EVALUATION OF THE PERFORMANCE OF POLYURETHANE FOAMS IN RIGID PAVEMENT PRESERVATION

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ABSTRACT

After several years of service, concrete pavement slabs tend to settle due to weak subgrade or erosion of the subgrade soil. Different treatment techniques have been used to rectify the problem. In recent years, high-density polyurethane (HDP) foams were introduced on concrete pavements after their success in leveling settled sidewalks and building bases/foundations. Compared to other traditional slabs jacking/stabilization material, HDP foams are cost-effective, their installation requires shorter lane closure times and protects the subgrade from subsurface water infiltration by filling the voids.

In 2015 and 2016, the Tennessee DOT applied HDP material on sections of Interstates I-24 and I-75 in Chattanooga, Tennessee to lift and level settled concrete pavement slabs. Longitudinal profiles data were collected using a standard high-speed inertial profiler before and after application of the material to assess the performance of the treated sections over time. These data were evaluated by using the profile viewing and analyzing (ProVAL) software to compute the international roughness index (IRI) and the transverse joint faulting.

Results show that application of HDP foams did neither improve nor retrogress the pavement condition but maintained it in its state before application of the material. This study recommends an in-depth ground investigation to be carried out before injection of the material, establishment of a standardized protocol for selecting pavement sections suitable for HDP foam injection, and contractors to use sophisticated leveling equipment, instead of the adjacent slab as a reference, to avoid accumulation of errors due to overcorrection.

DEDICATION

This thesis is dedicated to my late lovely mother Kariba Mashauri Lukiko.

May Her Soul Rest in Eternal Peace

(1963 - 2005)

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LIST OF ABBREVIATIONS AND ACRONYMS

AADT	Average Annual Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
ACPA	American Concrete Pavement Association
AFM	Automated Faulting Measurement
ARRB	Australian Road Research Board
ASCE	American Society of Civil Engineers
CRCP	Continuous Reinforced Concrete Pavement
DOT	Department of Transportation
DIM-LTTP	Distress Identification Manual for the Long-Term Pavement Performance
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
GPR	Ground Penetrating Radar
HDP	High Density Polyurethane
HMA	Hot Mix Asphalt
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
LTE	Load Transfer Efficiency
NDT	Non-Destructive Testing

NHS	National Highway Systems
PCC	Portland Cement Concrete
ProVAL	Profile Viewing and Analyzing Software
SHA	State Highway Agency
TDOT	Tennessee Department of Transportation
U.S.A.	United States of America

CHAPTER I

INTRODUCTION

Roads are the backbone of any society's socio-economic development. In the U.S. there are over four million miles of road network ranging from interstates to residential streets. In 2016 only, these roads enabled people and goods to move over 3.2 trillion miles (American Society of Civil Engineers, 2017).

The American Society of Civil Engineers report card of 2017 on U.S. infrastructure reports that $813 billion is required to renovate highways and bridges to an excellent condition; more than 50% ($430 billion) of the investment is for highways repairs whereas the remaining is for bridge repair, system expansion, safety improvements, operations and environment concerns (American Society of Civil Engineers, 2017). The reasons for huge funding required for highway repairs are due to low capital invested in repair/rehabilitation and their delay since roadways can still be used even if they are in poor conditions (American Society of Civil Engineers, 2017; Garber and Hoel, 2014).

The most common roadways pavements in practice in the U.S. are flexible pavement, rigid pavement, and composite pavement. Flexible pavements are constructed from hot mix asphalt (HMA) under laid by an asphalt binder on the base course/sub-base and subgrade. The layers are arranged depending on the material strength to resist the effect of loading with high-quality materials on the top (Huang, 2004).

Rigid pavement, also known as concrete pavement consists of a concrete slab constructed from Portland Cement Concrete (PCC), in which the slab can be reinforced or unreinforced with defined thickness and width. The PCC slab rests on the subgrade or granular base/sub-base course. The sub-base is introduced mainly for controlling pumping, frost action, subgrade shrinkage and swell, and improvement of drainage in the pavement (Huang, 2004). Composite pavement consists of an asphalt concrete surface which provides a smooth ride quality, a PCC slab which acts as a major load carrying component. Composite pavements are expensive, hence they are mostly constructed when concrete pavements are being rehabilitated (Huang, 2004).

There are different types of concrete pavement, but the following three are more common in roadway construction as compared to other types such as precast concrete, roller compacted concrete and porous concrete;

i. Jointed Plain Concrete Pavement (JPCP)

JPCP is a mass concrete slab 3.6-6.0 m (12-29 ft) in length, built with closely spaced contraction joints and load transfer mechanism is provided by dowels or aggregate interlocks, they have a risk of developing cracks as the joint spacing increases (Delatte, 2014; Huang, 2004). Joint spacing ranges from 4.5 to 9.0 m (15 to 30 ft) depending on climate, aggregate, and prior experience (Huang, 2004).

ii. Jointed Reinforced Concrete Pavement (JRCP)

JRCP is reinforced with a wire mesh or deformed bars to increase the joint spacing (which is larger than in JPCP) and hold the slab together after cracking, only dowels are used to transfer the vertical loads at the joints. According to Huang (2004), its joint spacing varies between 9.1 to 30 m (30 - 100 ft). Delatte (2014) states that slabs of length up to 30 m (100 ft) have been used, but their common slab length range from 7.5 to 9.0 m (25 – 30 ft).

iii. Continuous Reinforced Concrete Pavement (CRCP)

CRCP is reinforced continuously throughout the length of the pavement, with no joints unless when the pavement meets a bridge or another type of pavement; for instance, flexible pavement (construction joints). The pavement is left to crack within acceptable limit which is about 1 mm (0.04 in). Stresses induced in the pavement due to traffic and temperature gradient are released through these cracks. CRCP has higher initial construction cost but lower maintenance cost, as compared to JRCP which requires lesser initial construction cost but higher maintenance cost during its service life (Delatte, 2014; Huang, 2004).

After several years of service, concrete pavement may not function as they were intended due to distresses influenced by factors such as frequent heavy loadings, material properties of the supporting foundation, environments and climatic changes, and aging of the pavement over time. Slab settlement/ drop off is a common distress in jointed concrete pavements (JPCPs and JRCPs).

Slab drop off is mostly caused by weak foundation supporting the pavement which may be due to poor compaction of the layers, erosion of the subgrade soil by pumping, and inferior quality of the material. Apart from discomfort experienced by road users while traveling on differential settled concrete slabs, slab drop off also poses safety hazards to them. State DOTs are therefore compelled to maintain the smoothness and safety expected by road users by rectifying pavement defects by applying appropriate preservation/repair techniques.

The FHWA requires DOTs to include pavement preservations strategies in their pavement management program; because appropriate pavement preservation strategy applied at the right section, and at the right time is proven to be cost-effective and sustainable while providing smoother, safer and quieter riding (Van Dam et al., 2015).

Pavement preservation does not include structural and operational/capacity improvement of the roadway. All corrective or preventive maintenance, as well as minor rehabilitation activities, are regarded as pavement preservation (Davies and Sorenson, 2000; Huang, 2004).

This study evaluates the performance of JPCP treated with HDP foams to preserve its surface smoothness. To assess the effectiveness of the material in surface leveling settled concrete slabs, raw pavement surface roughness data collected by a standard inertial profiler before and after application of the material is analyzed using ProVAL to obtain the transverse joint faulting of the sections.

1.1. Problem Statement

Pavement preservation programs have improved the condition and extended the life of the pavement at a relatively low cost. Even though, some of the treatments had been reported to fail due to poor timing, material quality, inappropriate treatment selection and construction defects (Van et al., 2017). State DOTs still embrace pavement preservation philosophy since it is proactive and has been proven to meet expectations if the fore mentioned drawbacks are addressed.

Roadway pavements in the U.S. are deteriorating faster than they are being restored because funds invested are not enough to address all the needs (Garber and Hoel, 2014; Peterson, 1981). The ASCE report card on U.S. infrastructure of 2013 to 2017 states that highways are being underfunded; due to dwindling funds reserved for them, there is a backlog of $430 billion required to repair them to a good condition (American Society of Civil Engineers, 2017). Also, most concrete roads in the U.S. have served beyond their design life; hence they barely support the increasing traffic loads whereas some sections of rigid pavements have failed badly.

DOTs, metropolitan planning organizations and other stakeholders in the field of pavement preservations have been researching on treatments or materials that are cost-effective and sustainable while offering superlative long term-performance. Application of polyurethane material in foundation leveling of garages, buildings, and sidewalks etc. has attracted some DOTs to use them in raising/leveling and stabilizing soils underneath a settled concrete slab of a rigid pavement.

For the first time in 2015, the Tennessee DOT used PolyLevel® to level five settled sections of interstate I-24 and I-75 in Chattanooga, Tennessee. Since HDP foams have proven to be cost-effective, less time consuming and requiring less lane time closure as compared to other stabilization/jacking materials, it is, therefore, necessary to assess the performance of this material in improving the ride quality of concrete pavements.

1.2. Objectives of the Study

The main objective of this study is to assess the performance of polyurethane treated pavement sections by evaluating and analyzing the transverse joint faulting of the sections before and after application of the polyurethane material.

1.3. Scope of the Study

This research focused only on the performance assessment of HDP materials called PolyLevel®. The study evaluates only surface characteristics of the treated sections; the structural integrity of the treated sections is not assessed.

1.4. Thesis Overview

This thesis is divided into five chapters. Chapter I introduces the reader to the topic, state the problem being studied, identify the objective to be achieved and define the scope of the study. Chapter II presents an intensive literature review on concrete pavement defects, concrete pavement preservation strategies, slab stabilization and jacking, and case studies of several DOTs that have used HDP foams to rectify slab drop off problems in their concrete pavements.

Chapter III describes the methodology used to achieve the objective, wherein all the study sites, data collection and data analysis methodologies are explained. Chapter IV presents the results and analysis. In this chapter, analysis and results of raw profile data using ProVAL software are presented. Statistical analysis on the changes in transverse joint faulting before and after application of HDP foams are discussed too. Conclusion and recommendations made from this study are provided in Chapter V.

CHAPTER II

LITERATURE REVIEW

Pavement preservation is a proactive program of activities aimed at conserving the investment in highways, enhancing pavement performance, meeting users expectations, ensuring cost-effectiveness and prolonging its life (Davies and Sorenson, 2000; Huang, 2004). Pavement preservation is immanently a sustainable activity as it employs use of low cost and low environmental impact treatments to extend the life of the pavement or delay major rehabilitation/reconstruction works; thereby reducing consumption of virgin materials and conserve energy while minimizing emission of greenhouse gases and interference/disturbance of ecosystem (Gransberg et al., 2014; Van Dam et al., 2015). Despite the few documented historical data on preservations performance, several state highway agencies (SHAs) have reported them to be cost-effective as compared to the traditional rehabilitation/reconstruction approach (American Association of State Highway and Transportation Officials, 2012). Well maintained pavements provide smoother, safer and quieter riding to users. Thereby, improving vehicles fuel efficiency, and reducing traffic crashes and noise impacts to the surroundings (Van Dam et al., 2015).

Pavement preservation treatments are applied not only to reduce water infiltration or intrusion of incompressible material to the pavement structure through cracks but also to improve slab support, load transfer efficiency, rideability, surface friction and noise reduction (Smith et al., 2014). Pavement preservation treatments do not focus on upgrading the pavement. Hence, structural capacity improvements and reconstruction activities are not considered as preservation (Burningham and Stankevich, 2005; Huang, 2004).

2.1. Rigid Pavement Distresses

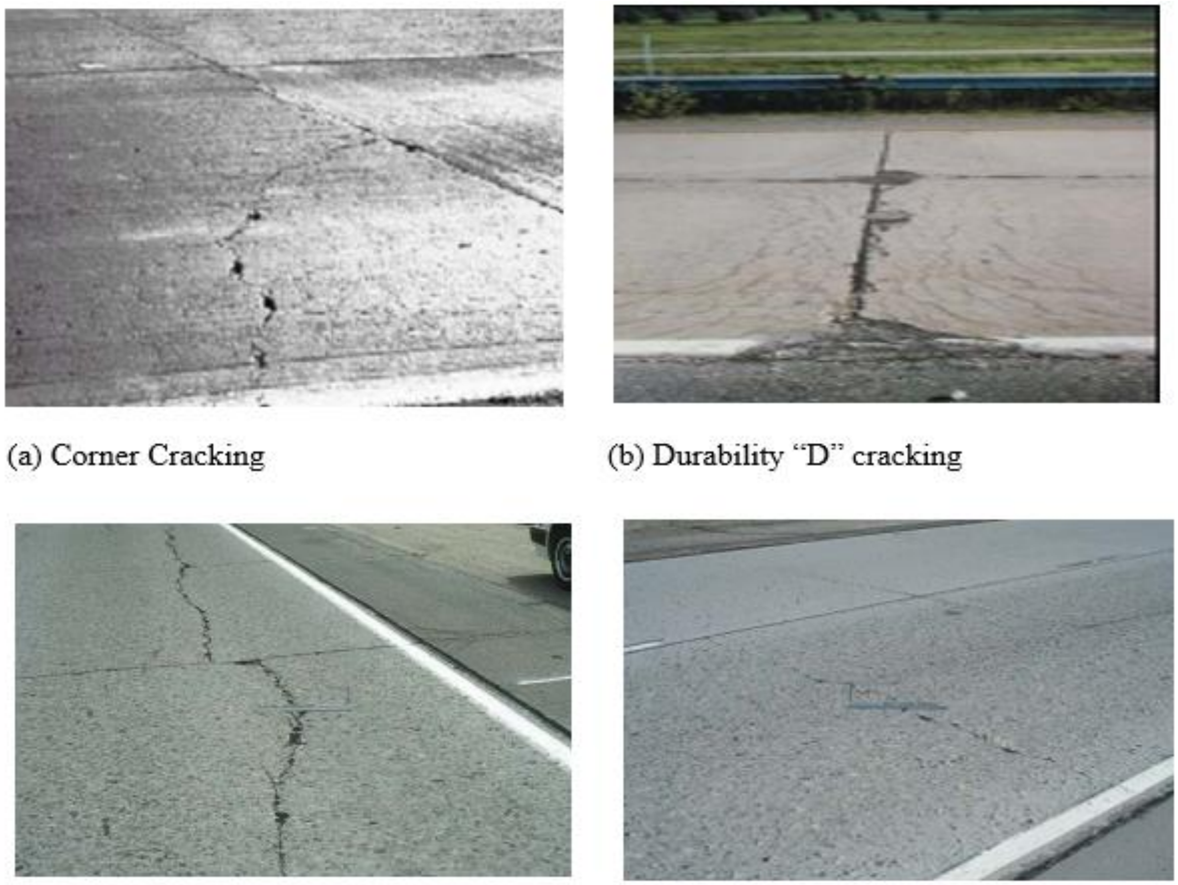
Distresses in rigid pavements are associated with induced stresses, age of the pavement, and deficiencies in materials, construction and maintenance (Garber and Hoel, 2014; Huang, 2004). Distress in pavements leads to either functional or/and structural failure of the pavement. Functional distress affects the ability of the pavement to provide a safe and smooth ride to its users whereas structural distress causes structural incapability of the pavements (Peshkin et al., 2011).

Before embarking into the repair of the damaged pavement section, engineers identify types of the distress, their causes, and severity; then select an appropriate preservation technique after conducting a life-cycle cost analysis of possible techniques based on the desired improvements. While pavement preservations are a suitable option for functional failure, they are not for structural enhancement of the pavement.

Apart from distresses, there are three other characteristics used to evaluate pavement rehabilitation or maintenance needs: (1) pavement ride quality for surface condition of the pavement, (2) pavement deflection for structural integrity, and (3) skid resistance for safety (Garber and Hoel, 2014; Huang, 2004; Shahin, 2005). Data from these four pavement conditions characteristics are not only useful in selecting a feasible treatments technique, but also in identifying its impacts, work prioritization and funds optimization (Huang, 2004).

The distress identification manual for the long-term pavement performance (DIM-LTPP) groups distress on jointed concrete pavement in the following manner: (1) cracking, (2) surface defects, (3) joint deficiencies and (4) miscellaneous and others (Miller and Bellinger, 2014). Each group is further divided into several sub categories. The most common distress joint failure in jointed plain concrete pavement (JPCP) are briefly described below:

i. Crackings in concrete pavements is a result of stresses caused by repeated traffic loading. These stresses may not even exceed the flexural strength of the concrete slab but still may lead to formation of structural cracks due to lack of uniform base support, among other things. Cracking is also influenced by weak subgrades, expansive soils, differential settlements and curling of concrete slabs due to temperature gradient (Bautista and Basheer, 2008). If not properly sealed cracks are likely to develop into concrete spalling in situation where there is erosion of subgrade/base support and crack formation as a result of moisture infiltration through cracks or joints (Bautista and Basheer, 2008). Figure 2.1 shows cracking distresses as categorized in the DIM-LTPP based on their location and formation on the pavement.



(a) Corner Cracking (b) Durability "D" cracking

(c)Longitudinal cracking (d) Transverse cracking

Figure 2.1 Types of cracking in rigid pavement (Source: Miller and Bellinger, 2014)

ii. Spalling of concrete pavement is identified by cracking, breaking, chipping or fraying of slab edge within 0.3 m from the face of longitudinal, transverse or corner of the pavement as shown in Figure 2.2 (Lee and Shields, 2010; Miller and Bellinger, 2014; North Carolina Department of Transportation, 2015).

Apart from being influenced by cracking near the transverse/longitudinal joints; according to Huang (2004) spalling (transverse or longitudinal) is also caused by poorly designed or constructed load transfer devices; corner spalling is caused by freeze-thaw condition, durability cracking or other factors. Usually spalling extends to intersect the joint at an angle and not throughout the whole slab thickness (Huang, 2004). Spalling is a joint deficiency related distress, others include longitudinal and transverse joint seal damage.

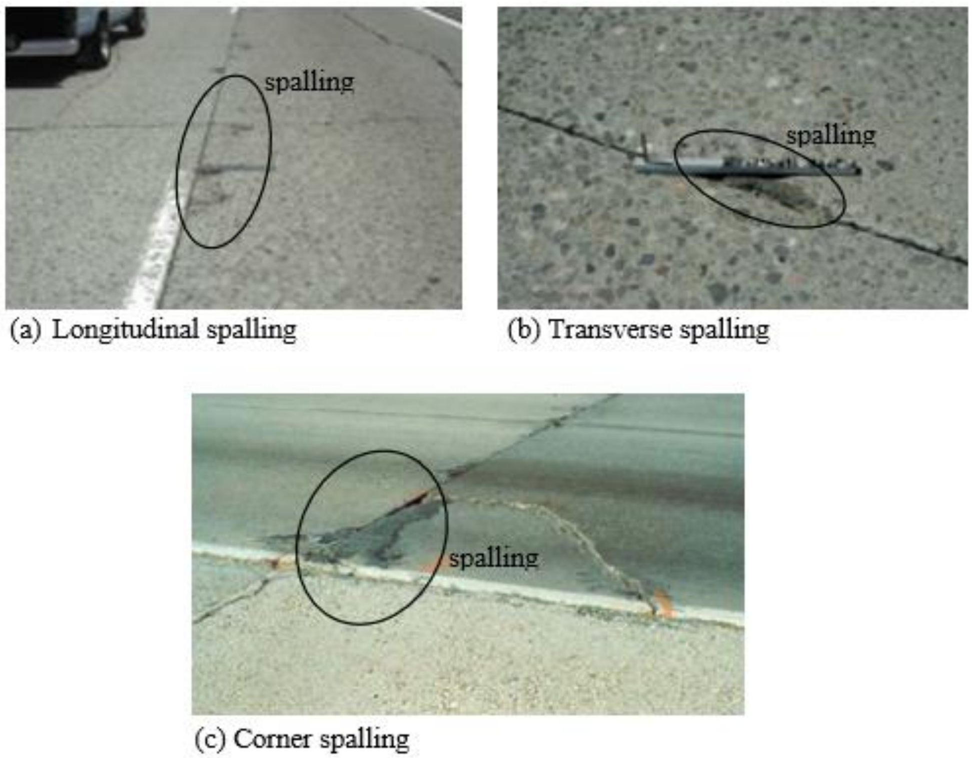


Figure 2.2 Slab spalling in rigid pavement (Source: Miller and Bellinger, 2014; North Carolina Department of Transportation, 2015)

iii. Faulting is commonly found in jointed concrete pavements without dowel bar reinforcement. It manifests as a slight settlement of the leading edge of each slab in the direction of traffic (Papagiannakis and Masad, 2017). Due to lack of dowel bars or aggregate interlock in JPCP for load transfer, sudden increases in pore pressure in wet subgrades occur, which in turn produces migration of fines and settlement under the leading edge of each slab. Where the sudden pore pressure buildup is accompanied by squirting of water and fines through the joint, the distress is referred as pumping (Huang, 2004; Papagiannakis and Masad, 2017). Faulting can be either in the longitudinal or transverse direction of the joint or crack (Figure 2.3). However, the most common ones are near the joint in the transverse direction.

Figure 2.3 Faulting of transverse crack (Source: Miller and Bellinger (2014))

According to Smith et al. (1998) faulting is considered as a drainage related distress. Improvement in pavement drainage system, shorter joint spacing, use of widened lanes and stabilized base/ subgrade reduces faulting effects significantly (Selezneva et al., 2000).

