Non-Destructive Testing for Optimizing the Rehabilitation of Rigid Pavements

by

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ABSTRACT

Factors contributing to the successful rehabilitation of a roadway system include the application of the appropriate tools and analyses at the appropriate time and location. This would ensure that only road sections that require attention are addressed with the right remedy to achieve optimized rehabilitation performance and cost. Falling weight deflectometer (FWD) is a tool that is widely used to evaluate joint performance (load transfer capacity) of Portland concrete cement (PCC) pavements. However, several challenges are typically encountered in that effort.

FWD testing was completed on three urban arterial sections and six residential streets prior to and after rehabilitation to select the appropriate layout for FWD testing, establish joint performance threshold values, optimize FWD testing time and cost, and evaluate the effect of asphalt overlays on joint performance parameters. Moreover, the performance of full-depth repairs (FDR) and the incorporation of joint performance at the design stage were studied. Residential streets joint and basin FWD testing was completed to evaluate the structural capacity of residential streets and compare their performance to arterial regional roads.

Comparisons between two widely used FWD geophones layouts allowed for the selection of a layout that represents the more critical loading condition on joints. Peak deflections and differential deflections were correlated with load transfer efficiencies to select threshold values for LTEs, peak deflections and differential deflections to trigger rehabilitation at the appropriate time. Statistical testing was used to optimize FWD testing to two load levels instead of four.

The study found that asphalt overlays reduce recorded deflections and overestimate computed joints performance parameters. The effect of asphalt overlays was evaluated to establish correction factors, $F_{asphalt}$, for each load level to estimate deflections on the concrete surface using deflections

obtained from testing on the overlay surface of Pembina Highway. The correction was found to improve the reliability of joint condition evaluation for that site. The study also found that FDRs generally restore the load transfer capacity of joints with good support and mechanical load transfer achieved. Moreover, it was demonstrated that joint performance information can be used at the design stage in the decision-making process to select the location and type of joint repairs. Lastly, residential streets were found to have less load transfer capacity than arterial regional roads and weaker pavement structure.

The findings from this study indicated that NDT can be used to evaluate pavement condition, determine layer stiffness for use in rehabilitation design and to improve planning of rehabilitation through timely determination of rehabilitation needs and improvement of the reliability of load transfer efficiency estimate and void detection of PCC joints.

ACKNOWLEDGEMENTS

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Last and not least, my sincere gratitude goes to my life pillars – my family. My father, Mustafa Al-Abbas, and mother, Basima Al-Almudaras, deserve to be named on this work. My gratitude goes to my parents and siblings for their everlasting support and love.

DEDICATION

Dedicated to

Mustafa Al-Abbas

Basima Al-Almudaras

Abdullah Al-Abbasi

Shahad Al-Abbas

for their love and support.

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List of Abbreviations

AADT	Annual Average Daily Traffic
DBR	Dowel Bar Retrofits
FDRs	Full-Depth Repairs
FEA	Finite Element Analysis
FHWA	The Federal Highway Administration
FWD	Falling Weight Deflectometer
GPR	Ground Penetrating Radar
GPS	Global Positioning System
JCP	Jointed Concrete Pavements
LTE	Load Transfer Efficiency
LTPP	Long-Term Pavement Performance
MET	Method Of Equivalent Thickness
MTO	Ministry Of Transportation Ontario
NCHRP	National Cooperative Highway Research Program
NDT	Non-Destructive Testing
PCC	Portland Concrete Cement
RDD	Rolling Dynamic Deflectometer
RTK	Real Time Kinematic

Chapter 1

INTRODUCTION

1.1. Background

A successful roadway rehabilitation project consists of triggering the rehabilitation at the right time and applying the selected rehabilitation strategies at the right locations. Accurate characterization of pavement performance and the selection of appropriate action trigger values are, therefore, important in achieving a successful rehabilitation project. However, different agencies have different trigger values of non-destructive testing (NDT) results for maintenance action. Moreover, multiple testing equipment and layouts are available which renders selecting the appropriate one to be difficult. Performing NDT in urban environments can present undesirable traffic interruption and so performing NDT at appropriate time and optimizing testing time are essential in the success of any testing program.

In the City of Winnipeg, and similarly in other cities, approximately 60% of roadways are Portland Concrete Cement (PCC) pavements that have been overlaid with an asphalt cement layer. The effect of the asphalt overlay on characterizing pavement performance should be investigated and accounted for at the planning stage of rehabilitation projects.

Joints deterioration and reflection cracking in composite pavements are major distresses that are frequently encountered in rehabilitation projects (National Cooperative Highway Research Program, 2004; R. Smith, Palmeri, Darter, & Lytton., 1984; H.L. Von Quintus, Finn, Hudson, & Roberts, 1979). Therefore, this study considers different test setups in evaluating joints performance, selects appropriate trigger values for joints rehabilitation and determines the effect of asphalt overlays on load transfer testing and void detection analysis. Moreover, this study aims to optimize NDT testing time and cost, includes a case study examining the performance of full-depth repairs in restoring the load transfer capacity of joints and evaluating the benefits of incorporating NDT at the design stage. Comparison is made between the structural capacity and performance of different road functional classes and design inputs are provided for future application of NDT findings.

The predominant state-of-practice in road rehabilitation projects is to conduct visual inspection on concrete and composite structures to decide the type and location of repairs. Non-destructive testing can provide insights into pavement performance to assist such decisions to preserve, maintain and rehabilitate pavements in a cost-effective and strategic manner. This would ensure that only road sections that require attention are addressed with the appropriate rehabilitation strategy to achieve optimized rehabilitation performance and cost.

One of the commonly used NDT is deflection testing and the falling weight deflectometer (FWD) has become the main testing device of choice for many agencies. Hveem (1955) demonstrated how pavement deflections can provide information about the pavements structural capacity to provide the service level it was intended to. FWD can evaluate the performance of joints in jointed concrete pavements (JCP) and identify possible voids under joints. Ideally, FWD testing should be conducted prior to the project’s commencement to select rehabilitation strategies, estimate quantities for bidding, quantity estimation and budgeting rehabilitation costs.

1.2. Objectives

The objectives of this thesis are to

1. Select a geophones testing layout after investigating the results of two geophones layouts in terms of load transfer efficiencies (LTEs), differential deflections and peak deflections;
2. Establish threshold values for joint performance in terms of LTEs, peak deflections and differential deflections;
3. Optimize FWD testing procedure through reduction of applied load levels;
4. Evaluate the effect of asphalt overlays on deflections, load transfer efficiency measurements and void detection analysis;
5. Recommend correction factors to account for asphalt overlays' influence on load transfer efficiency measurements and void detection;
6. Study the effectiveness of full-depth repairs in restoring load transfer efficiency and the benefits of incorporating joint performance at the design stage; and,
7. Evaluate the joint performance and structural capacity of pavement structures of different functional classes - residential streets and regional roads.

1.3. Methodology

The approach to achieve the objectives of this study is to conduct Falling Weight Deflectometer (FWD) testing using different geophone layouts. The geophone layout representing the more critical load condition is then selected for subsequent testing. FWD testing is then performed along different sections prior to and after asphalt overlay milling and concrete joints repairs. Correlations are then established between different joints performance parameters to establish threshold values for joints performance parameters. Statistical testing is then used on collected deflection values to optimize FWD testing time and cost.

FWD testing prior to and after asphalt overlay milling is used to evaluate the effect of overlays on joint performance parameters and void detection. Correlations are then established to recommend correction factors that account for the presence of asphalt overlays.

The performance achieved by full-depth repairs and the benefits of incorporating NDT at the design stage is also studied by examining FWD testing results. Lastly, the joint performance and structural capacity of residential streets are evaluated in comparison to regional arterial roads to determine the performance of different functional classes of roadways.

