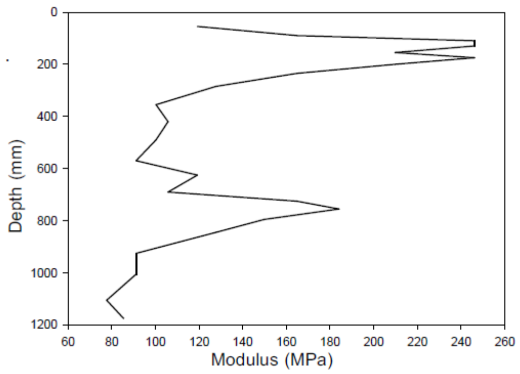

Figure 11: Modulus of subgrade reaction from core #9 at Withayawithee 2 Road on noncracked area.

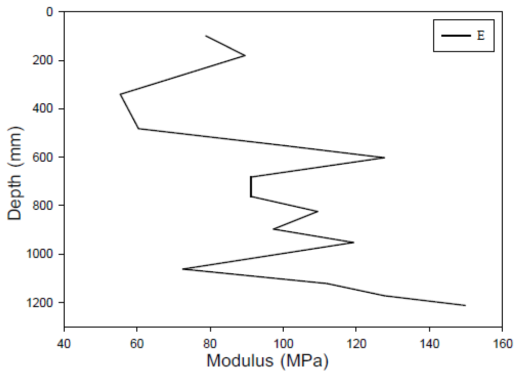

Figure 12: Modulus of subgrade reaction from core #7 at Withayawithee 2 Road at cracked location.

3.4 Pavement Cracks observations

After six month of observation, it was noticed that new cracks (traverse cracking) developed at Withayawithee 2 and 3 Road and moreover old cracks which were sealed with cement slurry were also reopened.

Figure 13 showed some of the pictures from the site location. The most severe distressed were noticed in Withayawithee 2 Road.

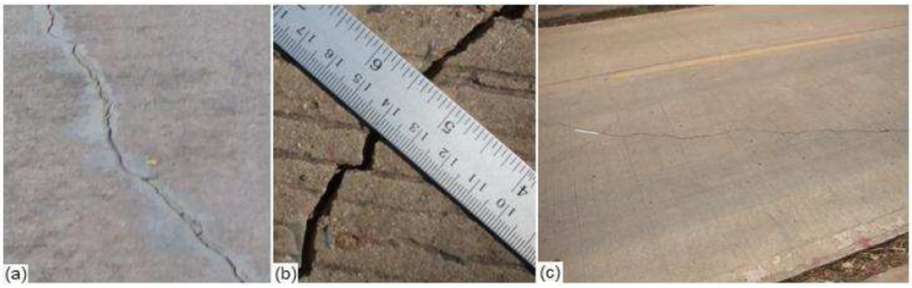

Figure 13: (a) old cracked opened, **(b)** New cracks found and **(c)** Traverse Cracking Observed

Discussion

The PFI has shown that it can be used to lift the previously settled pavement and to level the pavement by aligning the surface. Thus, it all contributed to a better ride quality and a leveled road. The PFI was carried out in the night when the traffic was minimum and time taken for injection was comparatively lesser than the conventionally repair methods. However, the profile leveling proved that there was at least 2 mm settlement at all the survey control points and in some areas even exceeding 50 mm in six month after PFI. Amongst the site locations, Withayawithee 2 Road showed the maximum settlements which might be because it experienced maximum lift during the PFI and possibly due to more traffic load at the construction zone. Even Withayawithee 3 Road showed more settlements than those at Sikkhawithee 1 Road.

The settlements and appearances of the cracks can only be concluded that the base support under the slab were inadequate which DCP tests proved. Based on years of field testing and performance monitoring, a good untreated granular base should have a modulus exceeding 345 MPa (Chen & Won, 2007). However, all the sites exhibited a maximum of modulus of 140 MPa.

The coring revealed the actual inner picture of the pavements where two cores exposed the voids that PFI could not fill. Cores also confirmed the diameter of the tie bar (i.e. 25 mm) which Micro-Covermeter has located. Further, layers of varying thickness of polyurethane foams were seen beneath the pavements but it did not penetrate the base material. The PFI can penetrate voids openings with a minimum dimension of 3.0 mm up to 0.62 m from the injection point (Soltesz, 2002). The slabs were lifted without strengthening the base material thus allowing the progression of the distresses and settlements.

Conclusion

The PFI method of rehabilitation of rigid pavement has not been successful in providing the support to the slab for long run. However, it had lifted the pavement to the desired level and provided a good ride quality for a short span of time. The pavement lifted by PFI still experiences settlement and even new cracks have appeared on pavement. However, the settlement cannot be reasoned solely on PFI, increase in traffic loading and sub-drainage can be also some of the contributing factors which can only be determined through further investigation. Information from the investigation of the pavement can be used to identify the causes of the problem and develop an optimal rehabilitation strategy to resolve it.

Following were some of the findings drawn from the investigation conducted in this study:

1. A core extracted from a faulted joint found void beneath it which clearly indicated that polyurethane was unable to penetrate through the fault. Therefore, it would not have occurred if the injection was thoroughly done on both sides of the slabs which would have proven a better workmanship.
2. The drawback of the Polyurethane not being able to penetrate through a opening less than 3 mm left the base material to progress with its original settlement rate.
3. Due to presence of hard base material, it had caused settlements at some portion while lifting the other portion. This may led to the movement of the slab resulting in instability which may have been prevented by uniformly lifting the pavement.
4. The extraction of core found to be a practical tool to present clear picture of the polyurethane thickness and pavement distresses. DCP results showed that the base material were weak at all the sites, which has led to cracking. These stresses on the need for a good base material to support the wheel loads applied.

The distresses in concrete pavements are caused by more than single factor which is why indentifying the distress mechanisms requires a comprehensive understanding of the factors involved and their interaction, which was a great challenge (Chen et al., 2009). Therefore, more investigation could be carried out to find out the optimal rehabilitation strategy. However, from this study it would be wise to recommend that PFI can be used as a supplementary to other rehabilitation methods or as a temporary method to realign pavement.

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BIOGRAPHY

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