In most cases of faulting mechanism, the approach slab is higher than the leave (departure) slab and is considered as positive faulting; negative faulting is recorded when the leave slab is higher than the approach slab (Miller and Bellinger, 2014). Joint faulting is measured in the nearest mm (in.) at 0.3 m (1 ft) from the outside pavement edge and 0.75 m (2.50 ft) from the outside wheel path (Miller and Bellinger, 2014). Faulting leads to unevenness of the pavement affecting the roughness and ride quality on the pavement.

iv. Pumping is the ejection of soft subgrade/subbase soil (muddy water) underneath the slab through cracks or joints, faults or along the edge of the pavement (Figure 2.4) due to slab deflection under dynamic traffic loading (Huang, 2004; Miller and Bellinger, 2014). Curling of slabs or plastic deformation of the subgrade creates void space, due to capillary forces (if the subgrade is on/under the water table) or ingress of water from the top into the subgrade through cracks or joints). The void space will be filled with fine-soft soil, when frequent passage of heavy wheel loads occur the fine-soft soil under the leading slab are pumped due its deflection to the trailing slab which had rebounded and created a vacuum; the fine material is sucked outside from underneath the leading slab through joints or cracks (Huang, 2004). Pumping and faulting are indicators of loss of slab support and likely to cause corner cracking.

Figure 2.4 Pumping and water bleeding in JPCP (Source: Miller and Bellinger, 2014)

Table 2.1 shows distress in jointed concrete pavements with respect to their causes and unit of measures used to define their extent of effects.

Table 2.1 Distress in Concrete Pavements

Distress	Causes	Unit of Measure
Corner breaks	Heavy repetitive loads, erosion of corner support soil, slab curling and/or warping	Number
Durability "D" cracking	Freeze-thaw effects in coarse aggregates	Number of Slabs, square meters
Longitudinal cracking	Fatigue damage combined with slab curling and/or warping, improper joint construction and foundation movement	Meters
Transverse cracking		Number, meters
Longitudinal Joint Seal damage	Hardening and cohesive or adhesive failure of the sealant	Number
Transverse joint seal damage		Number, meters
Spalling of longitudinal joints	Internal compressive stresses build up in the slabs due to infiltration of incompressible material in the joints and aggregate-alkali reaction; D-cracking; misaligned or corroded dowels; poorly consolidated concrete near the joint; or damage caused by joint sawing, joint cleaning, cold milling, or grinding	Meters
Spalling of transverse joints		Number, meters
Map cracking and crazing	Over-finishing and alkali-aggregate reaction	Number, square meters
Scaling	Poor concrete cover, over-finishing and inadequate air entrainment	Number, square meters
Polished aggregates	Polishing of aggregates by vehicle's tires	Square meters
Pop-outs	Freezing of course aggregates near the concrete surface	NA
Blowups	Slab build up compressive stresses due to infiltration of incompressible materials in the joints, expansion of the concrete	Number
Transverse Construction joint deterioration	Dusty construction joint, smooth joint surface which is likely not to bond with the new section	Number
Faulting of Transverse joints and cracking	Pumping of mud water from slab corner, and loss of support and buildup of fines under the leave and approach corner respectively	Millimeters

Lane to shoulder drop off	Improper joint construction and inadequate sealant material	Millimeters
Lane to shoulder separation		Millimeters
Punchouts	Heavy repeated loads, inadequate slab thickness, loss of foundation support, or a localized concrete construction deficiency	Number
Pumping and water bleeding	Heavy repetitive traffic loads, erosion subgrade/base course soil	Meters, number

Source: Miller and Bellinger (2014), Smith et al. (2014).

2.2. Concrete Pavement Preservation Strategies

Strategy selection for pavement preservation is substantially influenced by the pavement management system of the transportation agency. Pavement management data are essential in the screening process during treatment selection, as they are used to establish priorities among the competing pavement needs, determine candidates suitable for preservation treatments, evaluate the feasibility of the treatment and its cost-effectiveness, set performance targets, and forecast consequences of the treatment in the future condition of the network (AASHTO, 2012; Smith et al., 2014).

Table 2.2 shows different treatments description and their application in concrete pavements (Smith et al., 2014; Van Dam et al., 2015). Smith et al. (2014) suggest the following procedures be followed when the agency is selecting treatments to be applied to a damaged pavement section:

i. Conducting a thorough pavement evaluation

ii. Determining causes of distresses and deficiencies

iii. Identifying effective and sustainable treatments that address deficiencies

iv. Identifying constraints and key selection factors

v. Developing a feasible treatment strategy

vi. Assessing the cost-effectiveness of the alternative treatment strategy

Moreover, selection of appropriate treatment strategy for a particular segment of the pavement system depends on the following factors: (1) type of existing pavement, (2) type, severity and extent of distress (3) volume and type of current and projected traffic, (4) local climatic condition, (5) expected performance of the pavement, (6) work zone time restrictions, (7) agency and user costs associated with each treatment, (8) availability of qualified contractors and quality material, and (9) environmental sustainability (Gransberg et al., 2014; Moulthrop and Smith, 2000; Peshkin et al., 2011; Smith et al., 2014).

Table 2.2 Common Treatment Types in Rigid Pavement and Their Applicability

Treatment	Description	Applicability
Slab stabilization	Involves injection of flowable materials underneath a concrete slab through drilled holes to fill the voids	Sections likely to face loss of support, for example, areas showing early sign of pumping or mid-slab cracking
Slab Jacking	Lifting a settled slab to its original profile by injection of cement grouts or expansive polyurethane materials through drilled holes	Localized areas with depression or settlements
Partial-depth repair	Removal of small, shallow-top deteriorated areas (1/3 to 1/2) of concrete slab and replace with cementitious or polymeric material	Low to moderate spalled and cracked areas, localized areas with scaling and joint defects
Full depth repair	Total replacement of deteriorated concrete slab by casting in place a new slab or installing a pre-casted one	Slabs with distresses such as longitudinal cracking, transverse cracking, joint spalling. blowups, punch outs, corner breaks etc.
Retrofitted edge drains	Cutting of a trench along the pavement edge and placement of a longitudinal edge drain system along with transverse outlets and headwalls	Areas likely to develop moisture-related damages such as pumping, faulting, and corner breaks
Dowel bar retrofit	Restoration of load transfer of slabs by placement of dowel bars across joints or cracks	Slabs with poor load transfer efficiency due to lack of bars, poor aggregate interlocks or support erosion
Cross stitching	Involves maintaining load transfer across non-working longitudinal cracks that are in good condition by preventing horizontal and vertical movements	Longitudinal joints likely to faults, sections showing indication of slab migration and weak aggregate interlocks
Slot stitching	Involves repairing of longitudinal cracks and joints that develop as a result of dowel bar retrofit treatment by using deformed tie bars	Segments with longitudinal cracks due to dowel bar retrofit treatment

Diamond grinding	Removal of a thin layer of concrete (typically 3 to 6 mm) by using a self-propelled machine fitted with a series of closely spaced, diamond saw blades	Areas with faulted transverse joints over 2 mm, section with roughness more than 2.5 to 3.5 m/km, low surface friction and noise sensitive areas
Diamond grooving	Cutting of narrow, discrete grooves into the pavement surface either in the longitudinal or transverse direction	Sections prone to hydroplaning or splash and wet-weather related accidents
Joint resealing	Involves the removal of deteriorated joint sealant (if present), preparation of the joint side walls by refacing and pressure-cleaning the sides and installing new sealant material	Joints with no sealant or sealant not functioning as intended or sealed joints containing incompressible materials
Crack sealing	Involves routing, cleaning and sealing cracks wider than 3 mm (0.125 in) using high-quality sealant materials to minimize surface water infiltration into the pavements and slow down crack deterioration effects	Sections with low to medium severity levels of longitudinal and transverse cracking with minimal spalling and faulting
Concrete Overlay	Involves placing concrete layer either bonded or unbonded to an existing pavement surface	Segments with surface distresses (Overlay thickness and type varies based on the structural integrity of the existing pavement)
Ultra-thin wearing course	Consists a thin 10 to 20 mm layer of gap-graded aggregates and polymer-modified HMA layer placed on a polymer-modified emulsified asphalt membrane	Sections with low frictions or experiencing hydroplaning or water splash. However, its effectiveness is compromised by refractive cracks

Sources: Smith et al. (2014); Van Dam et al. (2015)

Performance indicators such as condition rating and smoothness indices and other key distress measures like mean joint transverse faulting and percentage of cracked slabs are used to establish pavement preservation windows, triggers and threshold levels that define the appropriate timing of the treatment (Smith et al., 2014). The expected performance projection of the particular treatment depends on the treatment type itself, surface and structural condition of the existing pavement, climatic condition and projected traffic load (Smith et al., 2014). Table 2.3 shows the life expectancy of several pavement preservation techniques achieved from Peshkin et al. (2011).

Table 2.3 Concrete Pavement Repair Treatments Life Span

Treatment	Expected Performance (Years)
Concrete joint resealing	2 to 8
Concrete cracking sealing	4 to 7
Diamond grinding	8 to 15
Diamond grooving	10 to 15
Partial-depth concrete patching	5 to 15
Full-depth concrete patching	5 to 15
Dowel bar retrofit	10 to 15

Source: Peshkin et al. (2011)

Applying pavement preservations at early stages accumulate many benefits. The pavement services longer without needing major rehabilitation or reconstruction hence reducing the life cost and extending its life. Smith et al. (2014) state few benefits associated with pavement preservation, and they are explained:

i. Higher consumer satisfaction - The public expect safe, smooth, comfortable and efficient flow of traffic when traveling on a road (Shah et al., 2011). Pavement preservation requires fewer resources as well as less lane closure time as compared to rehabilitation or reconstruction (Davies and Sorenson, 2000; Shah et al., 2011). A good pavement preservation program will benefits users from project selection by agencies prioritizing roadway's network sections in needs to treatment selection by applying a cost-effective strategy (responsible use of public money) to implementation by using less time with minimal or no disruption to traffic at all (Smith et al., 2014). After implementing the treatment, the whole network will be safer, smoother with significant noise reduction.

ii. Improved pavement condition - According to Smith et al. (2014) the typical approaches that most agencies apply to maintain their pavement networks are maintenance (routine and corrective) and rehabilitation. Routine and corrective maintenance are reactive since they treat existing deficiencies (distresses) whereas rehabilitation allows the pavement to deteriorate until the worst project rises to the top of the capital project list (worst first approach). Contrast to the typical approach pavement preservation improves the overall network pavement condition because of its best first approach principle; pavements in good condition are kept in the same condition, thereby delaying rehabilitation or reconstruction needs (Beatty et al., 2002; Shah et al., 2011; Smith et al., 2014; Van Dam et al., 2015).

iii. Increased safety - Safety of the roadways is a fundamental principle perceived by the public. Pavement safety is improved by applying treatments that involve polish resistant aggregates with macrotexture to increase wet-weather surface friction and avoid sliding and hydroplaning related traffic crashes (Smith et al., 2014). Pavement systems maintained in a good condition rides smoother with fewer defects that can jeopardize its safety; also work zone related crashes are reduced since it requires minimal or no disruptive repairs at all (Smith et al., 2014).

iv. Cost savings - Cost savings of pavement preservation are in terms of using less expensive treatments which extend the life of the pavement, delaying of more expensive options like major rehabilitation and reconstruction, and decreased user cost, vehicle operating costs and work zone crashes due to less time of lane closure time, smoother roads and few work zones (Smith et al., 2014) Pavement preservation strategies has saved the Michigan DOT about $700 million in their five years program (Smith et al., 2008).

For the agency to obtain the optimum benefits of the pavement preservations, Kercher (2011) suggest the following to be addressed/observed:

i. Selecting the right treatment to be applied to the right section and at the right time

ii. Up to date pavement management system for confident and informed decision making on the section to be treated, timing, cost associated, expected performance and future needs of the network

iii. Developing a long-term budget plan that will initially consider both the “worst first approach” and “best first approach” before shifting completely to the “best first approach”

iv. Involvement of trained personnel in all stages of the project; scope of work and contract agreement documents are well understood by contractors if developed/designed by personnel with engineering background and experience

v. Well defined quality control criteria, and threshold levels for payment and acceptance of work

2.3. Slab Jacking and Slab Stabilization

Slab jacking involves lifting/rising the slab in localized areas where slab settlement/depression has occurred due to poor foundation support to re-establish a smooth profile by using flowable material (Smith et al., 2014). Slab jacking is also known as mud jacking but due to the discovery of other materials apart from cement grouts such as polyurethane, the term mud jacking is becoming less common. Smith et al. (2014) recommend not to raise a slab more than 6 mm (0.25 in) past the neighboring slab level during material injection to avoid building up of excessive stresses which are likely to cause cracking.

Slab stabilization is a non-destructive concrete pavement restoration strategy which involves the injection of flowable material underneath the concrete slab through a 32 to 50 mm (1.25 to 2.00 in) drilled holes on the slab (American Concrete Pavement Association, 1994; Smith et al., 2014; Smith, 2005). According to Smith (2005) and American Concrete Pavement Association (1994), to avoid conical spalling at the bottom of the slab, the downward pressure on the pneumatic or hydraulic rotary percussion drill should be not more than 890 N (200 lbf.).

In granular subbases or subgrades, injection holes are drilled up to just below the concrete slabs while in stabilized base the injection holes go to the bottom of the stabilized base since voids are likely to form there (Lee and Shields, 2010; Smith et al., 2014). Sufficient holes should be drilled not near joints or cracks, but within voids region to ensure that the materials reach the voids. The drilling pattern of the holes may either be in the wheel path or in the centerline of the lane depending on the condition to be corrected (Lee and Shields, 2010; Su Jung et al., 2008).

Polyurethane is one among the material used in slab stabilization/jacking, others being cement-fly ash grouts and asphalt grouts. (Smith et al., 2014; Van Dam et al., 2015).

Polyurethanes used in slab stabilization/slab jacking is a high density expanding foam formed by blending two components referred as the “A side” which consist of methylene diphenyl diisocyanate and isocyanates with two or more functional groups (toluene diisocyanate and diphenylmethane diisocyanate), and the “B side” (or “Resin (R) side”) which is a combination of polyol compound (polymers with multiple hydroxyl group with repeating structure), catalysts and water (Chun and Ryu, 2000).

Polyurethane foams are either hydrophilic or hydrophobic depending on their ability to dissolve in water. Hydrophilic polyurethanes have a high affinity to water and cures to form flexible foams or gel. They react with water, to create a bond, making them useful for sealing leaks in cracks and joints. The expansion rate of hydrophilic is 5 to 7 times its original volume, making them not ideal for slab lifting or stabilization (Yu et al., 2013).

Hydrophobic polyurethanes are made to not react with either gaseous or liquid matter. With expansion rate of up to 20 times, low viscosity, high tensile and compressive strength, resistant to freeze/thaw cycles and low thermal conductivity; hydrophobic foams are suitable for PCC slab settlement mitigations (Yu et al., 2013). They are considered rigid foams due to their low water content, and once cured they tend to not shrink over time (Gaspard and Zhang, 2010; Yu et al., 2013).

Slab stabilization by polyurethane injection follows in permeation grouting or compaction grouting ground improvement type depending on whether the hydrophobic foam is a single component or two components respectively (Yu et al., 2013). Permeation grouting is mostly applied in asphalt roadways and for sealing water leaks through cracks on concrete structures, whereas compaction grouting is practical in filling voids and/or lifting concrete roadways, sidewalks, approach slabs, and sunken tanks (Yu et al., 2013).

Slab stabilization is intended to fill the voids in the layer supporting the concrete, not to raise the slab; by filling the voids deflection is reduced, and distress related deflections, such as pumping and faulting are also minimized (American Concrete Pavement Association, 1994; Lee and Shields, 2010; Smith, 2005; Smith et al., 2014). If the main purpose of the project is to raise or level settled concrete slabs, slab jacking should be opted. In cases where slab stabilization and slab jacking are performed simultaneously flowability of the material should be observed.

A successful slab stabilization strategy is a function of (1) accurate detection of voids, (2) suitable materials and quantity required (3) optimal time for stabilization, and (4) appropriate construction practices (American Concrete Pavement Association, 1994). In some situation, slab stabilization is accompanied by other pavement restoration treatments such as diamond grinding and slab jacking (Smith et al., 2014).

For optimum performance of slab stabilization, the technique should be used before the occurrence of distresses caused by loss of supports such as faulting, pumping and corner breaks (American Concrete Pavement Association, 1994; Smith et al., 2014).

Polyurethane injection is preferred over other slab stabilization/jacking treatment technique such as grout injection and mud jacking because of their lack of standard procedures, stresses induced in the slabs due to large access holes, grout spread limitation into voids, and curing time of the material before the lane is open to traffic (Brewer et al., 1994; Soltesz, 2002). The efficiency of the slab stabilization technique is influenced by the voids underneath the slab; excess injection of the material introduces stresses in the slab and accelerates the development of cracks.

2.4. Case Studies on Applications of HDP Foams

The Pennsylvania DOT used high-density polyurethane (HDP) to rehabilitate a section on U.S. Highway 402 of 9 km (5.60 mi)-four lane, divided highway, supported on an open-graded stone subbase; the intended task was to stabilize the open-graded stone subbase layer, mitigate faulting, and improve joint load transfer efficiency. The HDP foams were injected into the holes at a maximum flow rate and pressure of 272 kg/min (560 lb/min) and 378 kPa (54.82 psi) respectively (Vennapusa and White, 2015; Vennapusa et al., 2016).

The performance of the Pennsylvania DOT treated section assessed by Vennapusa and White (2015) identified the following; (1) average IRI increased from 1.70 m/km (107.71 in/mi) to 1.90 m/km (120.38 in/mi), indicating poor pavement surface levelling control, (2) spatial extent of foam propagation in the subbase layer ranged between 0.3 m (1.00 ft) and 1.0 m (3.28 ft) from the injection points, concentrated zones of foam mixed with subbase had low permeability, low stiffness, and high shear strength when compared to untreated areas, (3) falling weight deflectometer (FWD) tests indicated statistically significant improvement near cracks, load transfer efficiency (LTE) increased from about 15% before treatment to about 45% shortly after treatment and 86% after dowel bar retrofitting, no improvements were observed near slab joints or at mid-panel, (4) HDP injection minimized faulting of the cracks despite measurements from the robotic total station showing that pavement panels were raised by 6 mm (0.24 in) on average with a standard deviation of 3 mm (0.12 in), exceeding 1.3 mm (0.05 in) as in the project specification.

Soltesz (2002) evaluated the performance of URETEK® injected by the Oregon DOT to raise, stabilize and realign sections of Glenn Jackson Bridge and its adjacent concrete slabs. The test site was monitored for elevation changes, hole infiltration and water permeability, and compressive strength. In this project the following were found; (1) injected polyurethane raised the slab to the target profile, but slabs sunk up to 10.5 mm (0.41 in) after two years of injection, the cause of the settling was not investigated, and it was not known if it will continue, (2) HDP can penetrate through small openings such as 3.20 mm (0.13 in) due to its ability to flow, and protect the subgrade from water infiltration, and (3) compressive strength of HDP did not decrease after 23 days of exposure to air and ground condition.

The Wisconsin DOT used URETEK® material to rectify settled slabs near the bridge approach, the task took longer and more materials than expected, pavement ride quality was improved but fine cracks developed in the treated slabs. These cracks were likely caused by stresses induced during the injection process, and they were likely to reduce the service life of the slabs if left unattended (Al-Eis and LaBarca, 2007). The following were recommended for the use of URETEK® material for slab stabilization; (1) application of URETEK® is practical for high volume roads where lane closure time is very important, (2) due to likelihood of inducing cracks into slabs sagging in the middle, polyurethane injection may be substituted with slab replacement or concrete grouting for good performance of the slab, and (3) to reasonably estimate the cost associated with the procedure, ground penetrating radar (GPR) should be used to estimate the amount of material required to fill the voids (Al-Eis and LaBarca, 2007).

Gaspard and Zhang (2015) assessed the performance of polyurethane foam in the reduction of faulting by approximately 6.35 mm (0.25 in) on the jointed concrete pavement, the LA 1 by pass in Natchitoches, Louisiana an urban principal arterial roadway with the average daily traffic of 15,800, of which 20% were trucks. Its typical section consisted of a 230 mm (9 in) thick PCC pavement with a 150 mm (6 in) thick soil cement base course and asphaltic concrete shoulders, supported by group A-2-4 and A-4 soils. PCC slabs had faulted to about 25 mm (1 in) with IRI ranging from 2.37 to 7.10 m/km (150 to 450 in/mi). Pre-and post-measurements of faulting were measured using a high-speed profiler and manual faults measurements, IRI was measured using a high-speed profiler too, and the Australian Road Research Board (ARRB) walking profiler.

Moreover, in Gaspard and Zhang's (2015) study, FWD was used to measure LTE at the joints, void potential beneath the slab and slab deflection. In addition, to compare the free rise and confined polyurethane foams density and strength, polyurethane was injected in cylindrical 76.20 mm (3 in) diameter by 76.20 mm (3 in) height molds; and core samples were taken from the concrete slab and cement treated base course. Based on their findings, the polyurethane foam fault correction process was not recommended for pavement preservation as it neither improve the ride quality nor eliminates faulting as expected. Also, LTE was significantly reduced which was accompanied by deflection increases in the slab as well as in the joints.