1.4. Organization of Thesis

This thesis covers the research done over two years and has been organized as outlined below:

Chapter 1: Introduction

This chapter overviews the background and motivation to the research, the objectives and summarized methodology of the study conducted.

Chapter 2: Literature Review

This chapter summarizes the background information and the related research and agencies work in the application of FWD in joints evaluation and application at the project-level, and how this work complements existing research.

Chapter 3: Non-Destructive Testing Program

This chapter outlines the sections where NDT testing was performed, their planned rehabilitation and the testing program and analysis methodology.

Chapter 4: Falling Weight Deflectometer Testing Results

This chapter reports findings of FWD testing on selecting geophones layout, establishing threshold values, statistical selection of FWD load levels, evaluating the effect of asphalt overlays on measurements and recommending correction factors to account for the presence asphalt overlays.

Chapter 5: Full-Depth Repairs Performance and Incorporating NDT at the Design Stage

The chapter presents findings of the performance achieved by full-depth repairs and the benefits of incorporating FWD testing at the design stage to select the appropriate treatment.

Chapter 6: Residential Streets FWD Testing Results

Chapter 6 outlines the results of FWD testing on residential streets and evaluates joint performance and structural capacity of residential streets in comparison to arterial regional roads.

Chapter 7: Summary, Conclusions, and Recommendations

Chapter 7 summarizes the work done as part of this research, the conclusions and the recommendations determined from FWD testing. Lastly, the chapter discusses possible future research.

Chapter 2

LITERATURE REVIEW

2.1. Introduction

Non-Destructive Testing (NDT) offers fast, cost-effective and accurate methods to characterise pavement materials instead of conventional laboratory tests that are usually expensive and cannot be performed in large numbers. NDT allows for the in-situ testing of pavement layers in field conditions while reducing the number of cores required for laboratory testing. Therefore, NDT offers an appealing alternative to laboratory testing due to its lower cost, production rate and accuracy. For instance, the use of Falling Weight Deflectometer (FWD) on Virginia's network-level proved to be of high value in asset management (Diefenderfer, 2008). Another study emphasized how FWD can be applied in design, maintenance, rehabilitation and management at the network level in Indiana and recommended the use of NDT at the project level (Noureldin, Zhu, Li, & Harris, 2003). A study conducted in New Jersey found that FWD can enhance rehabilitation decisions compared to Pavement Management Systems and lead to better budget allocation (Zaghloul & Kerr, 1999).

As cities face aging infrastructure, there is an increase in the rehabilitation projects of roadways to maintain the structural and functional service life of pavements. For efficient cost allocation in such projects, information about the structural condition and areas requiring repairs would be valuable. Pavement stiffness and layer thicknesses are typically determined using pavement cores and laboratory testing. However, the time and cost they incur allow for only a small number of pavement locations to be evaluated. Moreover, in-field joint performance can help engineers decide on the location and the appropriate repair method. NDT can serve as a decision-making tool which provides data to estimate material properties and performance at a faster rate and lower cost than laboratory testing. Integrating NDT in pavement evaluation can be a valuable decision-making tool to better manage infrastructure. Different types of NDT methods and equipment are available. However, the FWD has been widely used by agencies and researchers to characterize pavement structures and joints performance.

2.2. Joints Function and Deterioration

Saw-cut joints are introduced in PCC pavements shortly after construction to control transverse and longitudinal cracking which occur due to concrete shrinkage and contraction. A joint system is considered functional when it allows for slab movements and transfers traffic loads between slabs, as well as improves the constructability of the pavement (Wang et al., 2018). Long term performance and service life of PCC pavements are significantly affected by the condition and performance of the joints. Therefore, early joint deterioration is a major factor contributing to shorter pavement service life, especially in cold regions where de-icing salts are used, and where poor drainage prevails under the slab (Peter, Zhang, & Wang, 2016). Typical deterioration of PCC joints are illustrated in Figure 2-1.

Figure 2-1: Typical Joint Deterioration (Wang et al., 2018).

Low LTE was identified to cause pumping and settlement in a continuously reinforced concrete pavement section (Chen, Hong, Yao, & Bilyeu, 2011). Moreover, the size of tie bars, anchorage quality and the construction method (such as hole size, type of drill and repair area) of full-depth repairs and the base and subgrade support condition were found to affect joints performance (Chen, Hong, et al., 2011). Figure 2-2 shows tie bars deterioration identified from cores obtaind as a part of the published study.

Figure 2-2: Tie Bars Deterioration in PCC Joints (Chen, Hong, et al., 2011).

Joint spalling is the cracking and chipping of concrete around slab edges which does not propagate through the full slab but propagates to intersect with the joint at an angle. Joint spalling is usually a result of various environmental and material properties as well as dowel insertion (construction) method and poorly functioning load transfer devices (State of California Department of Transportation, 2008). Figure 2-3 shows examples of joint spalling.

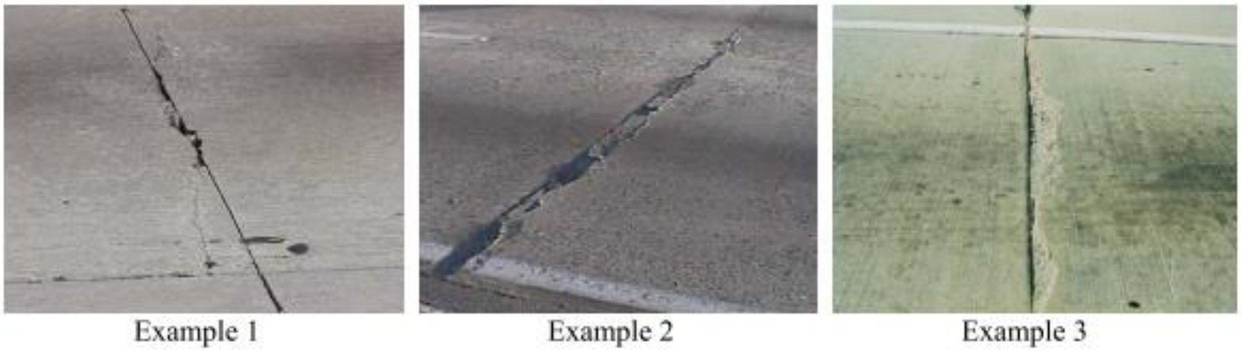


Figure 2-3: Examples of Joint Spalling (State of California Department of Transportation, 2004).

Faulting, as shown in Figure 2-4, is the permeant elevation variation between two adjacent concrete slabs and is caused in the absence of load transfer when slabs are free to move independently in the vertical direction. The variable slabs movement, in the presence of water, resulting in the movement of fines from one side of the joint to the opposite (State of California Department of Transportation, 2008).

Figure 2-4: Joint Faulting (Federal Highway Administration, 2003).

Deterioration of rigid pavement joints is ~~a~~ principal distress that can affect the serviceability of jointed concrete pavements (JCPs). It can also lead to further deterioration due to the ingress of water into the subsurface layers of the pavement structure. JCPs undergo loss of integrity at the joints due to the loss of shear force transfer across the joint via dowels. Due to repeated heavy traffic or construction method/quality, installed dowels may become loose or suffer from concrete delamination underneath them. This, in turn, disrupts the load transfer capacity of joints. Loss of load transfer across joints results in increased deflections on either side of the joints which may lead to cracking (Snyder, 2011). Moreover, after repeated load cycles and the loss of load transfer, base and subgrade materials are pumped and eroded which leads to the development of voids under slab corners as shown in Figure 2-5 (National Highway Institute, 1994; Stahl, 2006).