To evaluate the effectiveness of the URETEK® method applied by the Michigan DOT, Opland and Barnhart (1995) conducted a study on three selected tests section of interstate I-75 in Monroe County, on trucks lane with 254 - 280 mm (10 –11 in) thick reinforced concrete slabs, resting on an open-graded base course. Improvements in the base support were significantly observed at areas where slabs were severely damaged or cracked as compared to where the cracks were hairline or open by 3.18 mm (0.13 in). However, in areas were the slabs were severely faulted the material raised the slab and provided a temporary base stability. Also, one year after injection of the material, ride quality and LTE at cracks and joints were approximately the same as before application of the material. This study recommended the use of URETEK® as an alternate (not a substitute) of mud jacking on concrete pavements supported on open-graded base course until adequate experience and knowledge on the limitations and capabilities of the material is gained by the DOT.

In 2011, the Missouri DOT applied URETEK® on a subbase and its underlaying layer to rapidly improve its load-bearing capacity before placing an asphaltic base layer. FWD tests conducted after several hours of complete deep injection showed an improvement in stiffness of about 40%, and after the application of Geogrid and a 95.25 mm (3.75 in) layer of HMA wearing course. FWD test results showed a stiffness increase of about 70% as compared to the benchmark tests. After five years of in service, FWD test resulted in an average back calculated subgrade modulus of 160 MPa (23,000 psi), an improvement of about 160% compared to the benchmark tests. In general, no individual location had stiffness below 140 MPa (20,000 psi) whereas several benchmarks had low stiffness moduli of up to 30 MPa (4,000 psi) (Boudreau et al., 2017).

On behalf of the Louisiana DOT, Gaspard and Morvant (2004) assessed the performance of URETEK® material for leveling and void filling on CRCP and bridge approaches; whereas on JPCP it was used to reduce faulting, filling voids and under seal. IRI was reduced from 33 to 68% on CRCP and bridge approach slabs depressions decreased by 50 mm (2 in). Cores obtained from CRCP and bridge approach slabs had dense polyurethane while those from JPCP had layers ranging from soft to dense. The study recommended the polyurethane injection process to be included as an alternative to other rehabilitation methods such as asphaltic concrete overlay and patching. It also recommended that polyurethane suppliers and contractor should develop a detailed laboratory testing protocol that will address the mechanical properties of the material under different curing and injection condition, long-term durability of the material under repeated traffic loading and environmental conditions, and field-testing method and quality assurance values.

To stabilize a section of I 86 in Hartford, Vermont showing indication of subsurface instability, the Vermont Transportation Agency opted to inject URETEK® 486. Based on the 2007 non-invasive and non-destructive testing (NDT) report of Applied Research Associates' Consultants, 50,350 kg (111,000 lbs.) of foams were planned to be used, the project was delayed to 2013 and 113232 kg (249,634 lbs.) were injected instead. Also, the FWD tests indicated improvements, although some locations which previously weresubsidence had high stiffness modulus compared to after injecting the foam. The site is still being studied for the agency to reach a conclusion on the use of URETEK® 486 for slab stabilization. However, the section had not required any maintenance, two years after injection of the material (Ellis, 2015).

2.5. Summary

Literature were reviewed on most common rigid pavement distresses, rigid pavement preservation strategies, slab stabilization/jacking and several case studies of previous projects which used HDP foams to stabilize and lift settled concrete slabs. These projects have shown that slab drop-off (faulting) is caused by loss of foundation support due to either weak base/subgrade, poor compaction and/or erosion of subgrade materials due to pumping. To rectify defects associated with distresses, DOTs have established preservation strategies which specify when and where a specific treatment(s) should be applied. Unattended distress not only they deteriorate the condition of the pavement but also poses safety hazards to road users. Slab stabilization and slab liftings are among the pavement treatments, they are applied to fill voids underneath the pavement and level the sunken slab respectively. HDP foams are showing to be the most effective materials for slab stabilization/jacking.

DOTs have conflicting experience on the effectiveness of HDP foams in rectifying faulting defects in JPCPs. While others have reported an increase of IRI, joint faulting, and LTE, some experienced a decrease of these indicators after application of polyurethane materials. Early hairline cracks were observed due to stresses induced because of excessive injection of the materials. Due to lack of detailed ground investigation, some DOTs used more materials than specified in the project documents.

This study seeks to evaluate the performance of PolyLevel®, HDP foams injected by the Tennessee DOT underneath settled concrete slabs with a thickness of 250 mm (10 in), resting on granular base by assessing their transverse joint faulting before and after application of the material. To obtain transverse joint faulting measurements of the treated sections, raw longitudinal profile data were collected by a standard inertial profiler and analyzed using ProVAL software. In general, the performance of polyurethane treated section is significantly affected by the soundness of the slabs, type of foundation soils, and traffic loading and volume.

CHAPTER III

METHODOLOGY

This study evaluates the performance of five concrete pavement sections treated with high-density polyurethane (HDP) material. Transverse joint faulting among the concrete slabs is analyzed to assess the performance of the polyurethane material over time. Apart from other distresses such as cracking or spalling, transverse joint faulting is among the factors influencing smoothness of rigid pavement surface.

The study was performed on raw longitudinal profile data collected for a Tennessee DOT project from March 2015 to March 2018. The data was collected by a Tennessee DOT contractor using a standard inertial profiler at a sampling interval of 26.28 mm (1.03 in) before and after injection of HDP foams. Profile viewing and analyzing (ProVAL) software was used to analyze the raw profile data to obtain transverse joint faulting measurements before and after application of the polyurethane material.

3.1. Data Collection

As alluded earlier, this research was conducted on five sections of U.S. Interstate I-75 and I-24 as shown in Table 3.1 with distances ranging from 482.8 m (0.3 mi) to 3220 m (2.0 mi).

The sections were constructed of plain jointed concrete with a slab thickness 250 mm (10 in), resting on granular base. As per 2017 TDOT traffic data log the average annual daily traffic

(AADT) of treated sections on I-24 East and I-24 West was 134740 vehicles per day and 119930 vehicles per day respectively, with trucks representing 18.50% of the AADT. The AADT of treated sections on I-75 was 77150 vehicles per day; of which 14.5% were trucks. In Table 3.1, the treated lanes are counted from the left of the direction of travel; for instance, on section I-24 West, lane No. 2 was the one treated with HDP foams.

Table 3.1 Sections Treated With Polyurethane Material

Section ID	Start Mile	End Mile	Distance(mi)	Treated Lane ID
I-24 West	179.50	178.20	1.30	2
I-24 East	182.35	183.00	0.65	3
I-24 East	Moore Bridge	McBrien Bridge	0.30	2
I-75 North	7.00	9.00	2.00	3
I-75 South	9.00	7.00	2.00	3

Raw longitudinal profiler data were collected before and after application of the HDP foams using a standard inertial profiler. These data are analyzed using the automated faulting measurement (AFM) and ride quality module to transverse joint faulting and roughness indices (IRI and MRI) respectively.

Apart from the raw longitudinal profile data, ProVAL AFM module requires joint spacing, segment length, and joint window (i.e., uncertainty for joint location) as inputs for calculating the transverse joint faulting. The joint spacing and joint width were retrieved from TDOT standard and it specifies a joint spacing of 4.57 m (15 ft) and joint width of 25 mm (1 in). The default value for joint window in ProVAL is 50 mm (2 in), and it is adopted in this study.

3.2. Data Analysis

Raw longitudinal profile data collected using a standard inertial profiler was imported into ProVAL software for analysis. In ProVAL, sections were divided into segments of 100 m length, if the analysis is conducted in U.S. customary units, the segment length shall be 0.1 mi (American Association for State and Highway Transportation Officials, 2017).

The number of segments (samples) ranged from five to thirty-three depending on the length of the section. I-24 East_Moore is the shortest section; hence it has the fewest number of segments (i.e., five) whereas I-75 sections are the longest, with thirty-three segments.

The ride quality and AFM module in ProVAL was used to analyze the raw longitudinal profile data for IRI and transverse joint faulting respectively. After the ProVAL analysis, the outputs were exported into Excel® spreadsheets for statistical analysis, which was achieved by using R programming software.

3.3. Joint/Cracks Detections and Faults Computation

The ProVAL AFM module has three techniques for detecting joints/cracks in the longitudinal profile, which are down spike, step, and curled edge (Chang et al., 2012; The Transtec Group, 2016). These three techniques are now briefly described.

The down spike method is partly based on the FHWA curl-wrap method. Developed by Steve Karamihas for the FHWA, the FHWA curl-wrap method follows these four steps; profile filtering, identification of deepest dips, dip count assembling across the data count, and identification of joint location. It is effective for multiple profile runs collected at small sampling interval with clear down ward spikes at joints. The down spike method is suited for slabs with down spike with less or no sealant at all between the joints, and longitudinal profile collected at small sampling interval.

The step method is based on the Mississippi DOT joint/crack identification method. It was developed using longitudinal profile data collected at a sampling interval of 12.70 mm (0.50 in). Joint faulting is detected if the elevation differences among the adjacent sampling is greater than 2.03 mm (0.08 in). The Mississippi DOT approach is effective for network level joint faulting survey. However, there is no study which has confirmed that it is efficient beyond the required 12.70 mm (0.50 in) sampling interval.

The curled edge method was developed by the ProVAL team for differential elevation due to slab curling. Curling is a deformation that occurs due to the difference in temperature across the depth of a concrete slab. Apart from stresses induced in the slab due to temperature variation, slab curling also affects the surface smoothness of the pavement. On a half car roughness index, impacts of slab curling are as high as about 0.63 m/km (39.92 in/mi), with an average of 0.16 m/km (10.14 in/mi) (Chang et al., 2010).

In this study, the down spike method was used because prior to sections' treatment with polyurethane, sections showed indication of spikes, there was no sign of slab curling. The step method was not used since the longitudinal profiles were collected at a sampling interval of 26.28 mm (1.03 in), and no studies have shown the efficiency of the step method beyond its required sampling interval of 12.7 mm (0.50 in). The following are the steps in analyzing of longitudinal profiles as described in AASHTO R 36-17 and Chang et al. (2012) for joints or cracks identification:

i. After collection of longitudinal profiles along the section of interest using a high-speed inertial profiler; anti-smoothing filtering is performed using a moving average filter at a cutoff of 250 mm (9.84 in)

ii. The filtered profile is normalized by its root mean square to obtain unitless spike profile

iii. Identification of locations in which the spike profile values exceed the threshold values (the starting threshold value is -4)

iv. Values from (iii) above are screened to differentiate between joints and cracks

After identifying joints and/ or cracks, faulting is computed based on AASHTO R 36-17 (Method A) as follows:

i. A profile segment that centers a joint with a length of 2438 mm (96 in) is cropped

ii. The profile slices for the approach and departure slab is separated into two equal slices of 1219 mm (48 in)

iii. For the approach slab slice profile, the area close to the joint is masked based on the joint window input and least square fitting is performed. The fitting extends to the departure side of the faulting for an offset between 76 mm and 226 mm (3 in and 8.9 in)

iv. For the departure slab slice profile, the area close to the joint is masked based on the joint window input and least square fitting is performed. The fitting extends from the downstream end of the slice toward the joint.

v. Elevations at all data points within an offset between 76 mm and 226 mm are recorded. As shown in Figure 3.3 the elevation point from the fitted line of the approach slab slice is recorded as P_1^i, and its corresponding elevation points from the departure slab slice as P_2^i.

vi. The faulting value is computed by averaging the difference between the elevations data points (P_1^i and P_2^i) obtained in iv above.

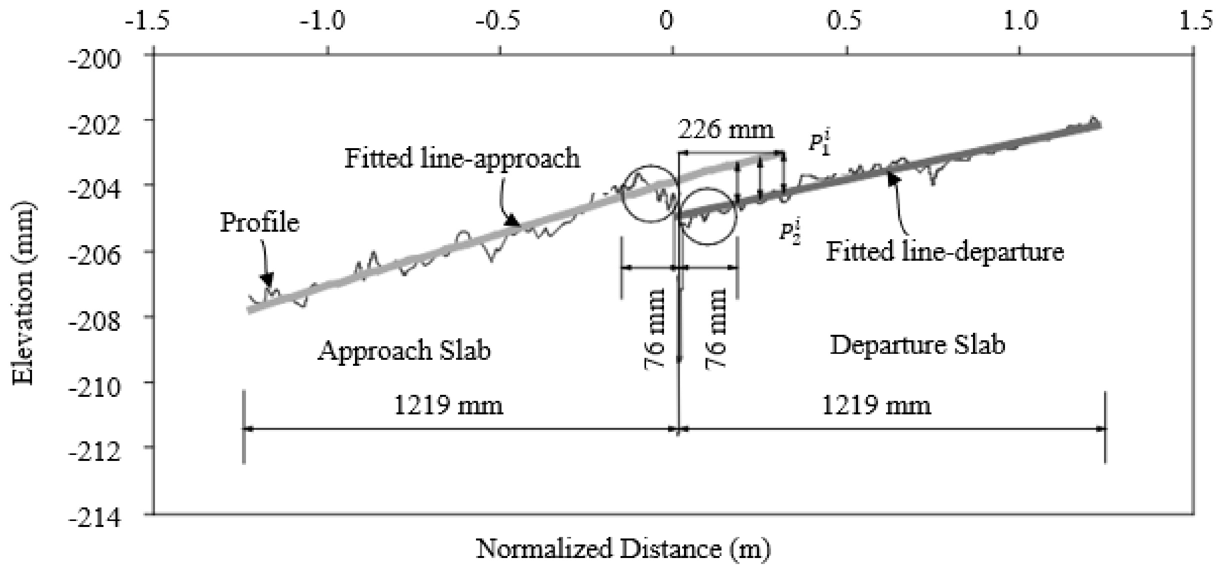


Figure 3.1 Cropped profile slices curve fitting and faulting computation (Source: Chang et al. [2012])

The ProVAL AFM module gives three outputs when exported into Excel® spreadsheets; joint locations which indicate where joints/cracks are located, faulting summary that summarizes maximum and accumulated faulting in every segment for the entire test section, and faulting details which show faulting value at every joint and/or crack detected.

CHAPTER IV

RESULTS AND ANALYSIS

Trigger levels for highway rehabilitation/maintenance are established by state DOTs or the FHWA depending on the jurisdiction under which the roadway falls. Threshold levels for concrete pavement condition rating indicators such as roughness, mean joint faulting and slab cracking are defined and established; road section maintenance/repair needs should be addressed when the threshold values exceed the trigger level set for the particular road functional class. All the three indicators mentioned above contribute to pavement ride quality.

According to Smith et al. (2014), transverse joint faulting significantly affects the ride quality of the pavement when it's in the range of 2 mm to 3 mm (0.08 to 0.12 in). IRI and joint faulting rating threshold values for the National Highway Systems (NHS) are shown in Table 4.1 as established by the FHWA [1 m/km is equivalent to 63.36 in/mi].

Table 4.1 Pavement Condition Metric Thresholds

Rating	Good	Fair	Poor
IRI (m/km)	<1.50	1.50 - 2.70	>2.70
Joint Faulting (mm)	<2.50	2.50 - 3.80	>3.80

Source: Constable and Blades (2017)

Longitudinal profile data collected using the standard inertial profiler was analyzed by ProVAL software in its AFM module to automatically compute transverse joint faulting. Studies have shown that AFM results/values are not statistically different from those obtained by using an absolute manual faultmeter such as the Georgia faultmeter (Chang et al., 2012). Also, AFM has the advantage of being safe as it does not expose the crew conducting the survey to traffic and no lane closure or traffic control is required since the high-speed inertial profiler can travel at the prevailing traffic speed.

The AFM module can be applied at all levels; AASHTO R 36-17 specifies a minimum sampling interval of 19 mm (0.75 in) for a site-specific project and 38 mm (1.50 in) for a network level project, profiles must be recorded on both the left and right wheel path/track.

4.1. Results

Transverse joint faulting is considered positive when the approach slab is higher than the departure slab, and vice versa is true for negative faulting. The overall transverse joint faulting values of the sections were calculated by averaging the absolute individual faulting values at every joint/crack detected in the specific section. The raw longitudinal profile data collected before and after application of HDP foams were analyzed in ProVAL to obtain transverse joint faulting and IRI. Table 4.2 and 4.3 summarize transverse joint faulting values and IRI measurements of the five treated sections. Values were computed for both wheel track (left and right), and then averaged.

In this study, the mean values were used for judging whether the transverse joint faulting or IRI increased/decreased after application of HDP foams. The decrease in either transverse joint faulting or IRI indicates improvement in ride quality, while vice versa is true for the decrease. Statistical analysis was conducted to evaluate if the changes were significant, and its results are presented in the progressing section.

Table 4.2 Treated Sections Transverse Joint Faulting Prior and Post Injection of Polyurethane

Highway section ID	I-75 South			I-75 North			I-24 East_Moore			I-24 East_182			I-24 West		
	Transverse Joint Faulting (mm)														
Wheel track profile	Left	Mean	Right	Left	Mean	Right	Left	Mean	Right	Left	Mean	Right	Left	Mean	Right
Before application	1.51	1.54	1.58	1.45	1.38	1.32	1.71	1.83	1.95	3.64	3.29	2.94	1.16	1.39	1.61
One week after application	1.81	1.56	1.31	1.33	1.30	1.27									
Thirteen months after application	1.92	1.68	1.45	1.59	1.54	1.49									
One month after application							1.24	1.34	1.45						
Thirteen months after application							0.72	0.72	0.72						
Twenty-nine months after application							0.67	0.70	0.73						
One week after application										3.44	3.26	3.09	0.67	0.73	0.79
Eight months after application										2.34	2.34	2.34	1.18	1.22	1.25
Nineteen months after application										3.08	3.00	2.92	1.50	1.45	1.39

Table 4.3 Treated Sections IRI Readings Prior and Post Injection of Polyurethane

Highway section ID	I-75 South			I-75 North			I-24 East_Moore			I-24 East_182			I-24 West		
	IRI (m/km)														
Wheel track profile	Left	Mean	Right	Left	Mean	Right	Left	Mean	Right	Left	Mean	Right	Left	Mean	Right
Before application	1.83	1.84	1.84	1.76	1.68	1.60	2.14	2.19	2.23	2.91	2.83	2.75	2.13	2.19	2.25
One week after application	1.81	1.80	1.79	1.72	1.64	1.56									
Thirteen months after application	1.94	1.87	1.79	1.91	1.82	1.73									
One month after application							2.04	2.05	2.06						
Thirteen months after application							1.75	1.75	1.75						
Twenty-nine months after application							1.68	1.68	1.68						
One week after application										2.85	2.89	2.92	1.99	2.06	2.13
Eight months after application										2.54	2.55	2.55	1.97	2.04	2.10
Nineteen months after application										2.91	2.86	2.8	2.06	2.09	2.12

4.2. Statistical Analysis

The maximum transverse joint faulting values at every 100 m segment data was used for a paired t-test using the R programming software. These data are assumed to be normally distributed, as they were tested and their quantile-quantile (qq) plots are presented in Appendix C. The paired t-tests were carried out to assess statistical significant changes at a confidence interval of 95% in the means of the maximum faulting values before and after application of the polyurethane material. The null hypothesis assumes that there is no significant difference between the means and vice versa is true for the alternative hypothesis. The null hypothesis is rejected if the p-value is below 0.05 and it indicates that the difference is statistically significant.

Table 4.4 shows a paired t-test of the maximum faulting values of before versus those collected one week after application of HDP foams on section I-75 South and I-75 North. For both sections, the p values are greater than 0.05 indicating that there were no statistical significant changes between before and one week after application of the material.

Table 4.4 Paired t-tests for Before Application Against One Week After Application for I-75 South and North

	Highway section ID	Wheel track	P-value	Remarks
One week after application	I-75 South	Left	0.5433	Accept the null hypothesis
		Right	0.1399	Accept the null hypothesis
	I-75 North	Left	0.7124	Accept the null hypothesis
		Right	0.1886	Accept the null hypothesis

Results of the paired t-test done on section I-75 South and I-75 North thirteen months after application of polyurethane material are shown in Table 4.5. As it was one week after application of the material (Table 4.4), still there were no statistical significant changes of transverse joint faulting thirteen months after application of HDP foams as the p values are greater than 0.05.

Table 4.5 Paired t-tests for Before Application Against Thirteen Months After Application for I-75 South and North

	Highway section ID	Wheel track	P-value	Remarks
Thirteen months after application	I-75 South	Left	0.3839	Accept the null hypothesis
		Right	0.1451	Accept the null hypothesis
	I-75 North	Left	0.1344	Accept the null hypothesis
		Right	0.3456	Accept the null hypothesis

Section I-24 East_Moore is the shortest sections of all the five test sections, therefore it has the fewest segments number in it (i.e., five segments). The paired t-test conducted on the data obtained one month after application of the material on this section showed that there were no significant changes since the p-values are greater than 0.05 (Table 4.6). However, the paired t-test indicated that the changes in transverse joint faulting are statically significant thirteen and twenty-nine months after application of the material (p values are smaller than 0.05).

Table 4.6 Paired t-tests Analysis of I-24 East_Moore

	Wheel track	P-value	Remarks
One month after application	Left	0.1025	Accept the null hypothesis
	Right	0.0699	Accept the null hypothesis
Thirteen months after application	Left	0.0363	Reject the null hypothesis
	Right	0.0239	Reject the null hypothesis
Twenty-nine months after application	Left	0.0031	Reject the null hypothesis
	Right	0.0054	Reject the null hypothesis

Table 4.7 shows a paired t-test of the maximum transverse joint faulting values of before versus those collected one week after application of HDP foams on section I-24 East_182 and I-24 West. For both sections, the p values are greater than 0.05 indicating that there were no statistical significant changes between before and one week after application of the material.

Table 4.7 Paired t-tests for Before Application Against One Week After Application for I-24 East_182 and I-24 West

	Highway section ID	Wheel track	P-value	Remarks
One week after application	I-24 East_182	Left	0.9928	Accept the null hypothesis
		Right	0.3985	Accept the null hypothesis
	I-24 West	Left	0.1715	Accept the null hypothesis
		Right	0.1871	Accept the null hypothesis

Eight months after application of the material, the paired t-test proved that the change was statically significant on the left wheel track of section I-24 East_182 (p value<0.05). The changes remained statically insignificant for the right wheel track of I-24 East_182, and on both wheel tracks of I-24 West (Table 4.8).

Table 4.8 Paired t-tests for Before Application Against Eight Months After Application for I-24 East_182 and I-24 West

	Highway section ID	Wheel track	P-value	Remarks
Eight months after application	I-24 East_182	Left	0.0036	Reject the null hypothesis
		Right	0.2923	Accept the null hypothesis
	I-24 West	Left	0.3994	Accept the null hypothesis
		Right	0.7236	Accept the null hypothesis

The change in section I-24 East_182 and I-24 west had remained statically insignificant (p values>0.05) nineteen months after application of HDP foams on these sections (Table 4.9).

Table 4.9 Paired t-tests for Before Application Against Nineteen Months After Application for I-24 East_182 and I-24 West

	Highway section ID	Wheel track	P-value	Remarks
Nineteen months after application	I-24 East_182	Left	0.3197	Accept the null hypothesis
		Right	0.2173	Accept the null hypothesis
	I-24 West	Left	0.4771	Accept the null hypothesis
		Right	0.9411	Accept the null hypothesis

The paired t-test results from Table 4.8 to 4.9 were summarized in Appendix B. To assess whether there were changes for both the left and right wheel track, the AND logic principle can be applied whereas the term “Yes” and “No” can be considered as “True” and “False” respectively.

4.3. Discussion

Before application of HDP foams, section I-75 South had transverse joint faulting of 1.54 mm (0.06 in). One week after application of HDP foams the transverse joint faulting increased by 1.30% and by 9.09% thirteen months later (Table 4.2). However, the increase was statically insignificant for both cases at significance level of 0.05 (Table 4.4 and Table 4.5).

In Table 4.2, the transverse joint faulting of I-75 North was 1.38 mm (0.05 in) before application of the material. It decreased by 5.80% one week after application of the material and increased to 1.54 mm (0.06 in) thirteen months later. Table 4.4 and Table 4.5 shows that these changes were not statistically significant at a confidence interval of 95%.

Section I-24 East_Moore had transverse joint faulting of 1.83 mm (0.07 in) before application of polyurethane material. Analysis conducted on data collected one month after application of the polyurethane material showed that the transverse joint faulting decreased to 1.34 mm (0.05 in). More ever, in Table 4.2 it is presented that the transverse joint faulting continued to decrease to 0.72 mm (0.028 in) and 0.70 mm (0.027 in) thirteen and twenty-nine months after application of HDP foams. The paired t-test conducted at a confidence interval of 95%, showed that the changes were statistically insignificant for measurement taken one month after application of the material, but significant for measurements performed thirteen and twenty-nine months after application of the material (Table 4.6).

Section I-24 East_182 and I-24 West had transverse joint faulting of 3.29 mm (0.13 in) and 1.39 mm (0.055 in) respectively before injection of HDP foams (Table 4.2). The transverse joint faulting decreased in both sections one month after application (3.26 mm on I-24 East_182 and 0.73 mm on I-24 West). It continued to decrease on I-24 East_182, eight and nineteen months after application while on I-24 West it increased nineteen months after application. In general, the decrease or increase in transverse joint faulting after application of the material in these sections was statistically insignificant at a significance level of 0.05 (Table 4.7, Table 4.8 and Table 4.9).

Application of polyurethane materials caused both increase and decrease in transverse joint faulting. Variation of transverse joint faulting over time was plotted using data in Table 4.2, and appended in appendix A. In summary, the increase in transverse joint faulting is statistically insignificant on I-75 South, I-75 North (thirteen months after application), and I-24 West (eight and nineteen months after application). The decrease on transverse joint faulting is insignificant on I-75 North (one week after application) and on I-24 East_182, but its significant on I-24 East_Moore thirteen and twenty-nine months after application of HDP foams. Therefore, section I-24 East_Moore is the only section that its transverse joint faulting decreased significantly (Thirteen and twenty-nine months after application of HDP foams) but, it has very few sample space (segments) making its results not statistically viable.

Although, the transverse joint faulting values increased for some sections such as I-75 South, I-75 North and I-24 West, their values are still in the acceptable range as per the FHWA threshold levels in Table 4.1.

Furthermore, maximum transverse joint faulting and mean IRI values in each 100 m segments from all the five sections were combined and correlated to obtain the relationship between the two indicators, for the left and right wheel track/path. Figure 4.1 and 4.3 show that mean IRI increases with increase in maximum transverse joint faulting for both wheel paths. Positive y-intercepts (i.e., 1.0948 and 1.2045) indicates that IRI is affected partly with transverse joint faulting. Other distress such as spalling, and punchouts have an effect on IRI too. In general, R-squared values of 0.6639 and 0.5867 indicate that the simple linear regression model/equation fits the data well. Also, the residuals against the fitted value (transverse joint faulting) plots are approximately evenly spread and randomly distributed above and below 0, which further shows that the linear regression equation is a good fit of the data (Figure 4.2 and 4.4). The relation between the maximum joint faulting and IRI may be explained by using this simple linear equation.

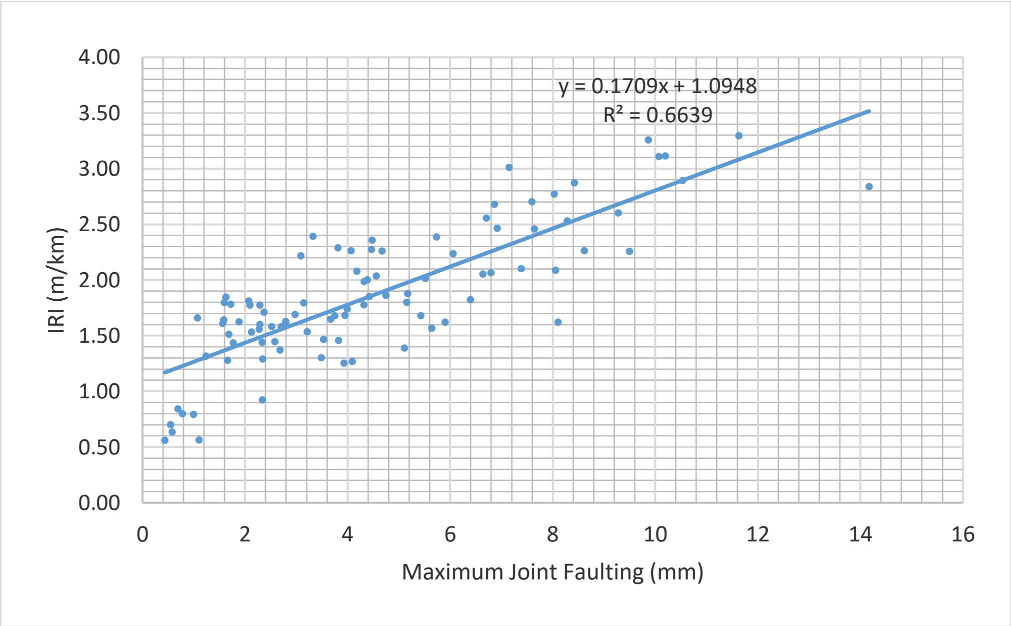


Figure 4.1 IRI against maximum transverse joint faulting for left wheel truck

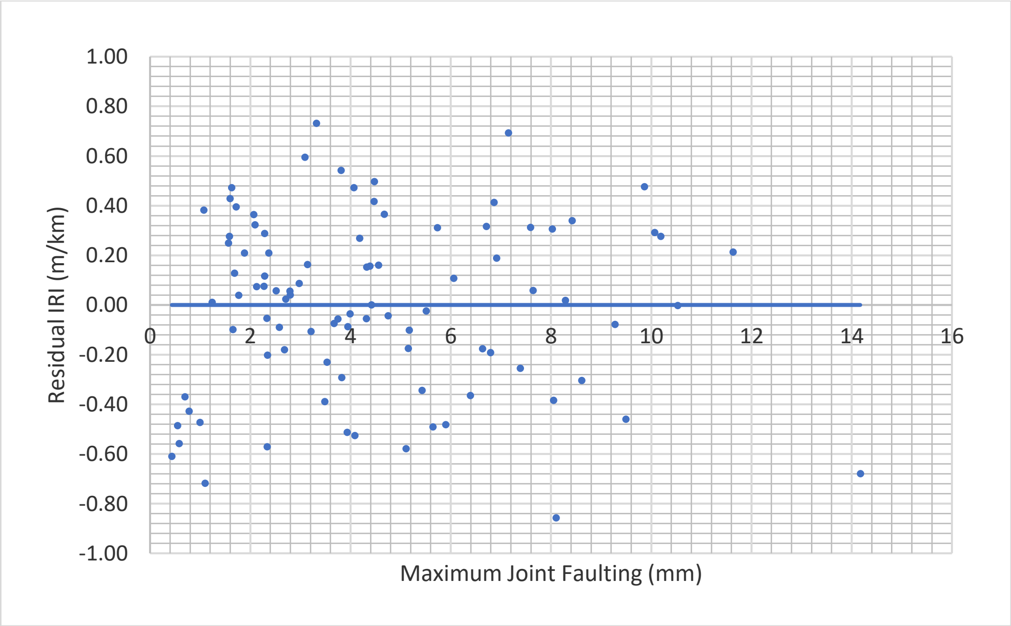


Figure 4.2 Residual IRI against maximum transverse joint faulting for left wheel truck

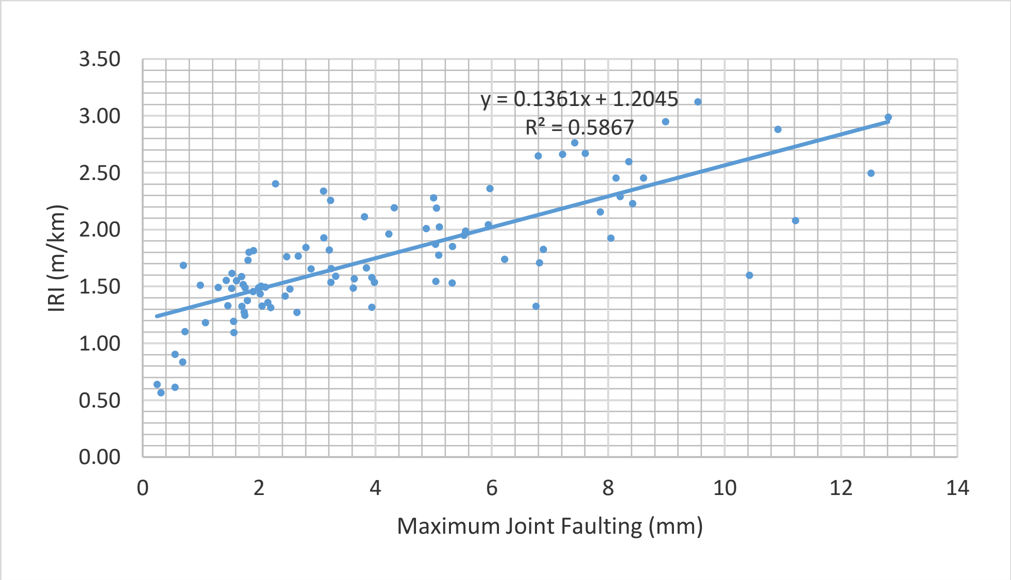


Figure 4.3 IRI against maximum transverse joint faulting for right wheel truck

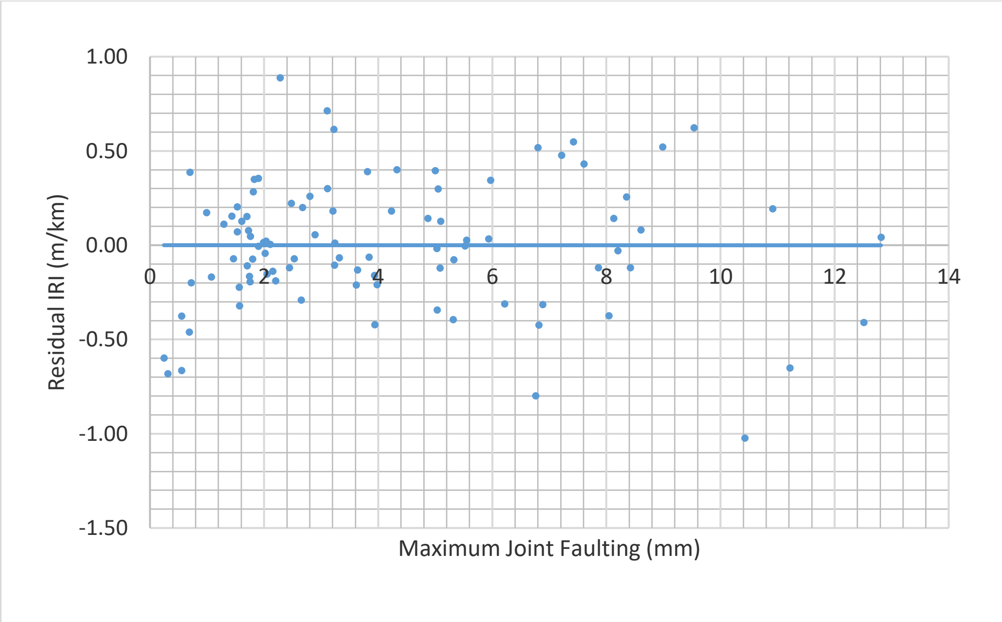


Figure 4.4 Residual IRI against maximum transverse joint faulting for right wheel truck

The overall absolute transverse joint faulting and mean IRI of the five sections in Tables 4.2 and 4.3 respectively were combined to obtain more samples for statistical correlation. The aggregated data is plotted as shown in Figure 4.5 to obtain the relationship between IRI and transverse joint faulting. It is observed that IRI increases with the increase of the mean transverse joint faulting, and they are polynomially related with a coefficient of determination of 0.83. The evenly distributed residuals above and below the zero line in Figure 4.6 further indicates that the model fits the data properly.

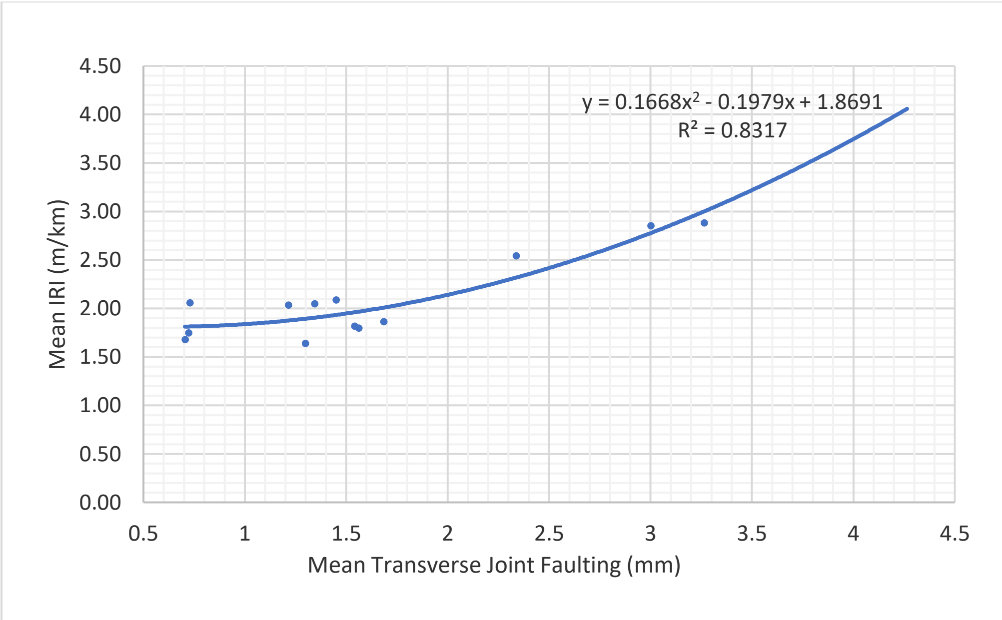


Figure 4.5 Mean IRI against mean transverse joint faulting

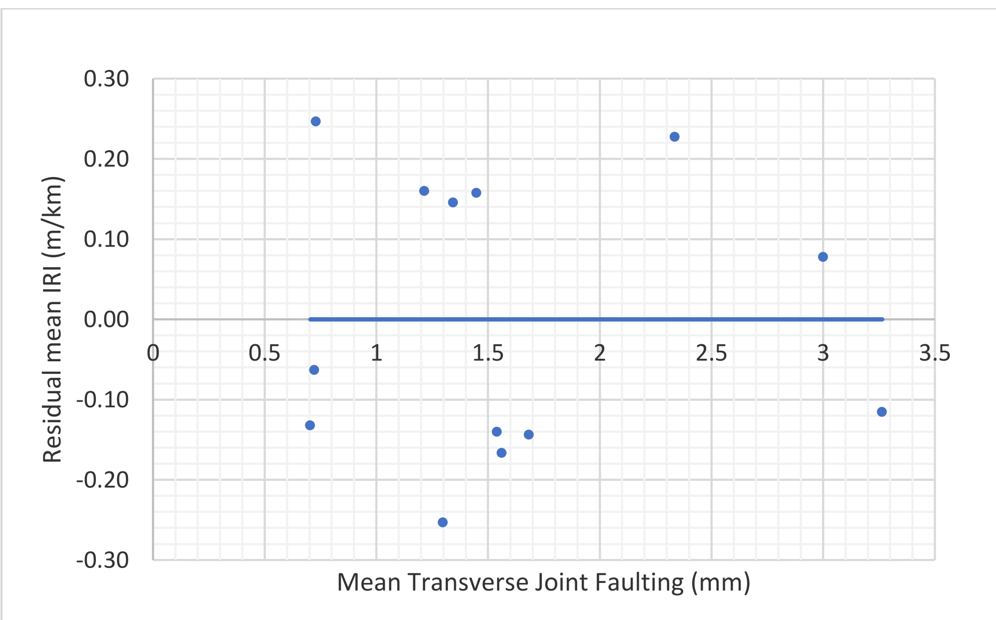


Figure 4.6 Residual mean IRI against mean transverse joint faulting

IRI data collected from the year 2005 to 2014 (prior to the application of the material) were obtained from the Tennessee DOT and analyzed. The IRI data obtained were from lane 1 of I-24 East from mile marker 11.3 to 12.1. There was no maintenance/repair performed on the section in this period. This data was plotted in Figure 4.7 as IRI versus time (in years). The fitted curve on Figure 4.7 indicates that IRI increases over time if no maintenance is applied.

The Tennessee DOT can use Figure 4.7 in timing maintenance need of its concrete pavement sections on I-75 and I-24, if it uses IRI as one of its trigger criteria. A validation of the fitted performance prediction model will be needed for it to be applied on different traffic conditions.

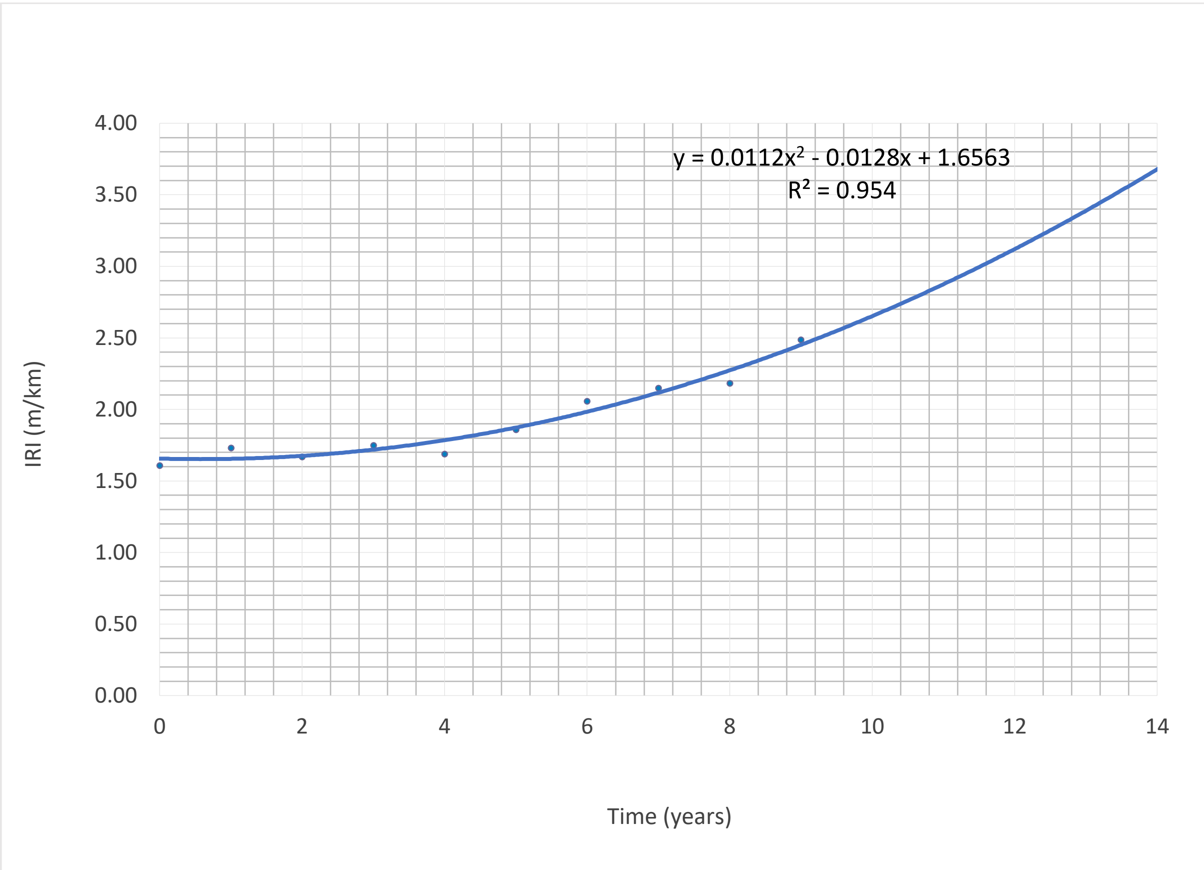


Figure 4.7 Overall IRI trend over time

Also, the long-term performance of polyurethane treated sections before it requires any maintenance or reconstruction work can be roughly forecasted by using curves fitted in Figure 4.5 and 4.7 or their respective model equations. In this case, the Tennessee DOT can choose either IRI or mean transverse joint faulting as its trigger criteria. However, the prediction will be affected by factors such as the soundness of the slab, traffic type and volume, and subsurface conditions.