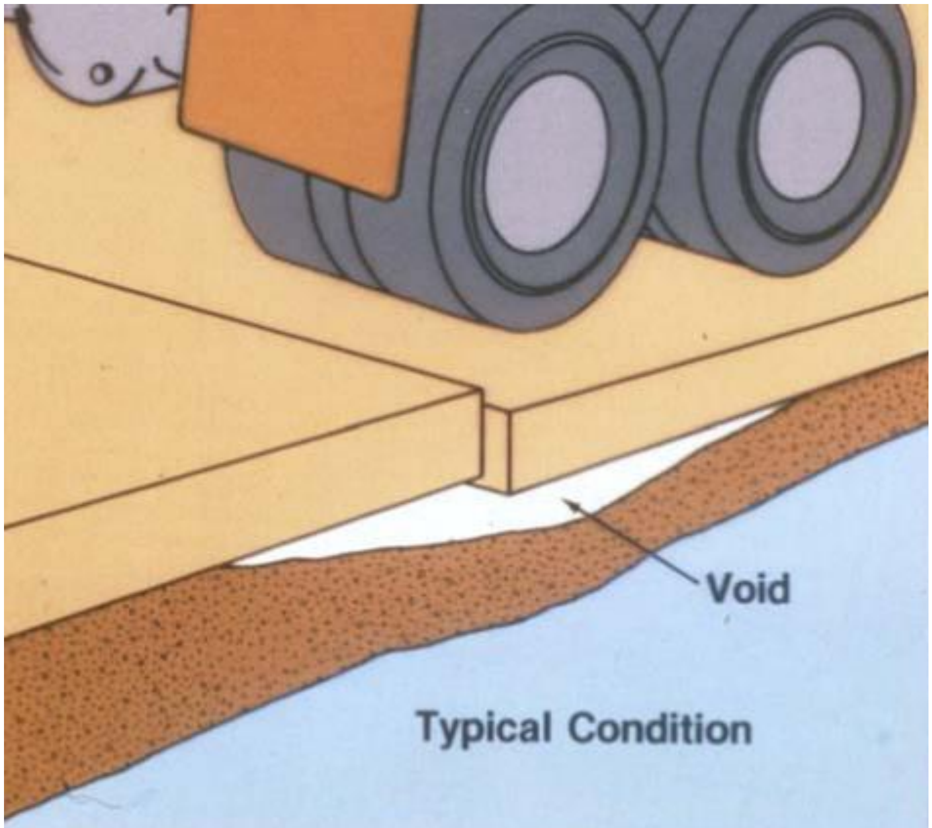


Figure 2-5: Development of Void under Joints (Stahl, 2006).

2.3. Falling Weight Deflectometer

The Falling Weight Deflectometer (FWD) was developed in the 1970s and was soon adopted as the standard method to carry out deflection testing on pavement structures. The FWD applies an impulse load on the pavement structure and measures its response via geophones placed at set intervals. The impact load is produced by dropping a known weight from a determined height on a loading plate, 150 mm in radius, resting on the pavement surface (Smith et al., 2017). The impact load on the pavement simulates a passing vehicle in load magnitude and duration. Figure 2-6 shows an example of an FWD system which consists of the following main components (Alavi, Lecates, & Tavares, 2008):

- Control system inside the truck
- Loading weights and plate
- Geophones
- Hydraulic system
- External camera to aid in testing alignment

Figure 2-6: Dynatest Falling Weight Deflectometer.

The control system performs the collection, processing and storing of collected data as well as identifying the drop height for each required load and moving the load plate and sensors bar down and up at the start and end of the test. Figure 2-7 shows a schematic of the FWD testing system. Figure 2-8 shows the load plate and geophones assembly from the equipment that was used as part of this research.

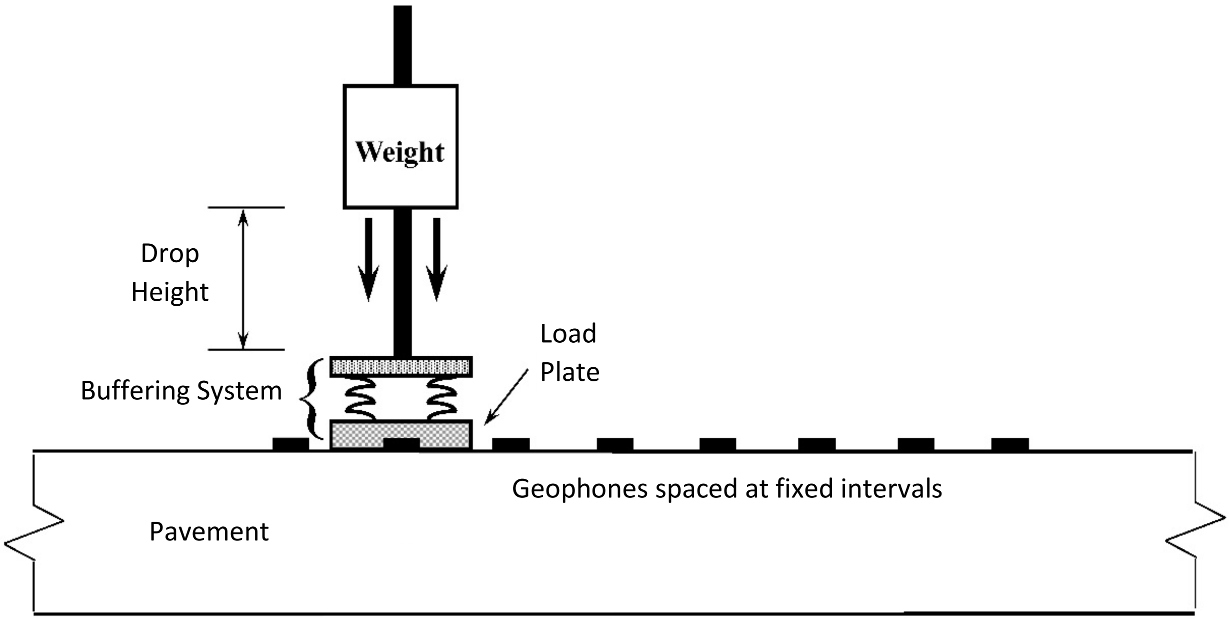
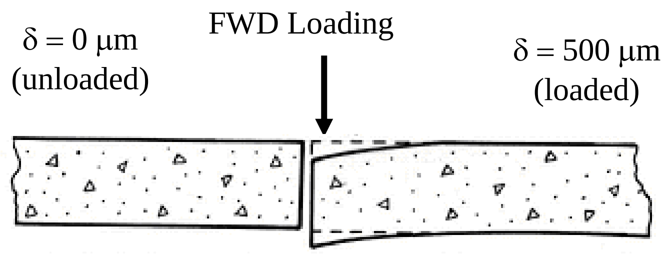


Figure 2-7: FWD Testing System Schematic (Smith et al., 2017).

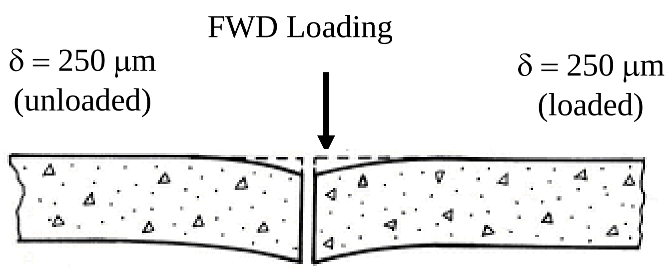
Figure 2-8: Falling Weight Deflectometer Load Plate and Deflection Geophones.

2.3.1. Joint Performance Evaluation

FWD deflection testing can be performed to evaluate the performance of joints through their load transfer efficiency (LTE) (Smith et al., 2017). LTE is the ratio between the deflection of the unloaded side of the joint and that of the loaded side of the joint. Figure 2-9 is a schematic of pavement deflections in failed and ideal conditions. Actual field joints performance typically falls between these two conditions.