CHAPTER V

CONCLUSION AND RECOMMENDATIONS

The aim of this study was to assess the performance of HDP foams injected in jointed concrete pavements by evaluating the transverse joint faulting of the treated sections. Transverse joint faulting and IRI readings were computed from the raw longitudinal profiles collected using a high-speed inertial profiler. The raw profile data were analyzed by the AFM and ride quality modules in ProVAL software.

Before application of HDP foams four sections (I-75 South, I-75 North, I-24 East_Moore and I-24 West) had mean IRI ranging between 1.68 m/km (106.44 in/mi) and 2.19 m/km (138.76 in/mi) and rated to be in a “fair” condition. I-24 East_182 had mean IRI of 2.83 m/km (179.31 in/mi) and rated to be in “poor” condition. The overall transverse joint faulting of all the five sections was in a “fair” condition as it ranges from 1.50 mm to 3.80 mm (0.06 in to 0.15 in). After application of the material both the mean IRI and transverse joint faulting have remained in the same group range as they were before application (fair condition).

All the sections have remained in an acceptable condition range except for I-24 East_182, which is in a poor condition. The overall transverse joint faulting of the treated sections decreased with time (one week and/ or eight months after application) and increased a year later on I-75 South (by 9.09%), I-75 North (by 10.39%) and I-24 West (by 4.32%); except for the I-24 East_Moore and I-24 East_182 which decreased by 60.66% and 8.81% respectively after application of HDP foams.

When transverse joint faulting values before application were statistically compared with after application; the changes were statistically significant on I-24 East_Moore and insignificant on the remaining sections. However, the sample spaces of I-24 East_Moore was very small making them unrealistic for statistical analysis.

In general injection of HDP foams underneath the pavement slabs neither did improve nor retrogress the condition of the sections but maintained it in their state before application of the material. Therefore, for a pavement section in a poor condition (transverse joint faulting >3.80 mm or IRI >2.70 m/km) HDP foams injection is not the best option, as the section will remain in the same state even after application of the material. However, state DOTs can apply HDP foams on section with good or those transiting to a fair condition to extend the life of the pavement.

This study recommends the following:

i. Prior injection of the material, a detailed ground investigation of the damaged pavement section must be carried out to establish the causes. Filling of voids requires different proportioning of diisocyanates and polyols from rising/ leveling a settled slab.
ii. Development of a standardized protocol for selecting pavement sections suitable for treatment with HDP foams.
iii. Contractors should use sophisticated leveling equipment, instead of the adjacent slab as the benchmark, to avoid accumulation of errors due to overcorrection
iv. Progressive monitoring of the sections to capture a full long-term performance of the material, and possible appropriate treatments after their service life time.

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APPENDIX A

TRANSVERSE JOINT FAULTING VARION OVER TIME

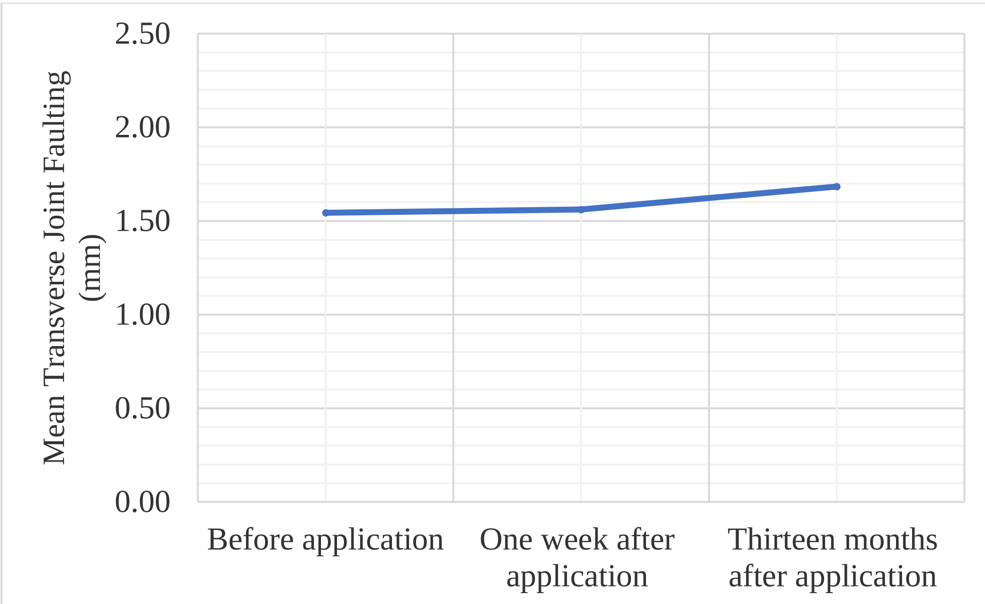

Figure A1 Variation of mean transverse joint faulting over time for section I-75 South

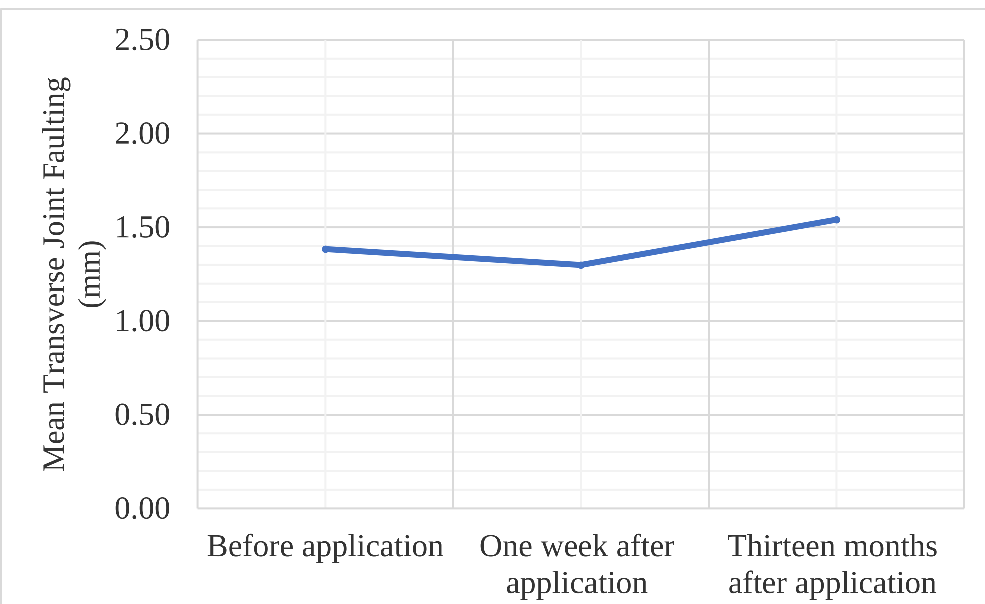

Figure A2 Variation of mean transverse joint faulting over time for section I-75 North

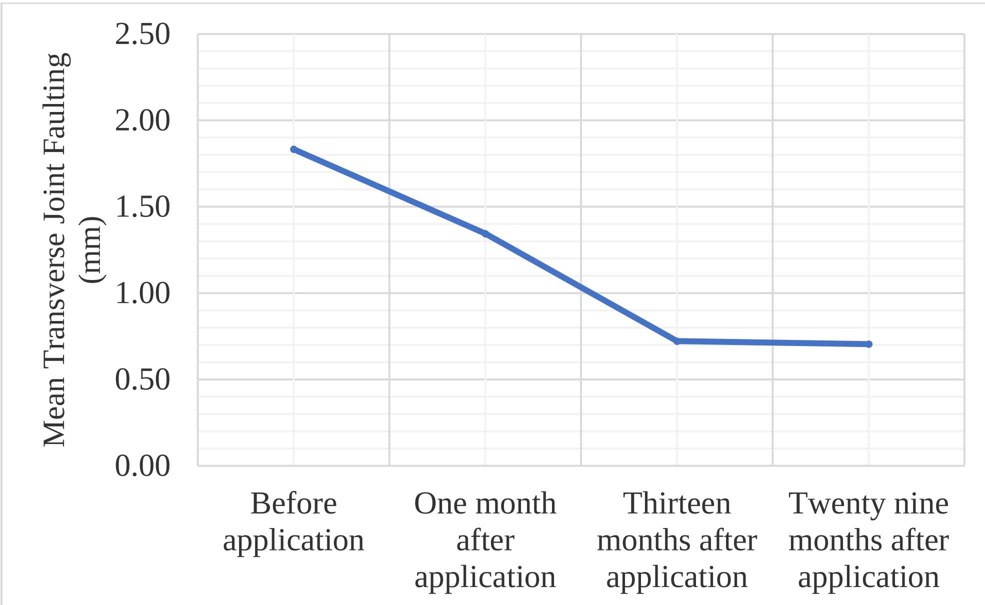

Figure A3 Variation of mean transverse joint faulting over time for section I-24 East_Moore

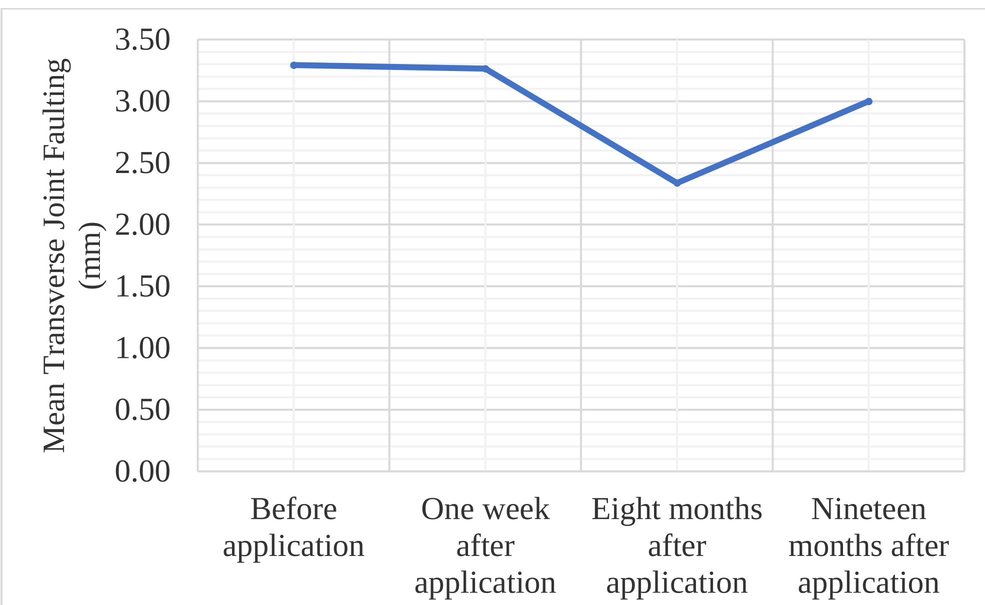

Figure A4 Variation of mean transverse joint faulting over time for section I-24 East_182

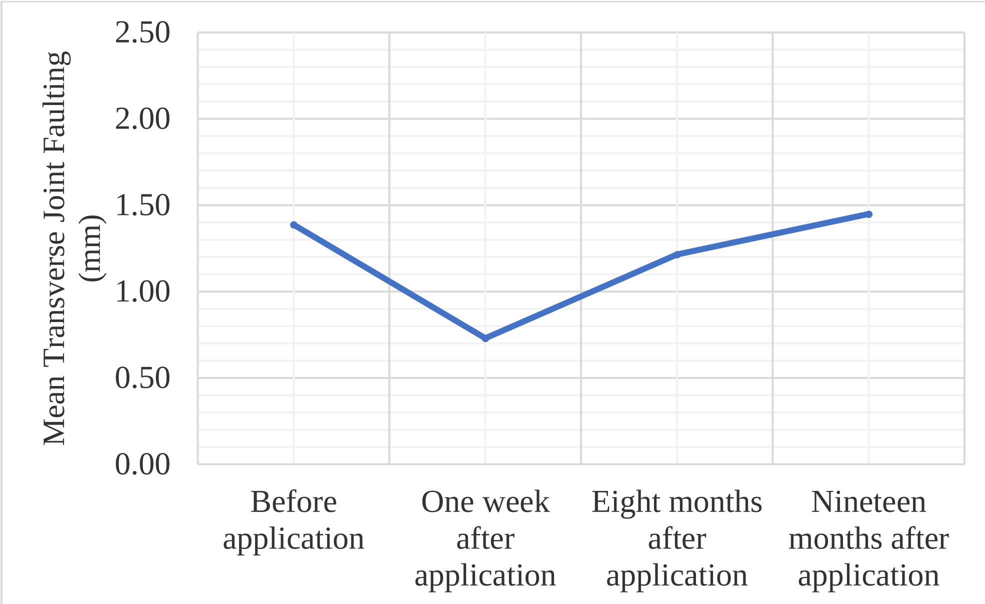

Figure A5 Variation of mean transverse joint faulting over time for section I-24 West

APPENDIX B

PAIRED T-TESTS ANALYSIS OF TRANSVERSE JOINT FAULTING FOR THE BEFORE AGAINST AFTER APPLICATION OF POLYURETHANE MATERIA

Highway section ID	I-75 South		I-75 North		I-24 East_Moore		I-24 East_182		I-24 West	
Statistically Significant Changes?										
Wheel track	Left	Right	Left	Right	Left	Right	Left	Right	Left	Right
One week after application	No	No	No	No						
Thirteen months after application	No	No	No	No						
One month after application					No	No				
Thirteen months after application					Yes	Yes				
Twenty-nine months after application					Yes	Yes				
One week after application							No	No	No	No
Eight months after application							Yes	No	No	No
Nineteen months after application							No	No	No	No

APPENDIX C

QUANTILE-QUANTILE PLOTS

SECTION I-75 SOUTH

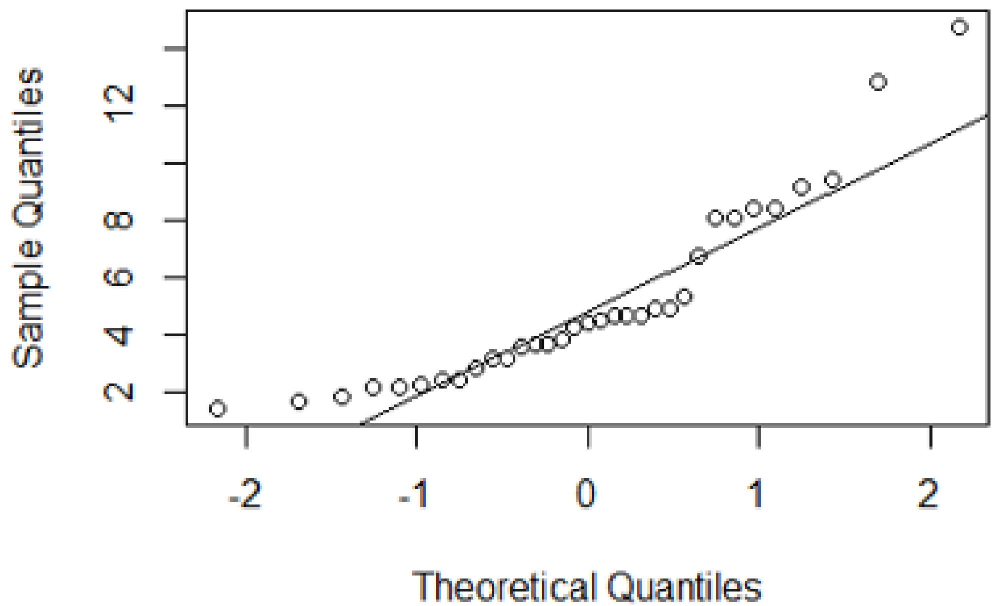

Figure C1 Left wheel track_Before application of HDP foams

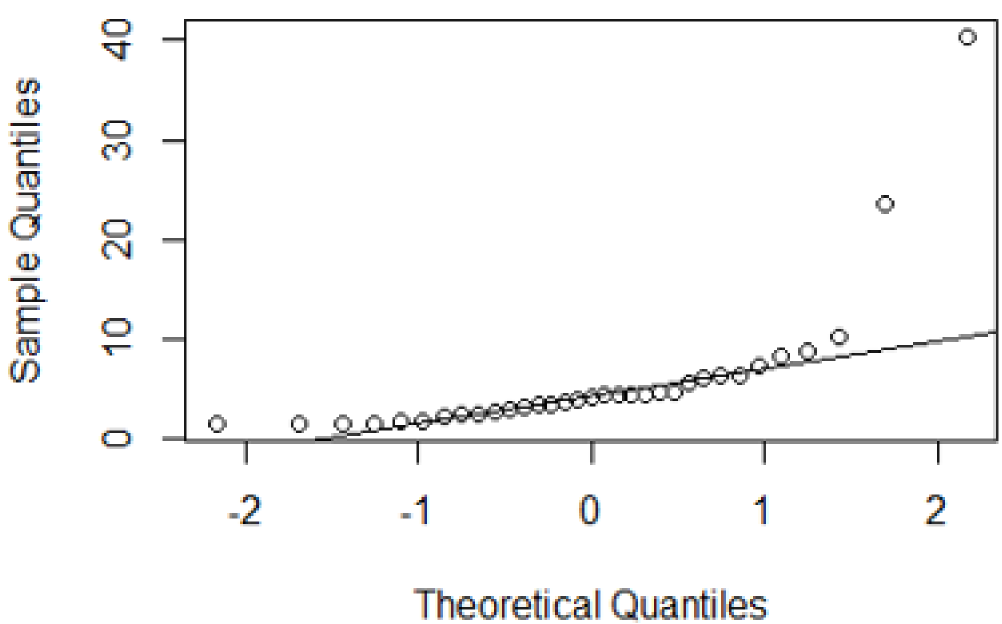

Figure C2 Right wheel track_Before application of HDP foams

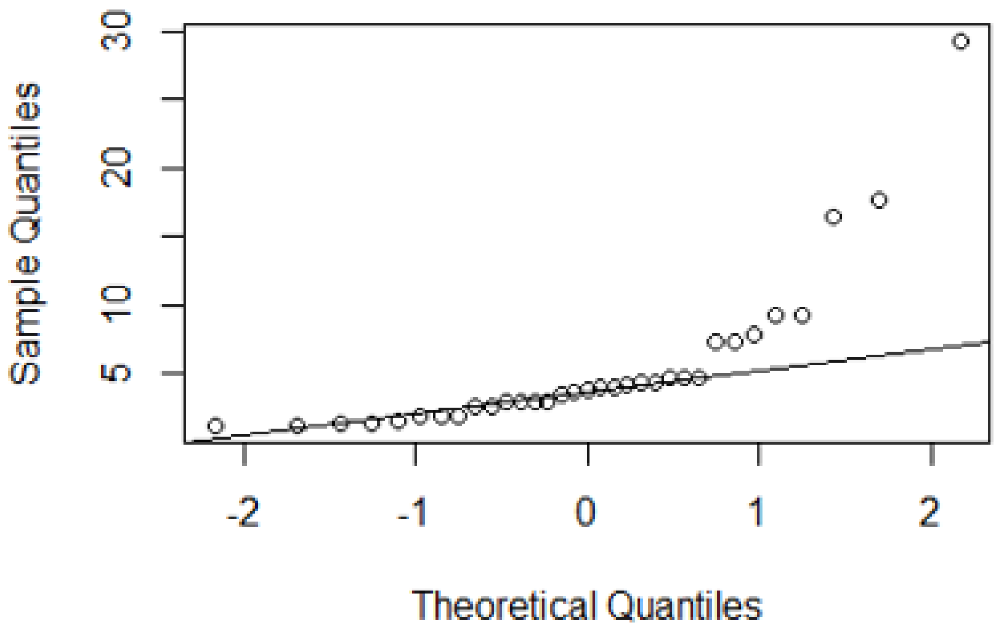

Figure C3 Left wheel track_One week after application of HDP foams

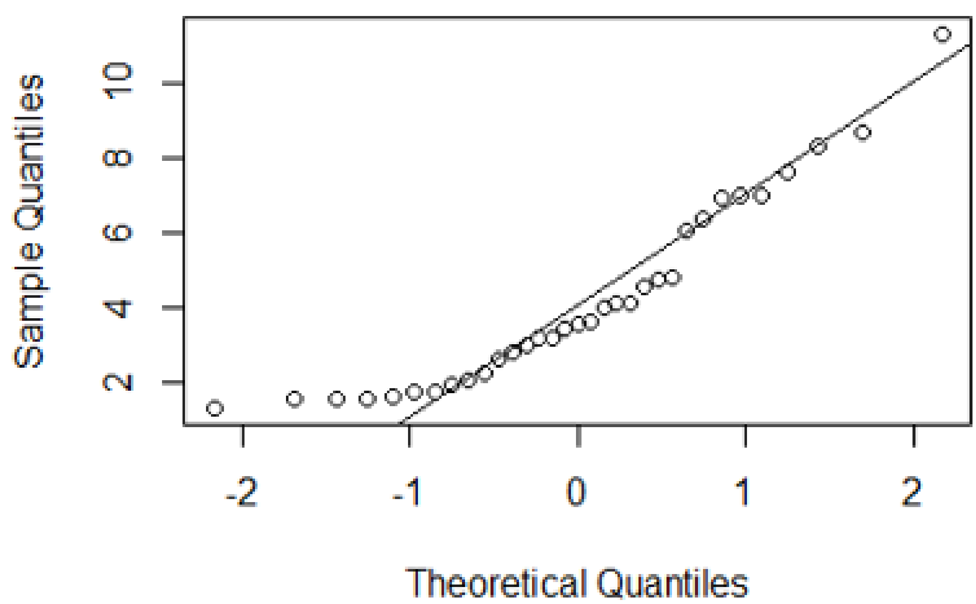

Figure C4 Right wheel track_One week after application of HDP foams

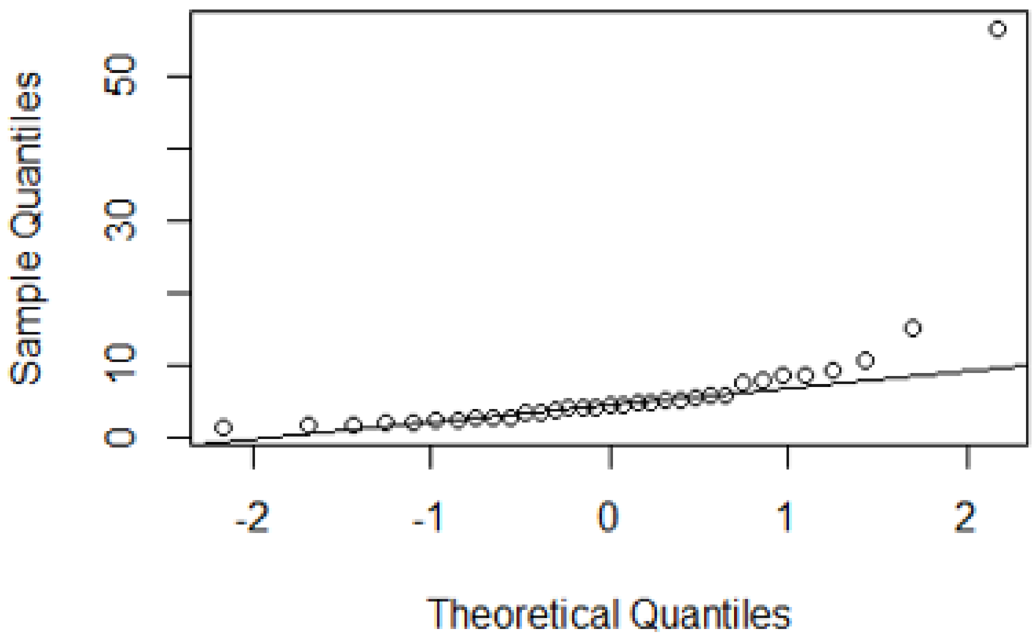

Figure C5 Left wheel track_Thirteen months after application of HDP foams

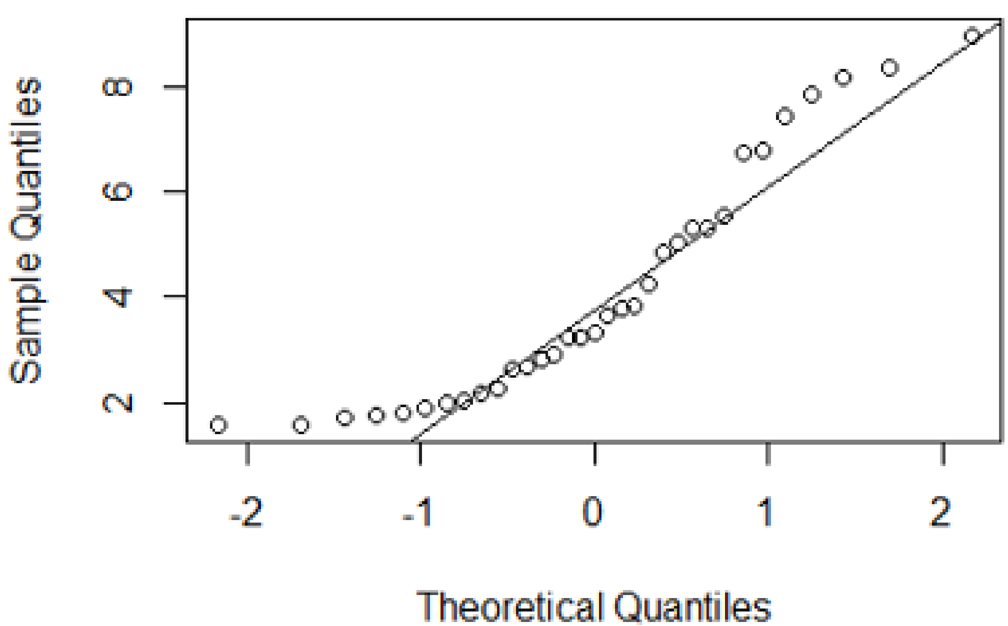

Figure C6 Right wheel track_Thirteen months after application of HDP foams

SECTION I-75 NORTH

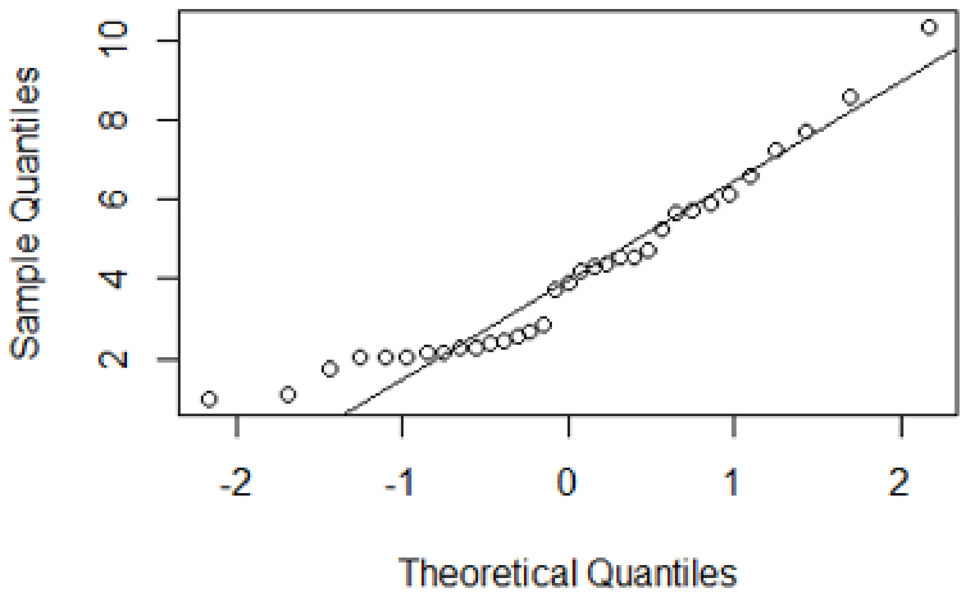

Figure C7 Left wheel track_Before application of HDP foams

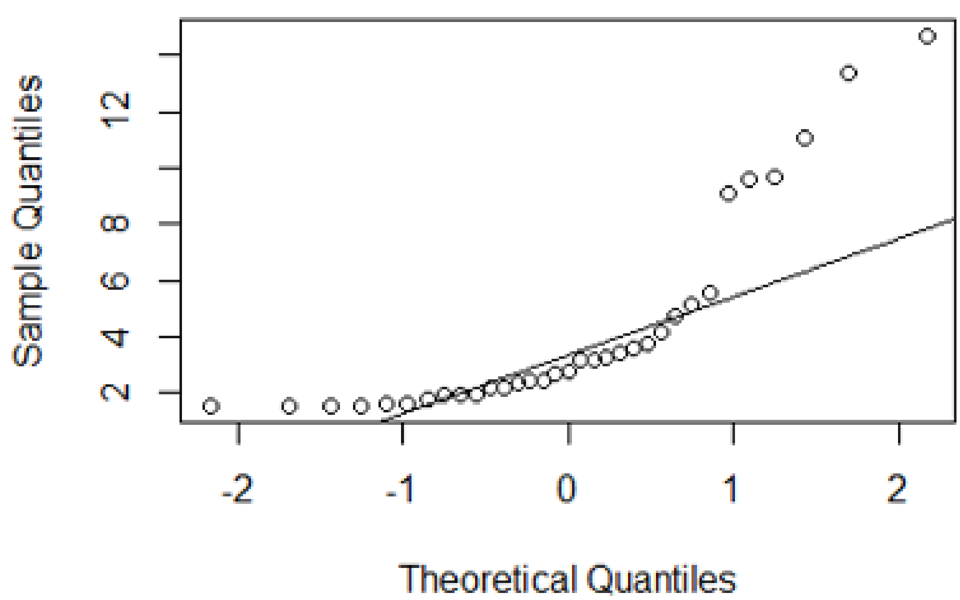

Figure C8 Right wheel track_Before application of HDP foams

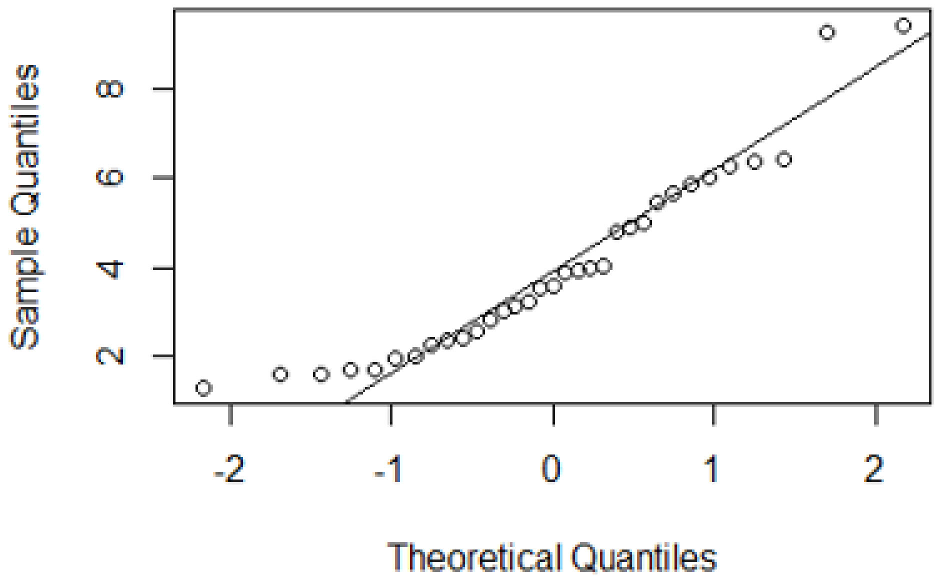

Figure C9 Left wheel track_One week after application of HDP foams

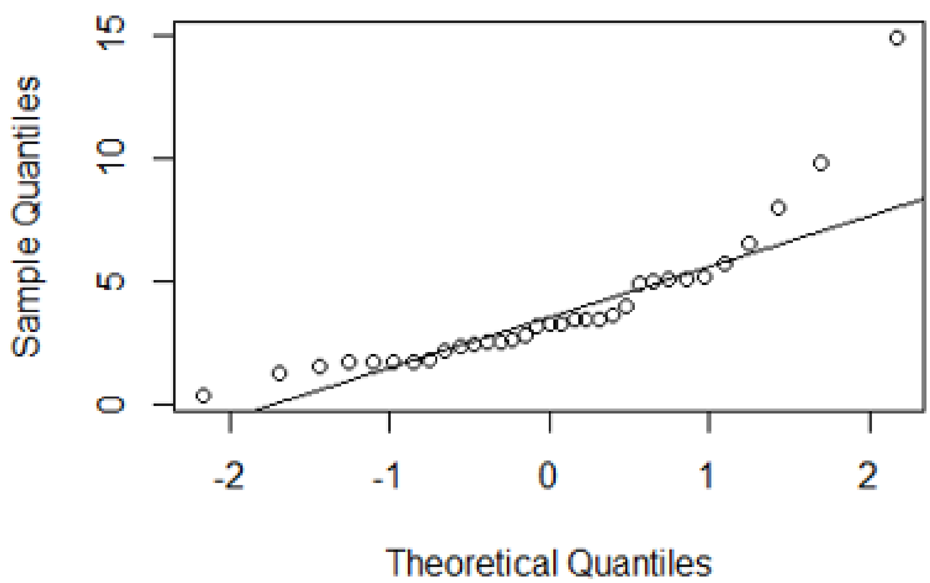

Figure C10 Right wheel track_One week after application of HDP foams

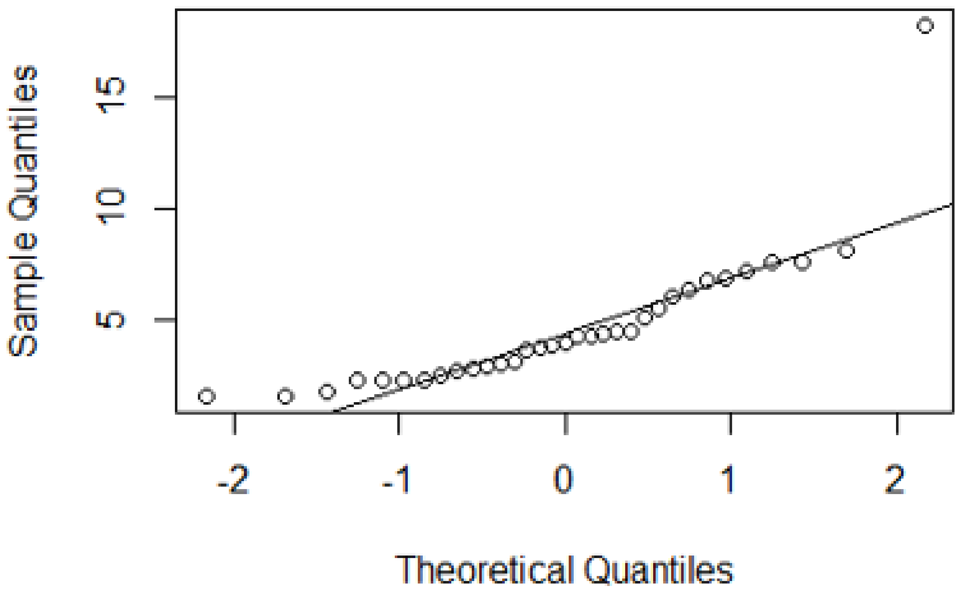

Figure C11 Left wheel track_Thirteen months after application of HDP foams

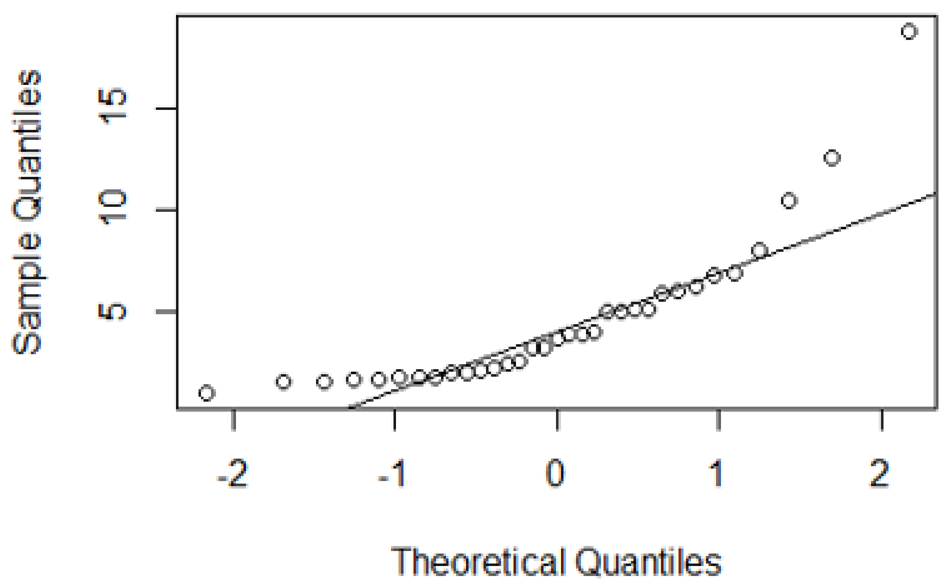

Figure C12 Right wheel track_Thirteen months after application of HDP foams

SECTION I-24 EAST_MOORE

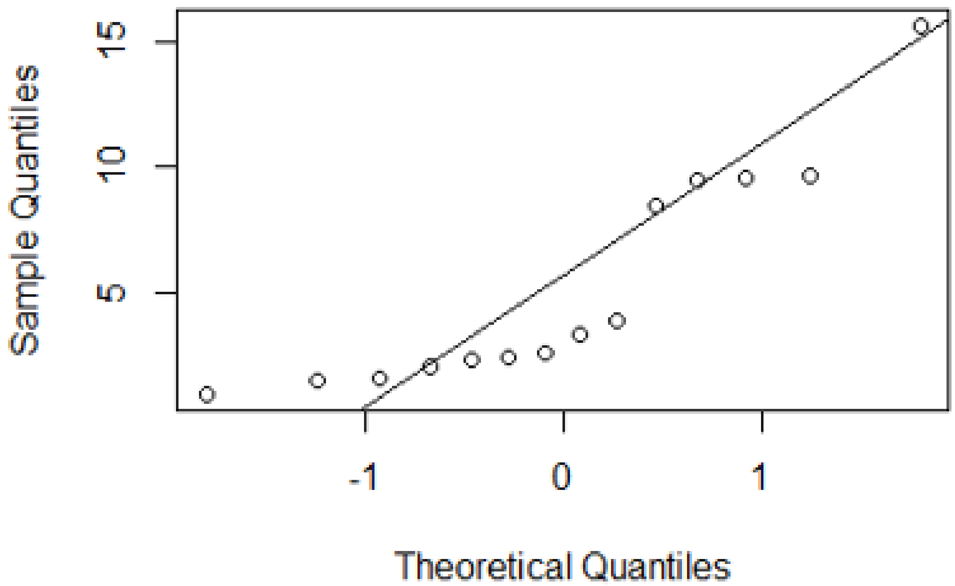

Figure C13 Left wheel track_Before application of HDP foams

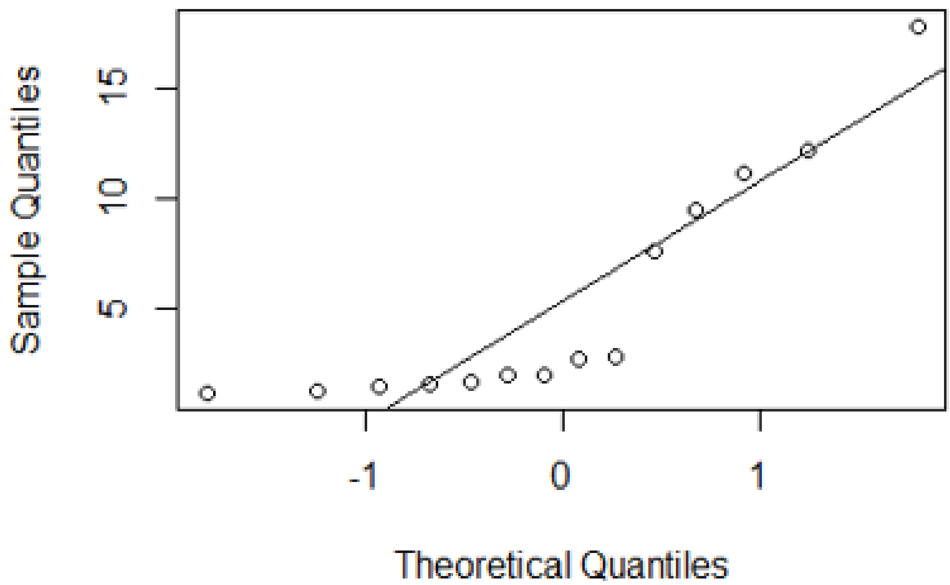

Figure C14 Right wheel track_Before application of HDP foams

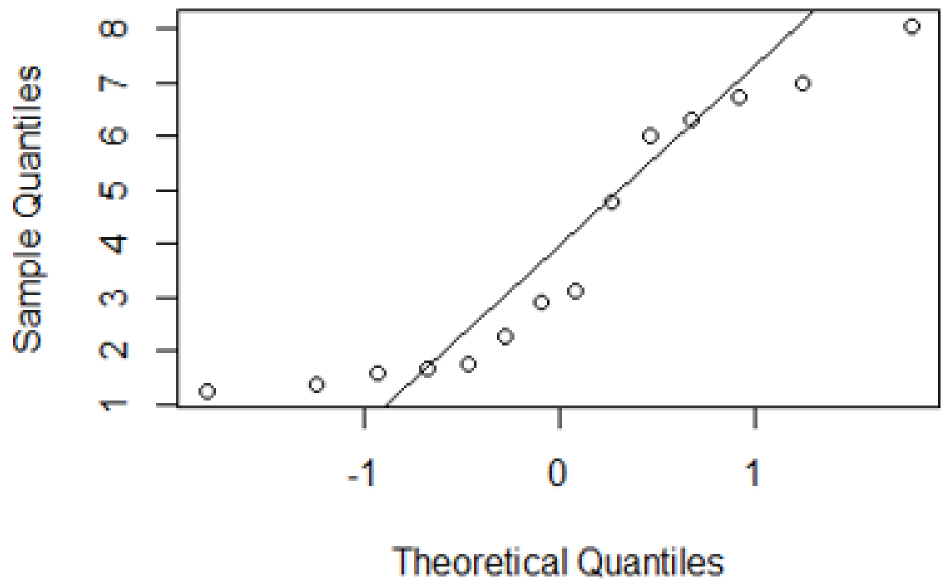

Figure C15 Left wheel track_One month after application of HDP foams

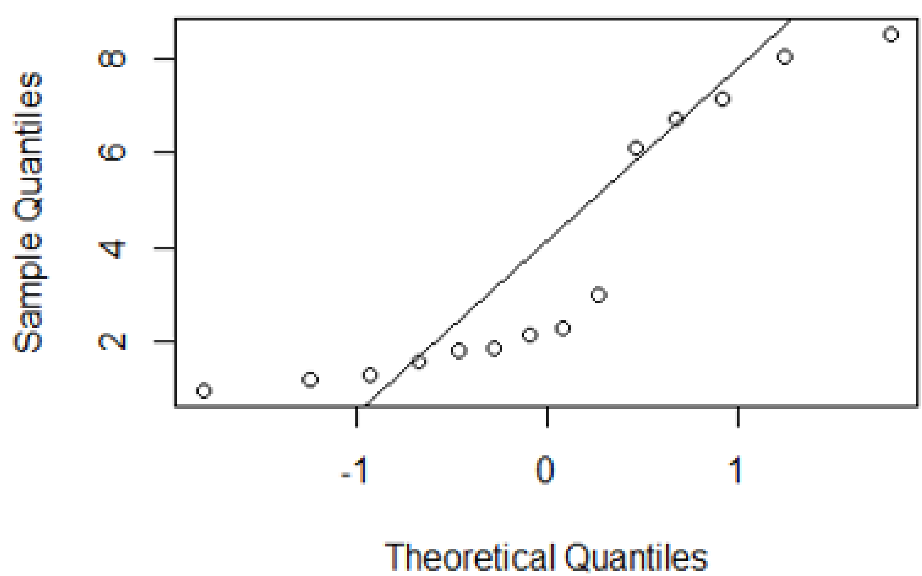