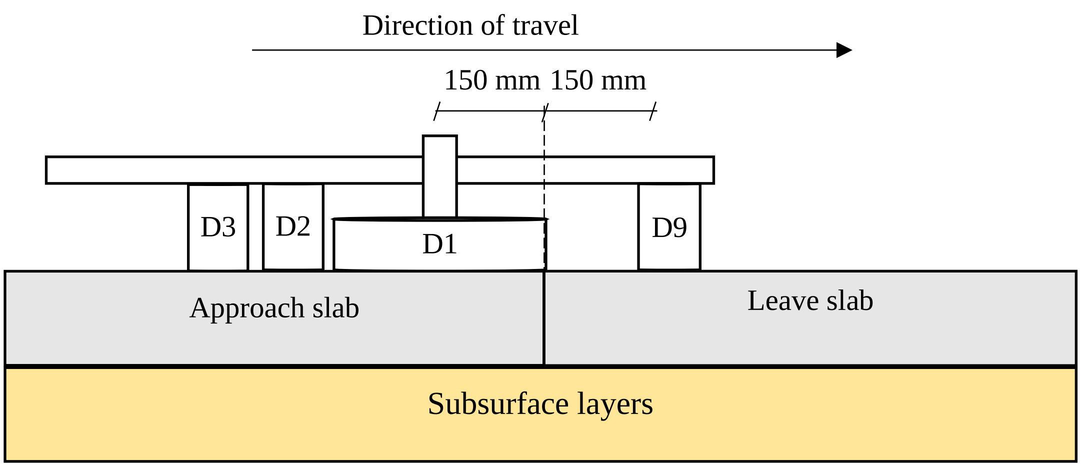
1) 0 % LTE



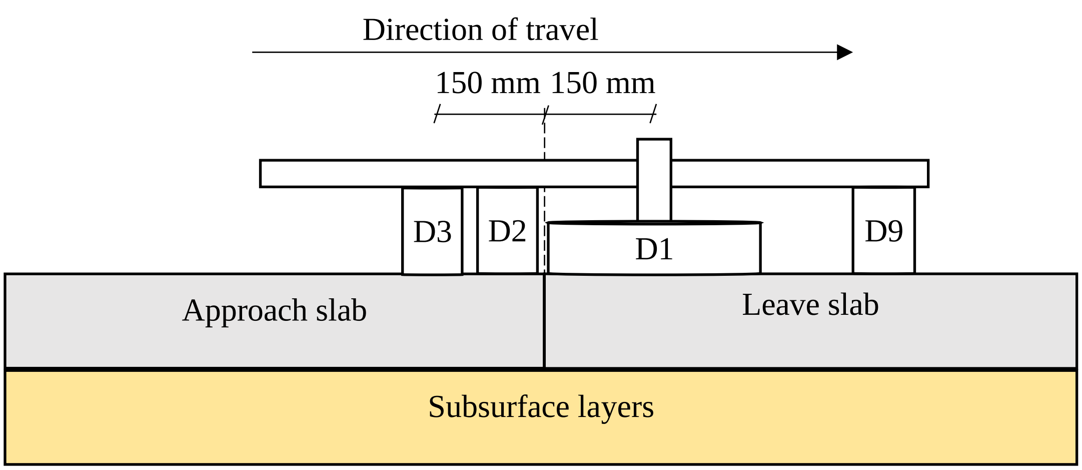
2) 100 % LTE

Figure 2-9: Comparison of a) 0 % and b) 100% LTE.

There are two geophones layouts that could be used in assessing joints performance. The Federal Highway Administration (FHWA) evaluates the approach and leave sides of joints for the Long-Term Pavement Performance (LTPP) test sections by positioning the edge of the loading plate tangential to the joint face using geophones layout as shown in Figure 2-10 (Schmalzer, 2006).



a) Approach Side of Joint



b) Leave Side of Joint

Figure 2-10: LTPP Geophones Layout in Evaluating a) Approach and b) Leave Sides of Joints.

In contrast, Dynatest® has recommended placing the loading plate away from the joint face and joint deflections are measured by geophones that are positioned 200 mm and 300 mm away from the loading plate, as shown in Figure 2-11.

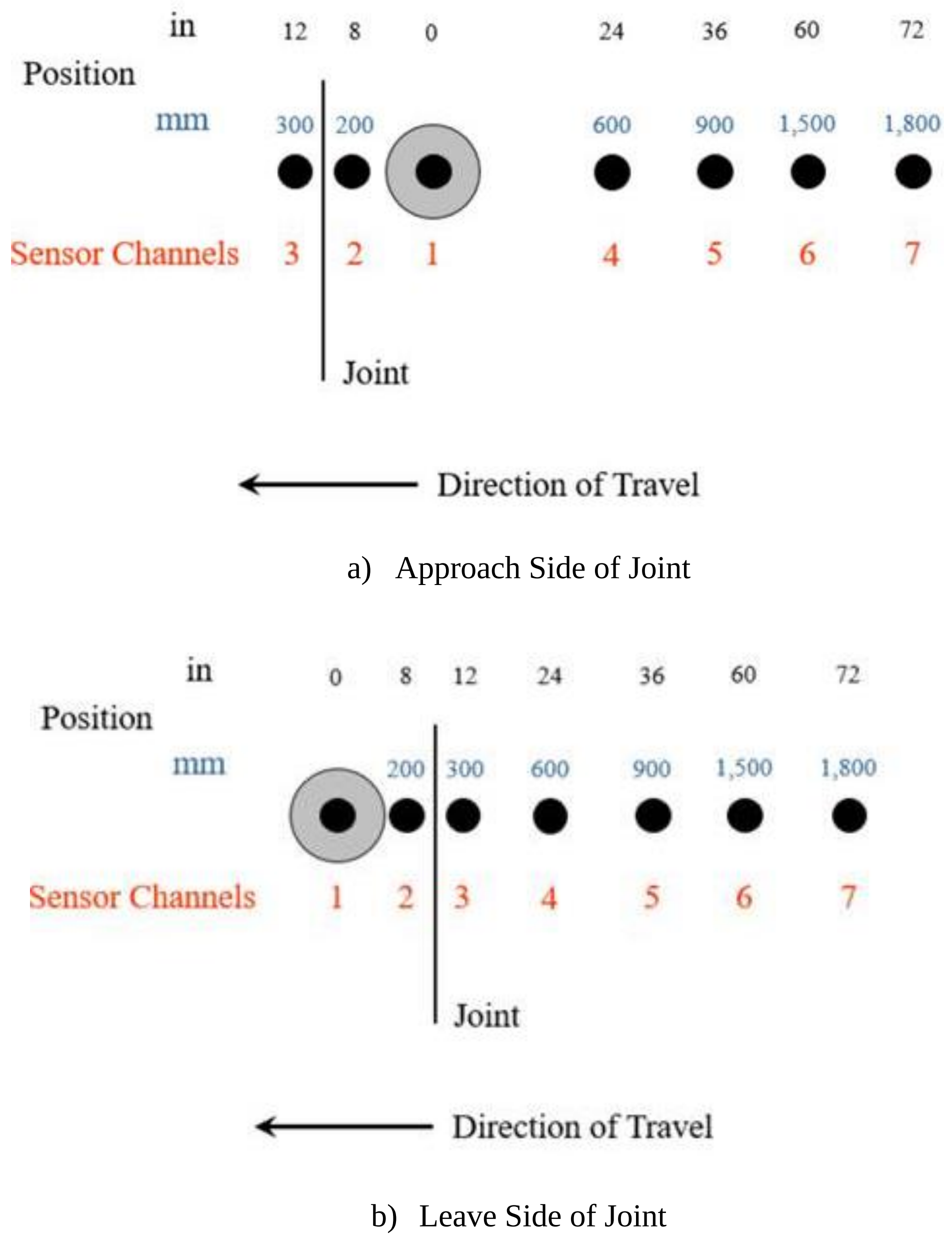


Figure 2-11: Dynatest® Geophones Layout in Evaluating a) Approach and b) Leave Sides of Joints.