Figure C16 Right wheel track_One month after application of HDP foams

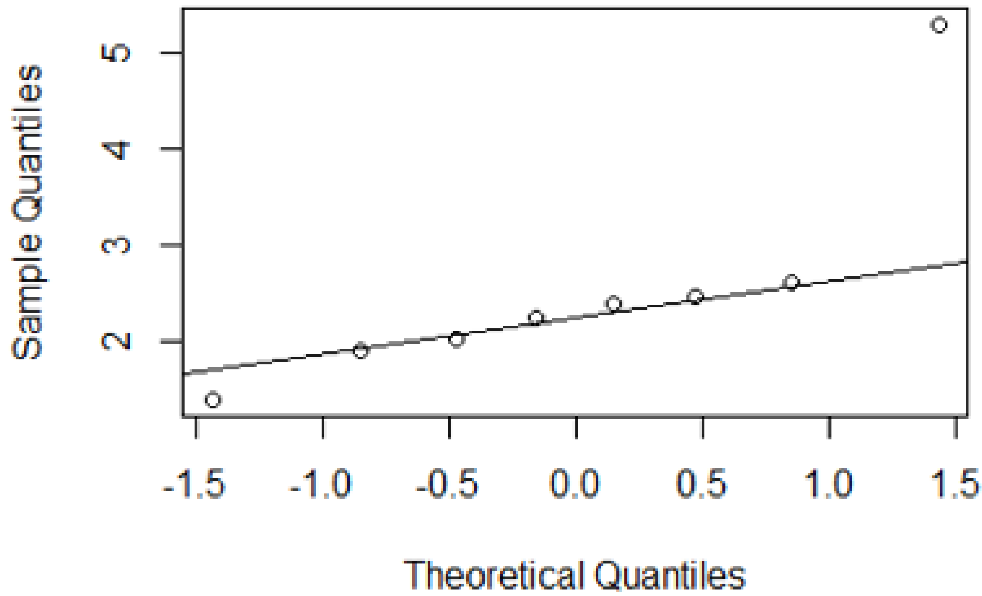

Figure C17 Left wheel track_Thirteen months after application of HDP foams

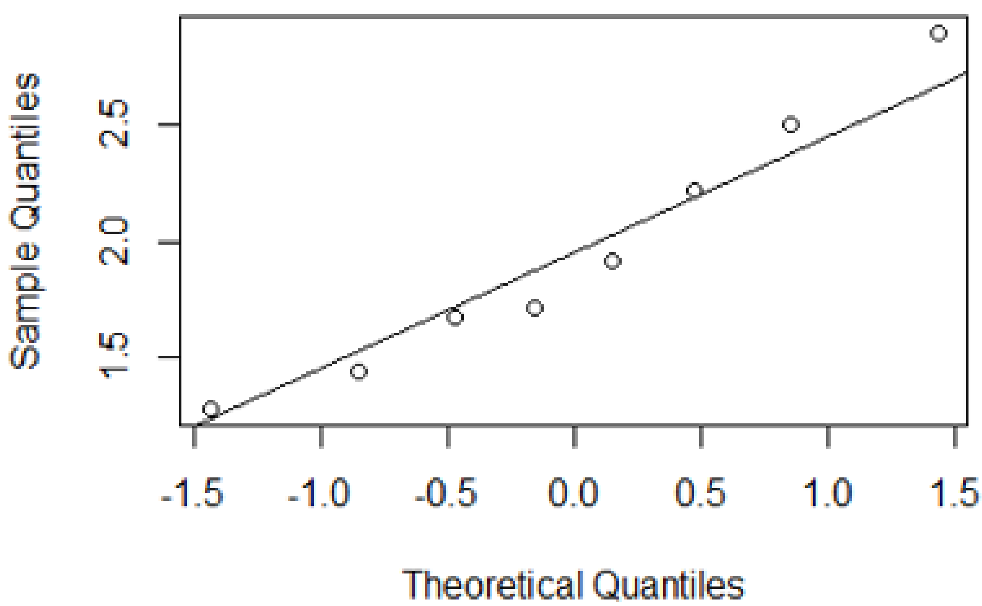

Figure C18 Right wheel track_Thirteen months after application of HDP foams

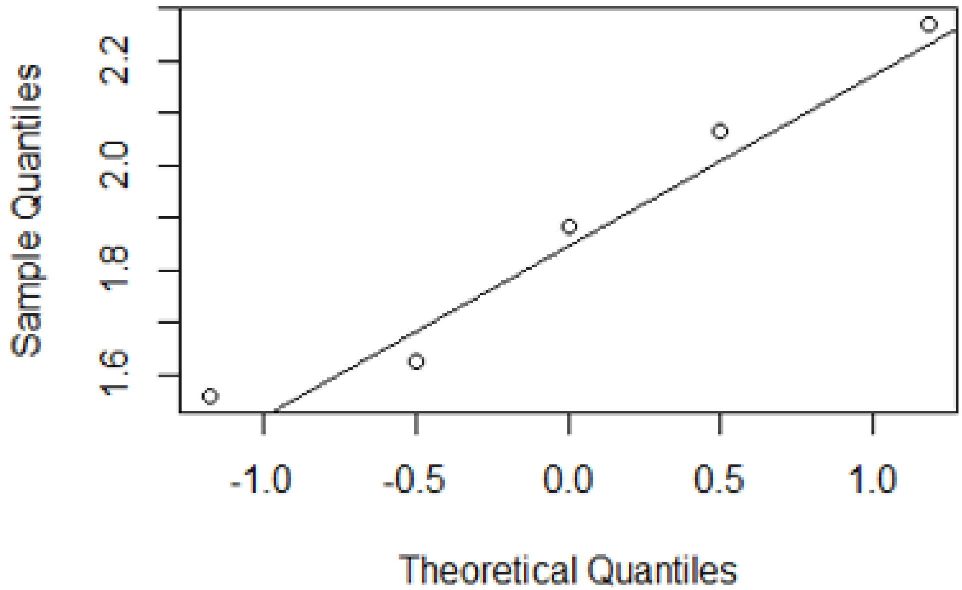

Figure C19 Left wheel track_Twenty-nine months after application of HDP foams

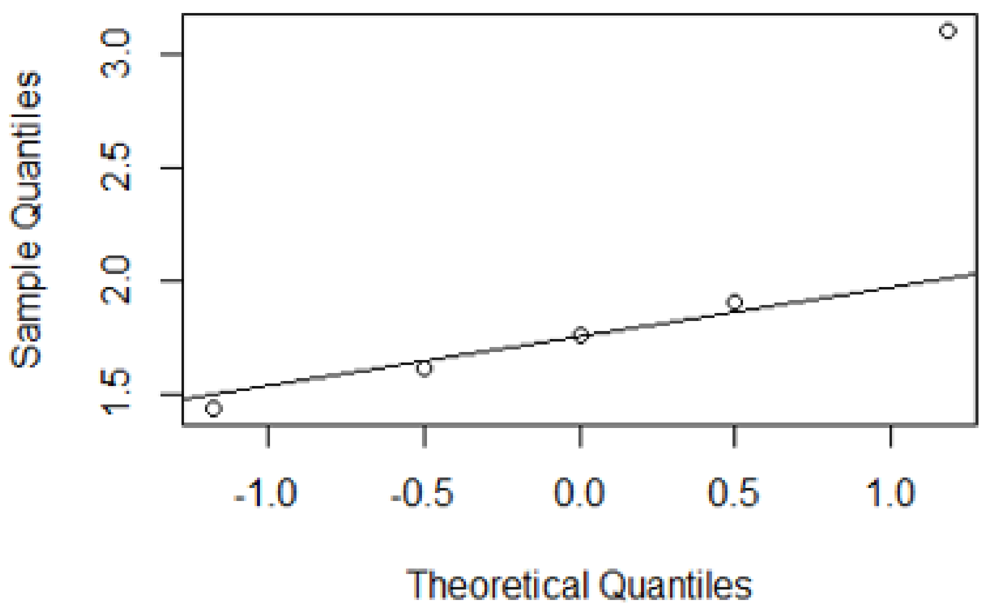

Figure C20 Right wheel track_Twenty-nine months after application of HDP foams

SECTION I-24 EAST_182

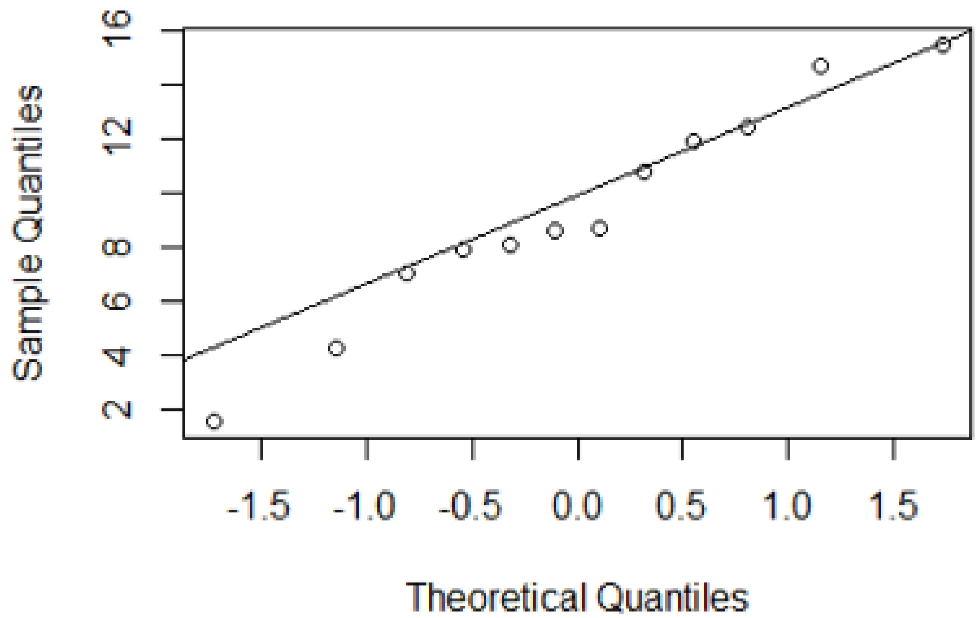

Figure C21 Left wheel track_Before application of HDP foams

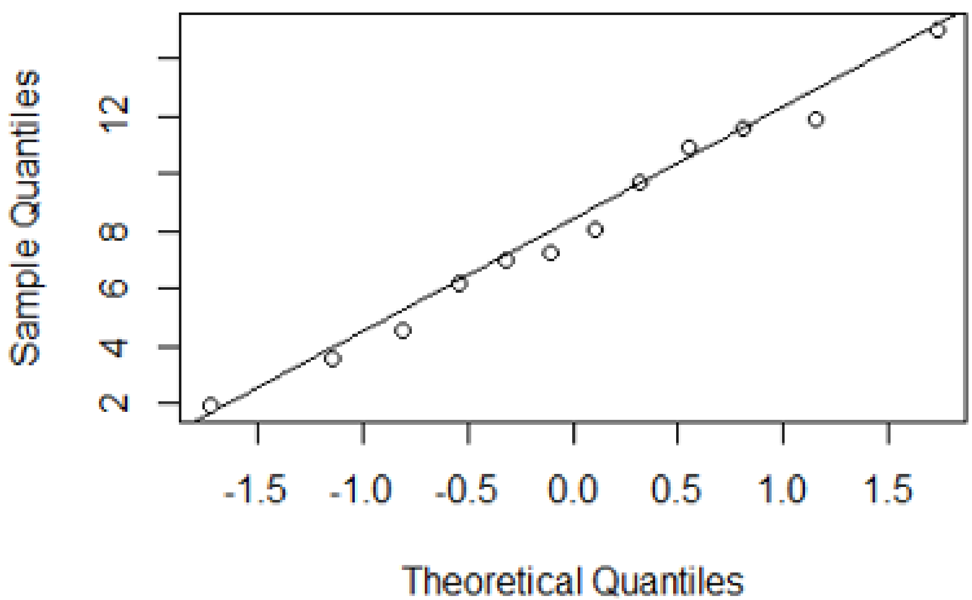

Figure C22 Right wheel track_Before application of HDP foams

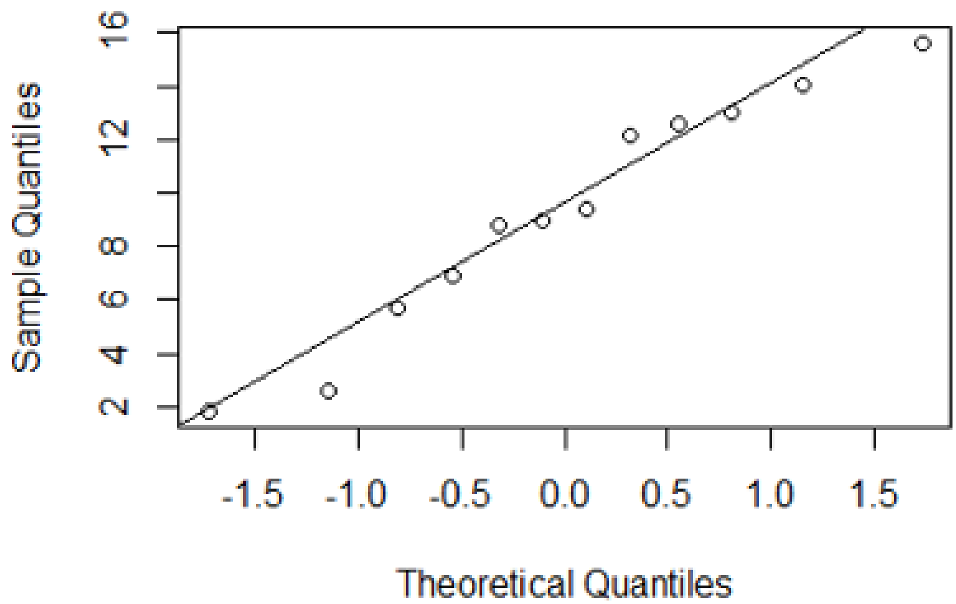

Figure C23 Left wheel track_One week after application of HDP foams

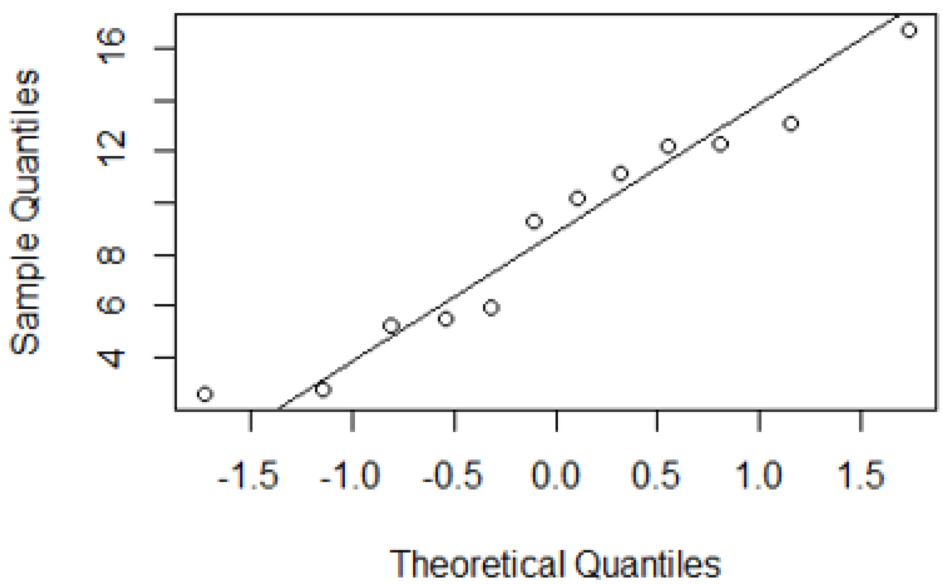

Figure C24 Right wheel track_One week after application of HDP foams

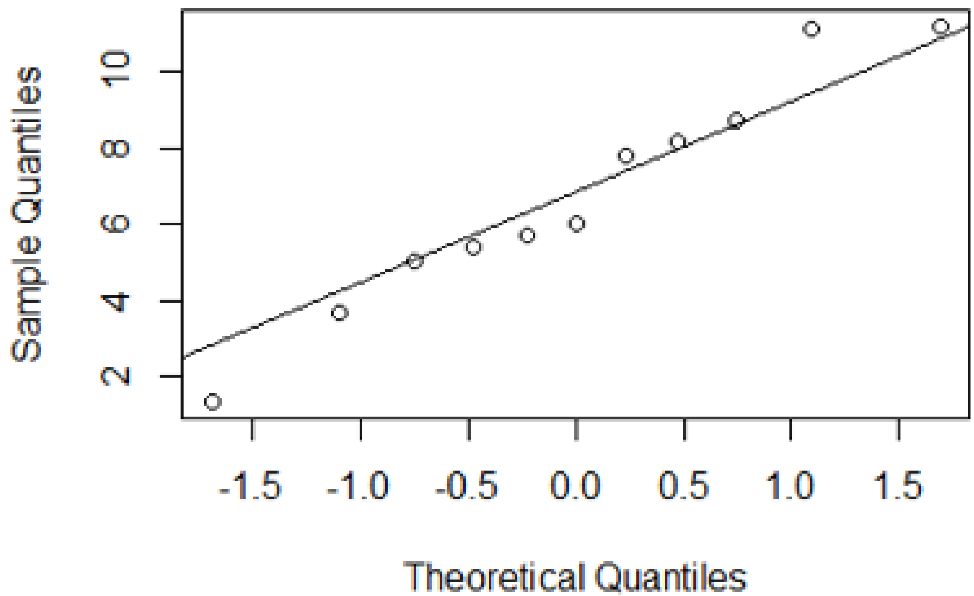

Figure C25 Left wheel track_Eight months after application of HDP foams

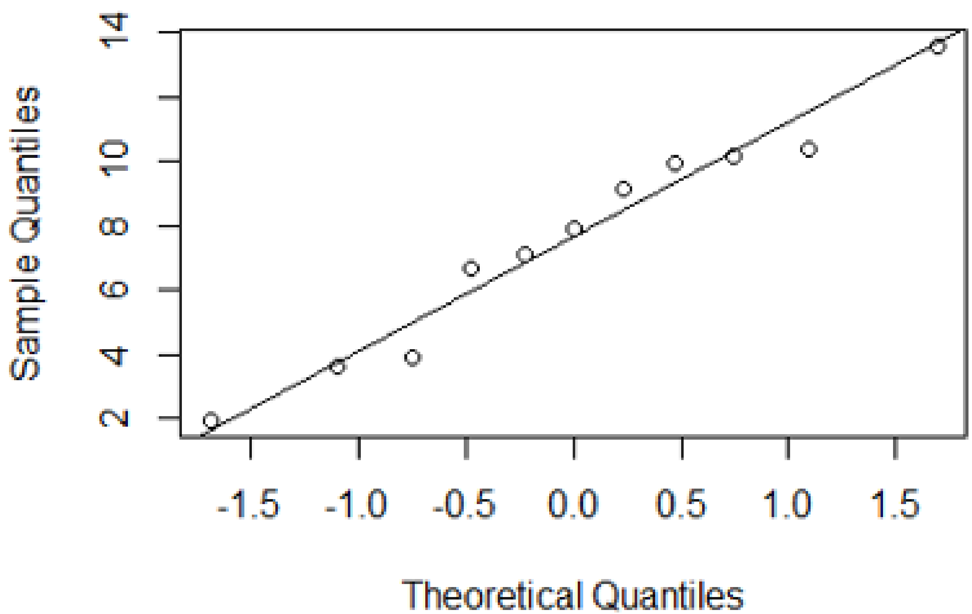

Figure C26 Right wheel track_Eight months after application of HDP foams

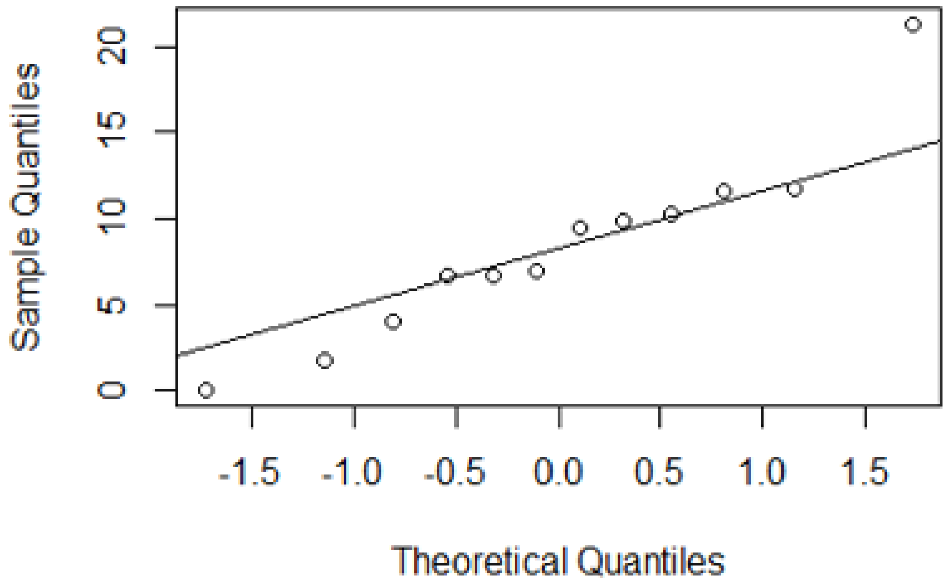

Figure C27 Left wheel track_Nineteen months after application of HDP foams

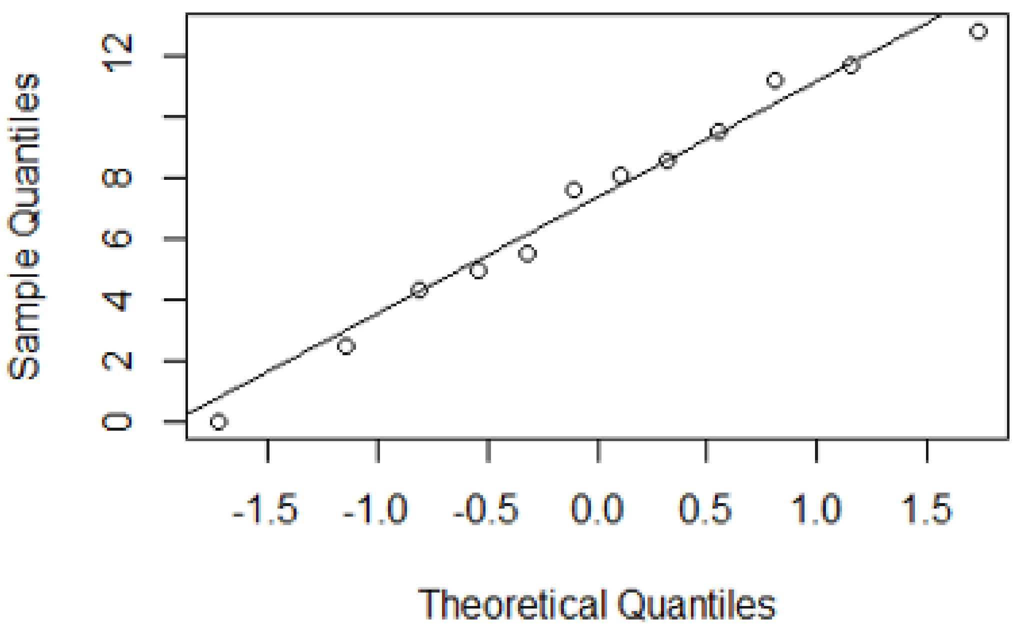

Figure C28 Right wheel track_Nineteen months after application of HDP foams

SECTION I-24 WEST

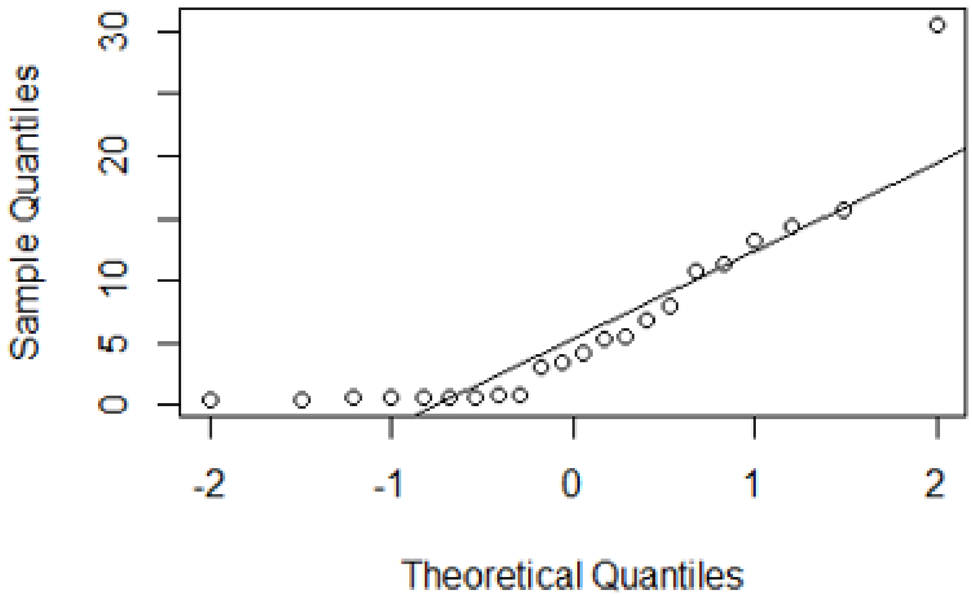

Figure C29 Left wheel track_Before application of HDP foams

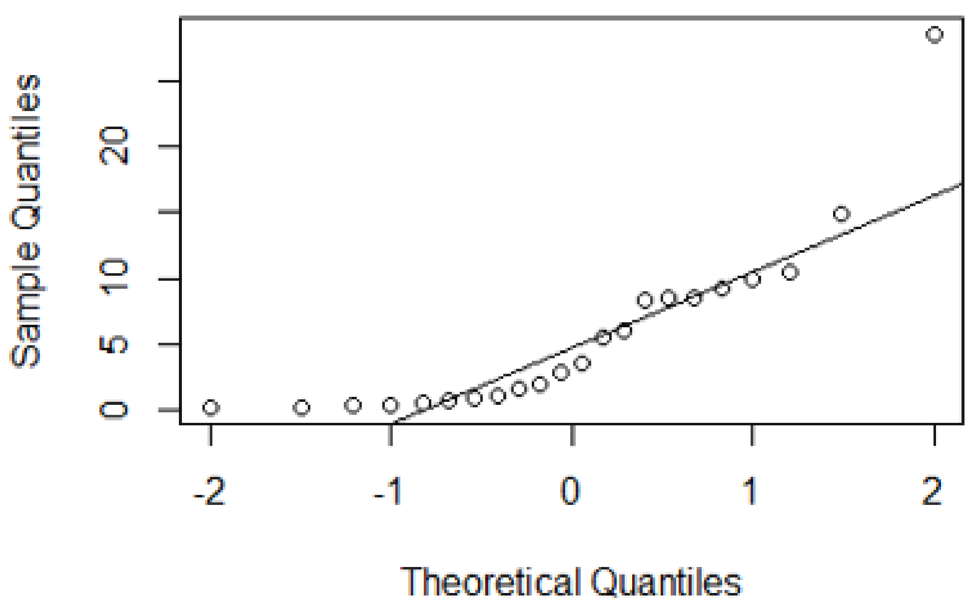

Figure C30 Right wheel track_Before application of HDP foams

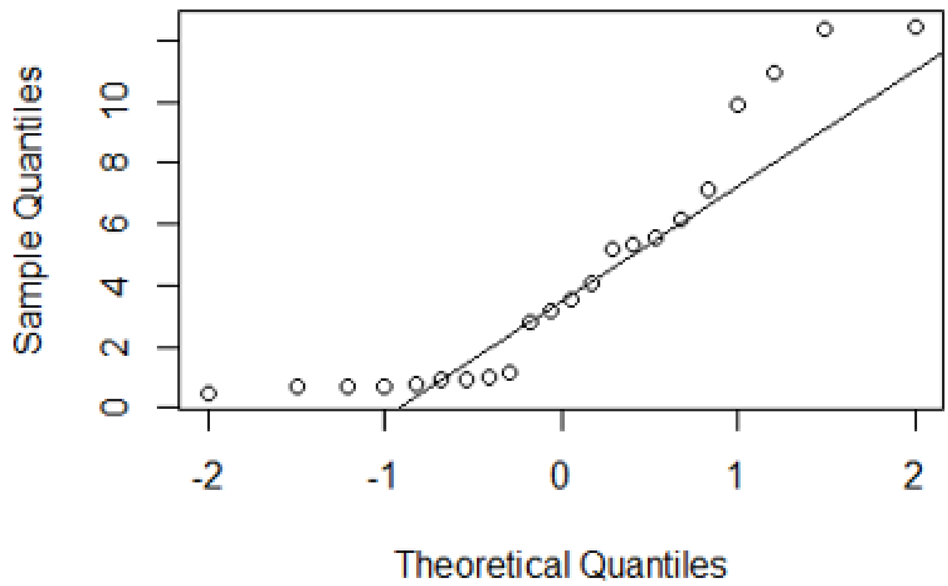

Figure C31 Left wheel track_One week after application of HDP foams