In addition to computing LTE, Snyder (2011) states that load transfer efficiency should be considered along with other parameters such as peak deflections and differential deflections. LTE alone may not be a good indicator of joints performance since LTE is only a ratio. High LTE values, which indicate good performance, can be obtained even when joints experience high deflections that lead to pumping and cracking. Similarly, low LTE values, signaling poor performance, can be found when joint deflections are too low to cause any deterioration. Figure 2-12 shows a severely deteriorated joint at Saskatchewan Avenue while Figure 2-13 provides a schematic of differential deflection.

Figure 2-12: Severely Cracked and Deteriorated Joint.

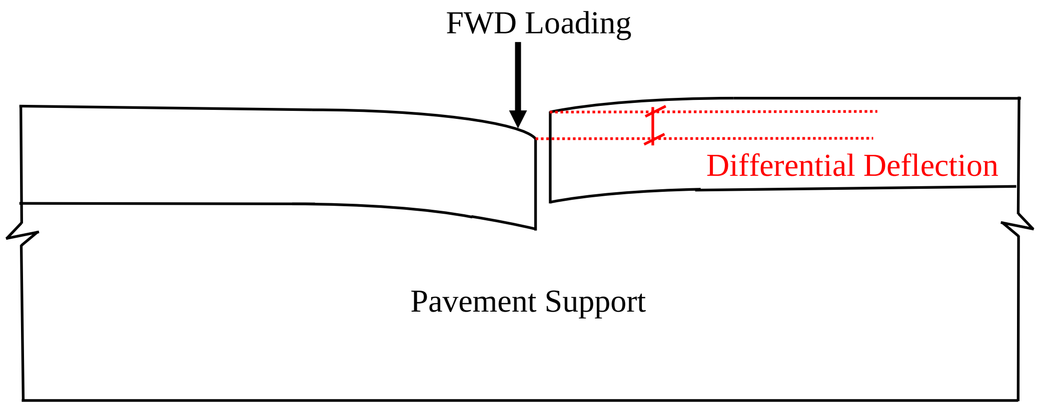


Figure 2-13: Illustration of Differential Deflection in Concrete Pavements.

Figure 2-14 through Figure 2-17 schematically demonstrate the importance of considering differential deflections with LTEs. In all the figures, the direction of travel is assumed to be from left to right and so the left side of the joint is the loaded while the right side is unloaded. In Figure 2-14, the computed LTE is 0% while differential deflection is 50 μm. In Figure 2-15, the LTE is still zero but the differential deflection is significantly higher at 500 μm. This is an example of how differential deflections can allow for a more robus evaluation of joints performance in addition to LTE. Figure 2-16 demonstrates the ideal condition shown in Figure 2-9 of a well-performing joint in which LTE is high (100%) and the differential deflections experienced are low (0 μm). Although large deflections are assumed in Figure 2-17, but the illustrasion shows a condition in which LTE is 88% but the differential deflections are also high at 300 μm compared to the other conditions. As such, differential deflections should be considered along with LTEs of joints.

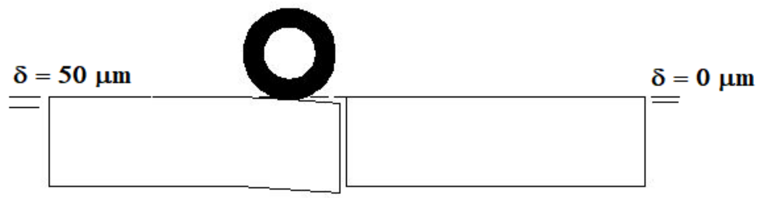


Figure 2-14: Schematic of Low LTE and Low Differential Deflection.

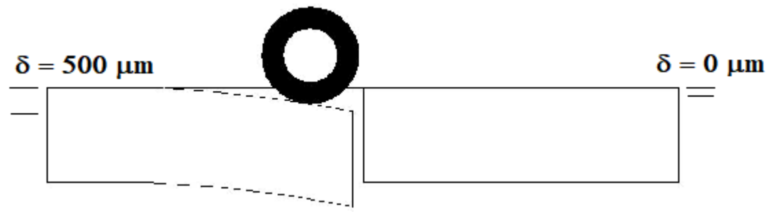


Figure 2-15: Schematic of Low LTE and High Differential Deflection.

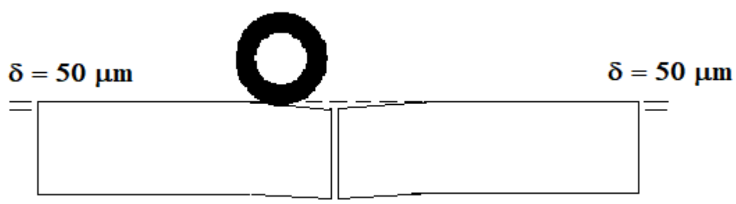


Figure 2-16: Schematic of High LTE and Low Differential Deflection.

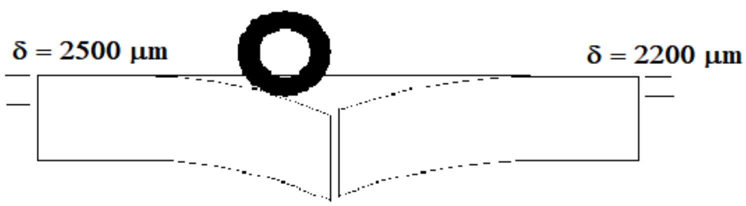


Figure 2-17: Schematic of High LTE and High Differential Deflection.

Washington State Department of Transportation (WSDOT) utilized FWD to determine effective Portland cement concrete (PCC) rehabilitation techniques through evaluating load transfer efficiency (LTE), corner slab differential deflection and void detection (Pierce, 1994). The study evaluated different rehabilitation and used FWD results to recommend the most cost-effective technique to restore load transfer in faulted PCC joints by evaluating LTEs and differential deflections of rehabilitation projects receiving dowel bar retrofitting, tied concrete shoulder, grinding and a combination of those. The project determined the effectiveness of each rehabilitation approach in Washington. Another study concluded that FWD could lead to characterizing pavement performance and, in turn, better rehabilitation planning (Zaghloul, He, Vitillo, & Kerr, 1996). FWD testing showed that base support is an important factor affecting deflections experienced by rigid pavements as low deflections (51–102 μm, or 2-4 mils, at a 40 kN load) were measured for cracked and non-cracked concrete sections with good support (Chen & Won, 2007). This indicates that, with good support, cracked rigid pavements may still experience low deflections and so support condition should be evaluated when the performance of a pavement structure is being characterized.

A joint performing well in terms of LTE and differential deflection can still deflect excessively and deteriorate if the slab is not supported well (Kathleen T Hall, Correa, & Scofield, 2001). This indicates that joints' peak deflections can be used to evaluate the support conditions. Meanwhile, the Minnesota Department of Transportation (MnDOT) uses differential deflections to evaluate dowel looseness in jointed PCC (Masten & Bruhn, 2017). Therefore, LTE, differential deflections and peak deflections are used as parameters to evaluate joints performance.

A number of studies on the application of FWD in evaluating different aspects of joints performances have been conducted. FWD was used to investigate Arizona's joints performance over 10 years period (Kathleen T Hall et al., 2001). In addition to other parameters, deflection measurements were used to compute LTE, differential deflections, total edge deflection, and transverse edge slab support ratio to monitor the long-term performance of joints and determine the most effective sealing treatment. FWD Field investigation on Texas DOT pavements was conducted to identify causes of joint pumping and settlement and to develop appropriate repair strategy (Chen, Won, & Hong, 2009). The study evaluated LTE, joints peak deflections on longitudinal joints and support condition in addition to other field tests to identify the presence of voids and the role base and subgrade support play in good JCP performance.

Moreover, the effectiveness of dowel bar retrofits (DBR) in restoring joint performance in the state of Texas was investigated using the FWD (Chen, Won, & Hong, 2011). FWD testing showed that DBR successfully improved LTE and minimized reflecting cracking if an AC overlay was placed. The results were also used to determine factors affecting DBR performance. The testing concluded that loose dowel bars were resulting in unsatisfactory DBR performance. Periodic FWD testing was performed to evaluate changes of LTEs at transverse joints to determine the causes of recurring full-depth repair failures and recommend designs to mitigate these causes (Chen, Zhou, Yi, & Won, 2014). In the study, FWD testing determined that LTE values were decreasing significantly within a short period of time which led to identifying tie bars rupture as a cause. FWD deflections and LTE findings were used to propose a two continuous reinforcements layers design with improved concrete support to address the identified issues. These studies demonstrate the FWD's use on a project level to investigate performance, identify factors that contribute to the performance of pavements and ultimately develop appropriate repair strategies that minimize cost and maximize performance.

2.3.2. Joint Performance Threshold Values

Different agencies and studies have adopted various trigger values for load transfer restoration/dowel bar retrofit and to assess a joint as to be in poor performance. Ministry of Transportation Ontario (MTO) specifies the use of FWD data along with visual conditions to trigger full-depth repairs (FDR) as shown in Table 2-1. This serves as an example of how FWD testing results can be considered with visual inspection as part of the decision-making process of selecting corrective action fot deteriorated joints. Table 2-2 summarizes the threshold values used by different agencies and studies.

Table 2-1:MTO's Full-Depth Repair Criteria for Dowelled JPCP. **(Chan & Lane, 2005)**

LTE	**Joint Severity**		
	Low	**Medium**	**High**
> 70%	No FDR	FDR decision based on FWD Void Detection / visual faulting, spalling and corner cracking	FDR
50 – 70%	FDR decision based on FWD Void Detection / visual faulting, spalling and corner cracking	FDR	FDR
< 50 %	FDR	FDR	FDR

Table 2-2: LTE and Differential Deflection Threshold Values in Various Studies and Agencies.

Study / Agency	LTE Threshold (%)*	Differential Deflection Threshold (μm) *
(Pavement Rehabilitation Manual, 1990)	70	250 (10 mils)
(Vandenbossche, 2007)	70	250 (10 mils)
(Jung, Freeman, & Zollinger, 2008)	70	-
(Larson & Smith, 2005)	-	130 (5 mils)
(Odden, Snyder, & Schultz, 2004)	70	130 (5 mils)
(Priddy, Pittman, & Flintsch, 2014)	70	130 (5 mils)
(Smith et al., 2017)	50 - 74	-
(Pennsylvania Department of Transportation, 2016)	65	-
(Minnesota Department of Transportation, 2017)	50 - 70	-
Ministry of Transportation Ontario (Chan & Lane, 2005)	50 – 70 with void detection analysis	-
(State of California Department of Transportation, 2008)	60	-
(American Concrete Pavement Association, 1997)	50	-

* "-" indicates that threshold was not mentioned by study/agency.

As seen in Table 2-2, most agencies and studies agree that for load transfer restoration, LTE threshold should be in the range of 50% to 70 %, with an agreement that LTE threshold to trigger repairs should not be higher than 70%. Table 2-2 reflects how LTEs and joint deflections can supplement visual inspection in the decision-making process of rehabilitation selection. In rehabilitation projects, joints with medium distress severity or LTE are difficult to elect for treatment as their load transfer or visual condition may be acceptable. FWD void detection analysis becomes instrumental in triggering repairs, as shown in Table 2-1. Differential Deflection threshold values fall under two categories – 130 μm and 250 μm. Although (Larson & Smith, 2005) suggested that joints with differential deflections exceeding 130 μm would not perform well in the long term, there does not seem to be an agreement on the differential deflection thresholds as in the case of LTE. To establish LTE and deflection thresholds, joints perforamnce in airfield panels were evaluated and their characteristics analysed (Priddy et al., 2014). The study concluded differential deflections should be considered with LTE for joint performance and 130 μm (5 mils) differential deflection and 70% LTE threshold values were proposed. A study aimed to establish peak deflections and differential deflections threshold values for Rolling Dynamic Deflectometer (RDD) by determining the deflection values corresponding to 60% and 70% FWD LTE (Nam et al., 2011).

Peak deflections are typically used by agencies to evaluate support conditions through determining the presence of voids and the need for undersealing or slab stabilization at the joints. Slab stabilization involves filling the voids under the concrete slab with grout or bituminous materials. Peak deflections can also be used in void detection as outlined in section 2.3.4. Figure 2-18 summarizes the peak deflection trigger values used in various states, produced from data found in (Taha, Selim, Hasan, & Lunde, 1994).

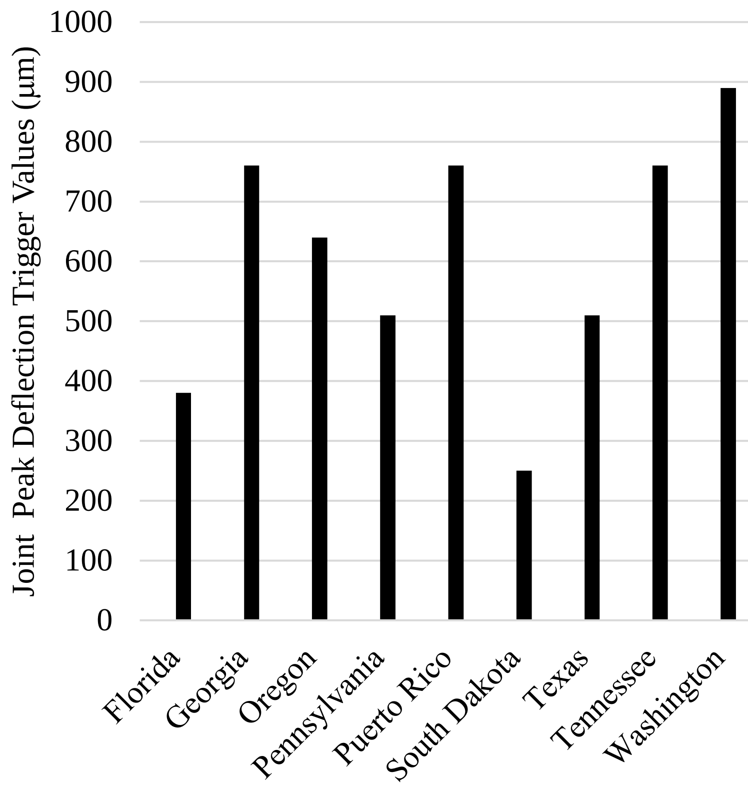


Figure 2-18: Peak Deflection Trigger Value in Different States.

Figure 2-18 shows that different states have very differing peak deflection trigger values for identifying the presence of voids and loss of support in order to perform undersealing/slab stabilization. These states use peak deflecton as an indicator of the support condition and if the peak deflections experienced are high, slab stabilization is performed to provide a better support to the concrete slab. Peak deflection trigger values range from 250 μm to 890 μm. These values are influenced by the type of support used in different states, traffic volumes, slab thicknesses as well as the quality of construction and climate. Therefore, it is rational that each state develops their own threshold values. However, there has not been a framework for determining these threshold values to date.

2.3.3. Joints Evaluation of Composite Pavements

Asphalt overlaying is a routine practice in concrete pavements rehabilitation/preservation wherein a hot mix asphalt surface layer is used to restore and maintain pavements structural adequacy and functionality. Many of existing concrete pavements have been overlaid to preserve the roadway systems. As many of these composite pavements age and become candidates for rehabilitation, appropriate evaluation of these structures is essential in selecting rehabilitation options, planning maintenance, and estimating quantities for bidding purposes.