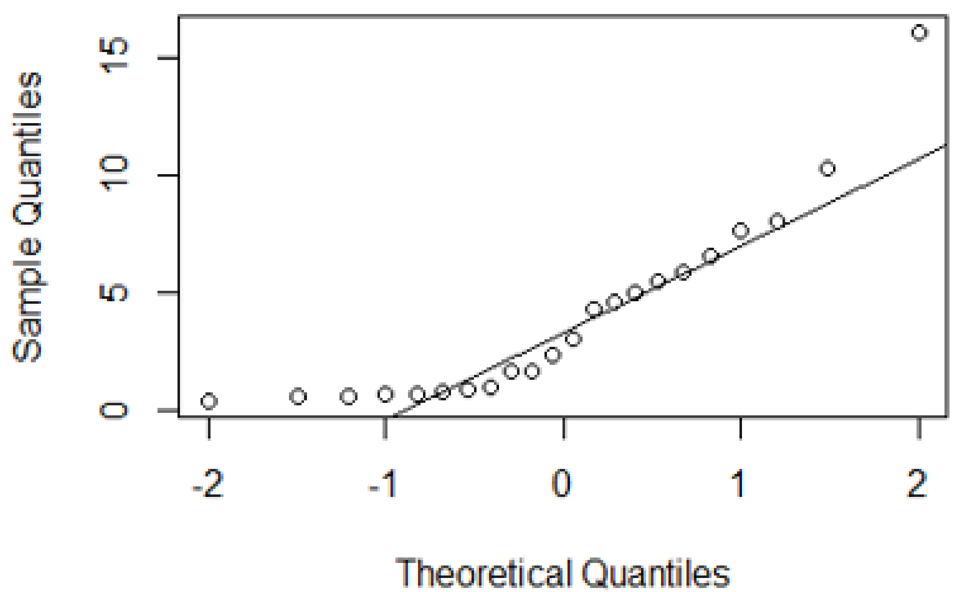

Figure C32 Right wheel track_One week after application of HDP foams

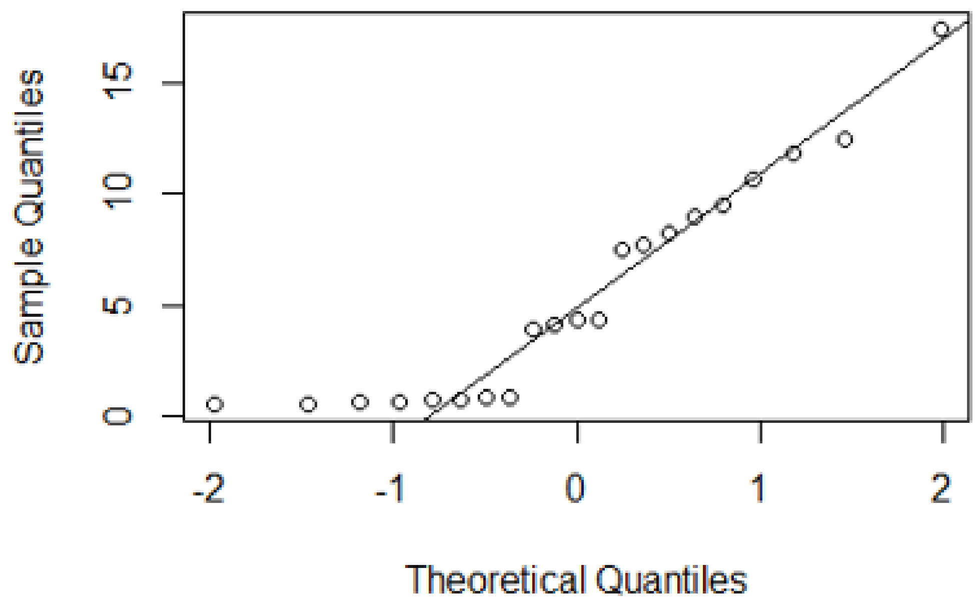

Figure C33 Left wheel track_Eight months after application of HDP foams

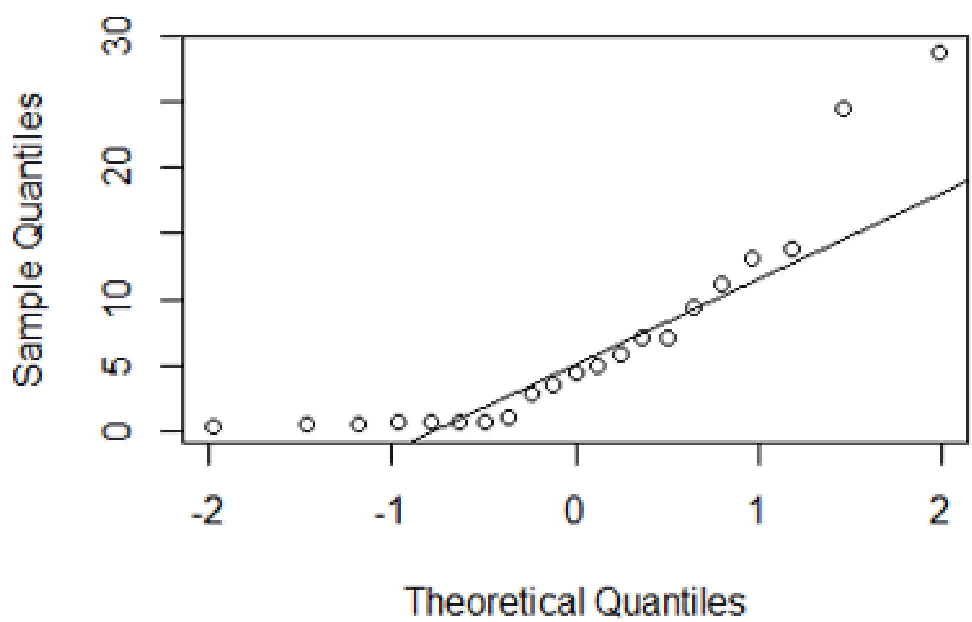

Figure C34 Right wheel track_Eight months after application of HDP foams

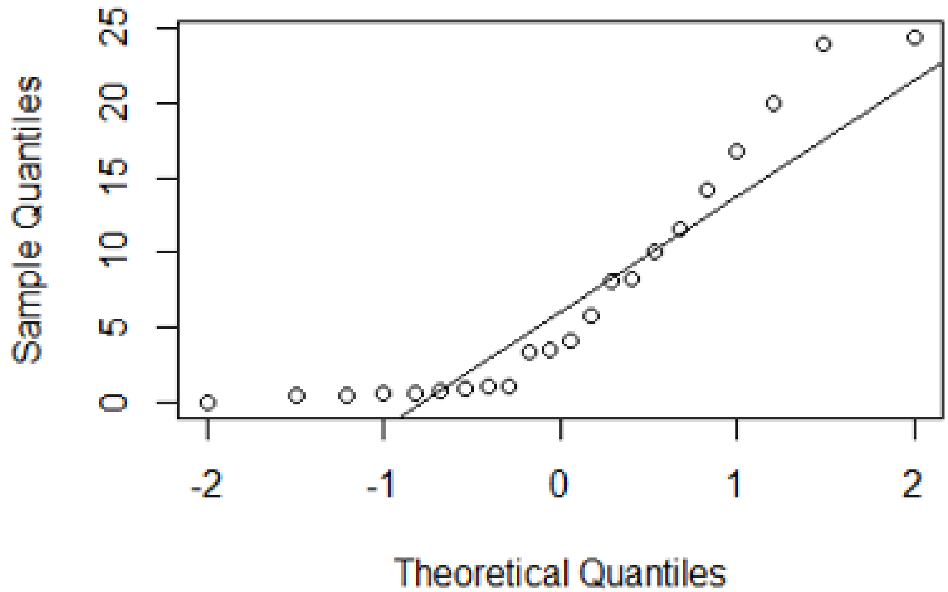

Figure C35 Left wheel track_Nineteen months after application of HDP foams

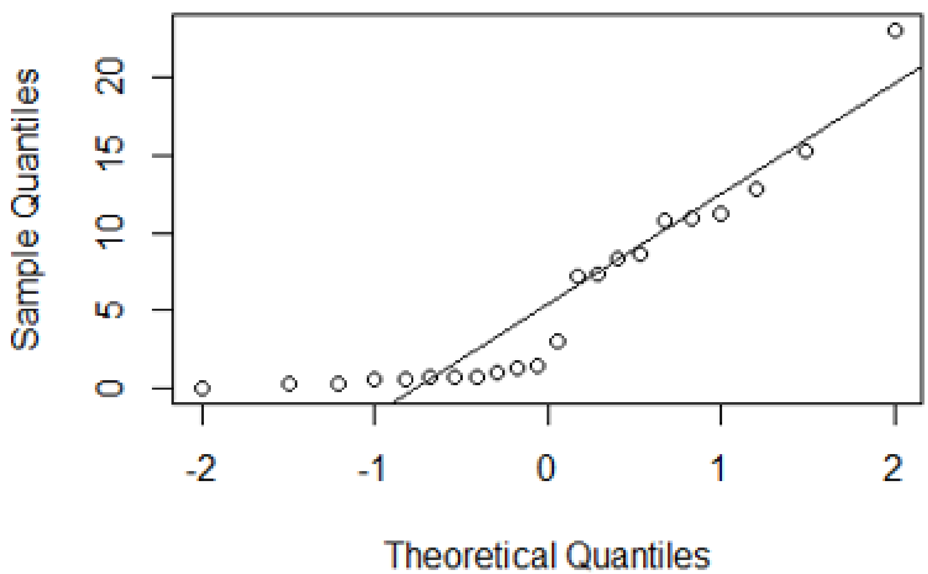

Figure C36 Right wheel track_Nineteen months after application of HDP foams

VITA

Mawazo Fortunatus is a Tanzanian born to Nyamususa Chiyanda and the late Kariba Mashauri. He has three sisters and one brother. He attended Moshi Technical Secondary School for his ordinary secondary school education and continued to Kibaha Secondary School for the advanced level in 2008. He joined the University of Dar es Salaam, Tanzania in 2011 and graduated in 2015 with a B.S. degree in Civil and Transportation Engineering.

After graduating, Mawazo joined the Dar es Salaam Rapid Transit (DART) agency as an Intern Engineer for three months. (DART is the agency responsible for overseeing the operation of the bus rapid transit system in the city of Dar es Salaam.) In December 2015, Mawazo returned to the University of Dar es Salaam, where he worked as a Tutorial Assistant to the Department of Transportation and Geotechnical Engineering.

He joined the University of Tennessee at Chattanooga (UTC) in August 2016 after acquiring a research assistantship position. Mawazo will graduate from UTC in August 2018 with a M.S. degree in Civil Engineering. In Fall 2018, Mawazo will join Auburn University, where he will pursue a Ph.D. degree in Civil Engineering.