Multiple studies have considered reflection cracking to be a majordistress type in composite pavements (National Cooperative Highway Research Program, 2004; R. Smith et al., 1984; H.L. Von Quintus et al., 1979). These cracks develop on the asphalt surface course where active cracks or joints exist in the underlying rigid pavement layer. In evaluating composite pavements, it is essential to determine the condition of the underlying concrete joint and crack, its corresponding base support and the presence of voids under the slabs. However, load transfer evaluation has primarily been developed based on deflection testing on the concrete surface directly. This has led to performing the FWD after the commencement of the rehabilitation project and the milling of the asphalt overlay.

The availability of joints deflections ahead of construction can provide information essential to the development of rehabilitation plans, selection of repair strategies and estimation of quantities and budgets for bidding. The FWD testing limitation can be mitigated if the effect of the asphalt overlay on measured deflections is examined and the presence of the asphalt overlay is accounted for so that FWD testing can be conducted prior to milling.

Hein et al (Hein, Olidis, Magni, & MacRae, 2002) noted that the selection of rehabilitation strategies and quantity estimates are difficult when FWD testing is performed on the asphalt overlay surface. The difficulty in evaluating composite pavements using FWD has been attributed to the compression of the asphalt layer (Construction Technology Laboratories, 2003). To limit the influence of asphalt compression on measured deflections, joints were tested prior to milling using geophones layout in Figure 2-19 as opposed to the conventional layout suggested by Long-Term Pavement Performance (LTPP) manual (Schmalzer, 2006). Dynatest® Inc. recommended a similar layout with different geophones spacing that effectively applies the pulse load further from the joint face to limit the effect of asphalt compression on deflection readings at the joint. However, this assumes that the asphalt layer only affects the deflection under the load (D_0) due to asphalt compression. It is believed, and will be confirmed in Chapter 4, that deflections further from the load (D_1, D_2,….) are still impacted by the presence of the asphalt because of the way stress is distributed from the top of the asphalt to top of the concrete surface. Moreover, applying the FWD load further from the joint face is anticipated to reduce the measured deflections and computed LTEs as it is not representative of the more critical load condition on the joint. This is equivalent to considering the load from a moving vehicle before it reaches the joint and cause maximum deflection at the joint compared to considering a vehicle load as it passes over the joint. Joints evaluation with different geophones layout are completed in this study to select the more critical layout for joints evaluation.

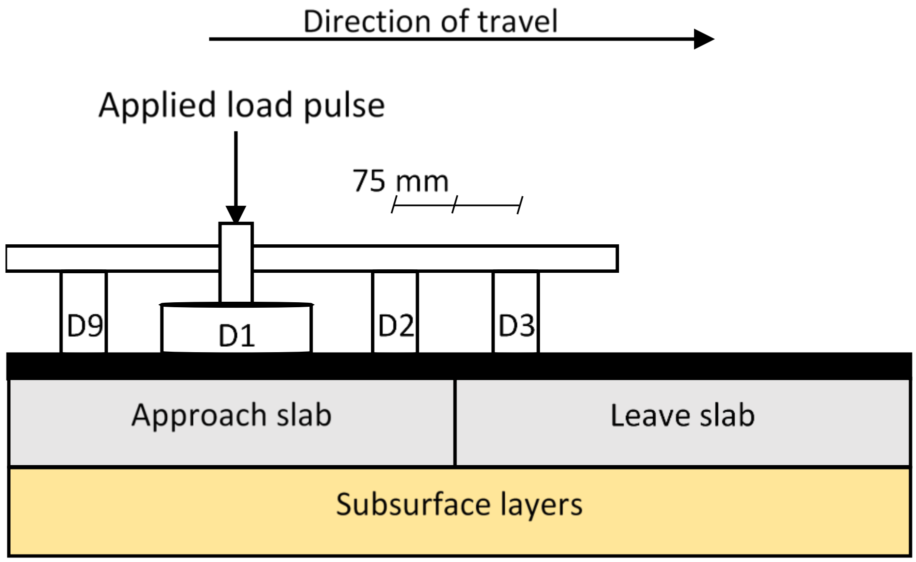


Figure 2-19: FWD Geophones Layout for Composite Joint Testing (Strata Engineering Corporation, 1998).

Construction Technology Laboratories (2003) found that LTEs computed from testing on the asphalt layer had significant variation as LTEs decreased when the severity of reflection cracking increased. However, Hein et al. concluded that concrete conditions could not be inferred from asphalt surface conditions accurately (Hein et al., 2002). The study also found that LTEs measured on the concrete surface were consistent with its visual conditions rather than the asphalt surface conditions. National Cooperative Highway Research Program (NCHRP) Web Document 35 (Project C1-38) report concluded the lack of an established procedure for void detection testing of composite pavements (Hall, Correa, Carpenter, & Elliott, 2001). Therefore, accurate concrete condition rating or joint FWD testing may only take place after the milling of the asphalt overlay. The City of Toronto recommends a detailed inspection of the exposed concrete upon milling the overlay layer to determine repairs needed (Applied Research Associates Inc., 2006). Composite pavements evaluation proposed by Hein et al. (2002) recommends FWD testing on the asphalt overlay and the exposed concrete pavement after milling. Testing the exposed concrete after the beginning of construction would result in opertaional constraints as operations will have to account for FWD testing time, data anlysis and time for decisions made using the newly available information. To overcome the impracticality of performing FWD testing on the exposed concrete, a correlation between joint performance (LTEs, peak deflections and differential deflections) from FWD testing on the asphalt surface and the concrete surface should be developed to provide a useful tool in the development of rehabilitation plans, selection of repair strategies and estimation of quantities and budgets for bidding.

2.3.4. Void Detection from Deflection Testing

In addition to evaluating load transfer, it is important to locate loss of support and the presence of voids under the concrete slab. However, existing void detection methodologies from FWD deflection data have been developed for concrete surface directly (Crovetti & Darter, 1985; Missouri Department of Transportation, 2004). With the increased implementation of FWD on composite pavements, the effect of asphalt overlays on joint performance, and void detection should be investigated and accounted for at the testing stage to improve the selection of rehabilitation strategies and quantity estimates for budgeting and bidding purposes.

Voids develop near PCC pavements joints when base materials arepumped and eroded under repeated loadings and/or ineffective load transfer (National Highway Institute, 1994) resulting in weak support, increased deflections under loading, and ultimately faster deterioration. Different methods have been developed for the identification of voids under joints which are summarized in Table 2-3.

Table 2-3: Summary of Void Detection Analysis Methods.

Reference	Method	Comment
(Crovetti & Darter, 1985)	Deflections analysis under different loads	Quick and simple to apply. Requires deflection tests under a minimum of 3 loads. It does not estimate the size of void as it does not take load transfer into account.
(Shahin, 1985)	Comparison of measured deflections and deflections from finite element analysis (FEA)	Estimating material properties, as well as joint load transfer modelling may be challenging. The process may be more accurate but requires computational power.
(Ullidtz, 1987)	Comparison of k-value at the edge of the slab and k-value at slab center	Voids can exist below slab center reducing k-value and affecting the ratio of joint and center k-values.
(Smith et al., 2017)	Deflection comparison to pre-set threshold or project average	Load transfer variation can affect deflection values. Recommended to only be used as an indicator of potential voids.

Crovetti's method of void detection for jointed plain concrete joints has been the more widely used approach due to its simplicity and fair accuracy. It uses peak deflections corresponding to the different load levels at a test location to determine the best-fit line on a deflection-load plot. The best-fit line is then extrapolated to find the deflection intercept when the load is zero, D_0. If the deflection intercept is larger than 50 μm (2 mils), a void is identified. The void detection procedure is constructed for illustration purposes in Figure 2-20. Crovetti recommended that testing of joints be under at least three load levels including 40 kN within the range (Crovetti & Darter, 1985).

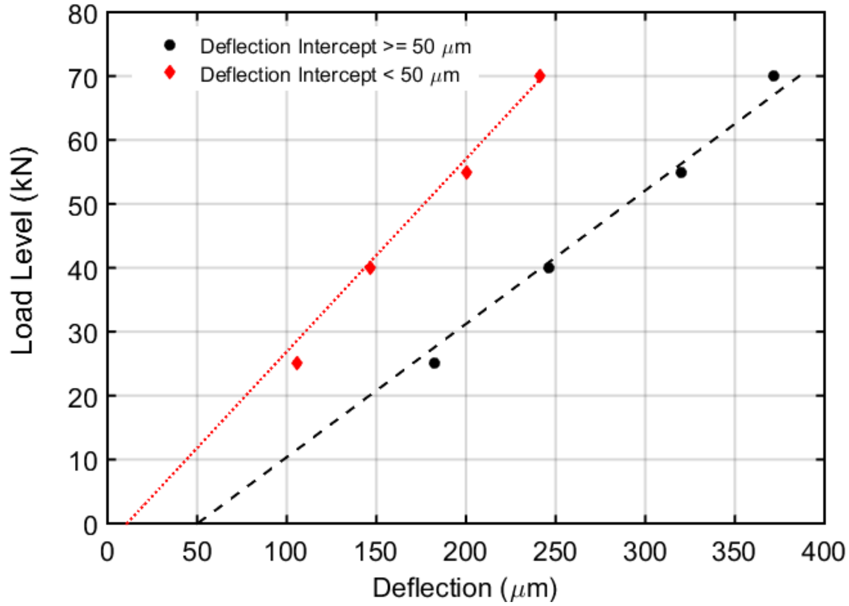


Figure 2-20: Crovetti's Void Detection Procedure Using FWD Data

2.3.5. Backcalculation Software

There are a number of backcalculation methods that differ in the theory used in their development, the type of pavement system analysed, number of layers and method of convergence. In estimating pavement stiffness, closed-form solutions have been developed as well as software that employs iterations in their estimation. Table 2-4 summarizes a number of backcalculation software, their pavement types compatibility and their forward, backcalculation and convergence approaches.

Many studies have been conducted to compare backcalculation results obtained from different software. More information on the comparison of different backcalculation software can be found in Ameri, Yavari, & Scullion (2009); Ellis (2008); Mahoney, Coetzee, Stubstad, & Lee (1989);

Tarefder & Ahmed (2013); Harold L Von Quintus, Rao, & Irwin (2015); and Yin & Mrawira (2009). Details of closed form solutions can be found in Khazanovich, Tayabji, & Darter (2001) and Smith et al. (2017).

Table 2-4 shows that only three software are compatible with both rigid and flexible pavement. ELMOD, which stands for Evaluation of Layer Moduli and Overlay Design, is proprietary software that is available with Dynatest® FWD equipment, which most agencies use, including Manitoba Department of Infrastructure (MI). Moreover, it has the capability to conduct backcalculation using method of equivalent thickness (MET), developed by (Odemark, 1949), and finite element analysis. It is also capable of batch processing FWD field data as well as incorporating Ground Penetrating Radar (GPR) layer thicknesses in the analysis. For the reasons aforementioned, it was adopted for backcalculation in this study.

MET analysis is based on the assumption that deflections experienced by a multilayered pavement structure with *hi* thicknesses and *Ei* moduli can be analysed as a one layer system of thickness H and modulus *E* provided that *H* satisfies the equation below (Ullidtz, 1987):

$$H = \sum_{i}^{n} Ch_i \left(\frac{E_i}{E}\right)^{\frac{1}{3}}$$

Wherein H = thickness of one layer system, C = layer coefficient (0.8-0.9), hi = thickness of each layer, Ei = modulus of each layer, and E = modulus of one layer system.

Table 2-4: Select Summary of Backcalculation Software (K. Smith et al., 2017).

Software	Developer	Public Domain	Analyzed Pavement Type	Forward Calculation Approach	Backcalculation Approach	Convergence Approach
BAKFAA (2013)	Federal Aviation Administration (FAA)	Yes	Flexible & Rigid	Multilayer Elastic Theory	Iterative	Sum of Squares of Absolute Error
CHEVDEF (1980)	USACE-WES	Yes	Flexible	Multilayer Elastic Theory	Iterative	Sum of Squares of Absolute Error
DIPLOBACK (1997)	Khazanovich and Roesler	No	Composite	Multilayer Elastic and Westergaard	Closed Form Solution	Closed Form Solution
ELMOD® (2009)	Ullidtz (for Dynatest®)	No	Flexible & Rigid	Equivalent Thickness Method and Finite Element	Iterative	Relative Error of 5 Sensors
EVERCALC© (2001)	Mahoney et al.	Yes	Flexible	Multilayer Elastic Theory	Optimization	Sum of Absolute Error
ILLI-BACK (1994)	Ioannides	No	Rigid and Composite	Closed Form Solution	Closed Form Solution	Closed Form Solution
MODCOMP© (1983)	Irwin, Szebenyi	Yes	Flexible	Multilayer Elastic Theory	Iterative	Relative Deflection Error at Sensors
MODULUS (1999)	Texas Transportation Institute	Yes	Flexible	Multilayer Elastic Theory	Database (Optimization)	Sum of relative squared error

Chapter 3

NON-DESTRUCTIVE TESTING PROGRAM

3.1. Test Sections Overview

Three arterial pavement sections, rehabilitated in 2017, were selected for FWD testing. Figure 3-1 shows the locations of these projects, namely Brookside Boulevard, McGillivray Boulevard, and Pembina Highway. Table 3-1 summarizes the projects limits and the planned rehabilitation and Table 3-2 shows background information about the test sections' including functional class, structure, Annual Average Daily Traffic (AADT), construction year and condition as provided by the City of Winnipeg. FWD joint and basin tests were also performedzin 2018 on six (6) residential streets in the City of Winnipeg. Table 3-3 summarises the background information of these residential streets and Figure 3-2 shows the typical pavement structure of these streets. Figure 3-3 shows the outline of the methodology to meet the study's objectives

Brookside Boulevard and McGillivray Boulevard are concrete pavements that were scheduled to receive some partial and full-depth repairs based on their visual conditions and then asphalt overlays. For this reason, Brookside Boulevard and McGillivray Boulevard were tested prior to and after repairs to establish threshold values for joint load transfer efficiencies (LTEs) and deflections, and study these parameters of newly constructed full-depth repairs. Pembina Highway was tested prior to and after asphalt overlay milling to characterize the effect of asphalt overlays on deflections, LTEs and void detection. Residential streets were all of the same typical structure and their FWD results were utilized in assessing residential streets' performance and structural capacity in comparison to arterial regional roads.

Table 3-1: Summary of Arterial Regional FWD Project Limits, Structures and Planned Rehabilitation.

Project	Direction	Project Extent	Planned Rehabilitation
Brookside Boulevard	Southbound	Mollard Road to 100m North of Inkster Boulevard	1. Joints partial and full-depth repairs. 2. Paving a new asphalt overlay.
McGillivray Boulevard	Westbound	Fennell St. to Waverly St	1. Joints full depth repairs. 2. Paving a new asphalt overlay.
Pembina Highway	Northbound and Southbound	DeVos Rd. to Kirkbridge Drive/Killarney Ave.	1. Milling the existing asphalt overlay. 2. Joint repairs on the concrete layer. 3. Adding an active transportation (cycling) lane. 4. Paving a new asphalt overlay.

Table 3-2: Arterial Regional Test Sections Background Information.

Section	Functional Class	Average Asphalt Thickness (mm)	Average Concrete Thickness (mm)	AADT	Construction Year	Condition	Rate Year
Brookside Boulevard	Arterial	-	230	10,534	1984	Fair	2015
McGillivray Boulevard	Arterial	-	230	7,868	1970	Poor	2015
Pembina Highway	Arterial	100	200	13,757	1960	Good	2015

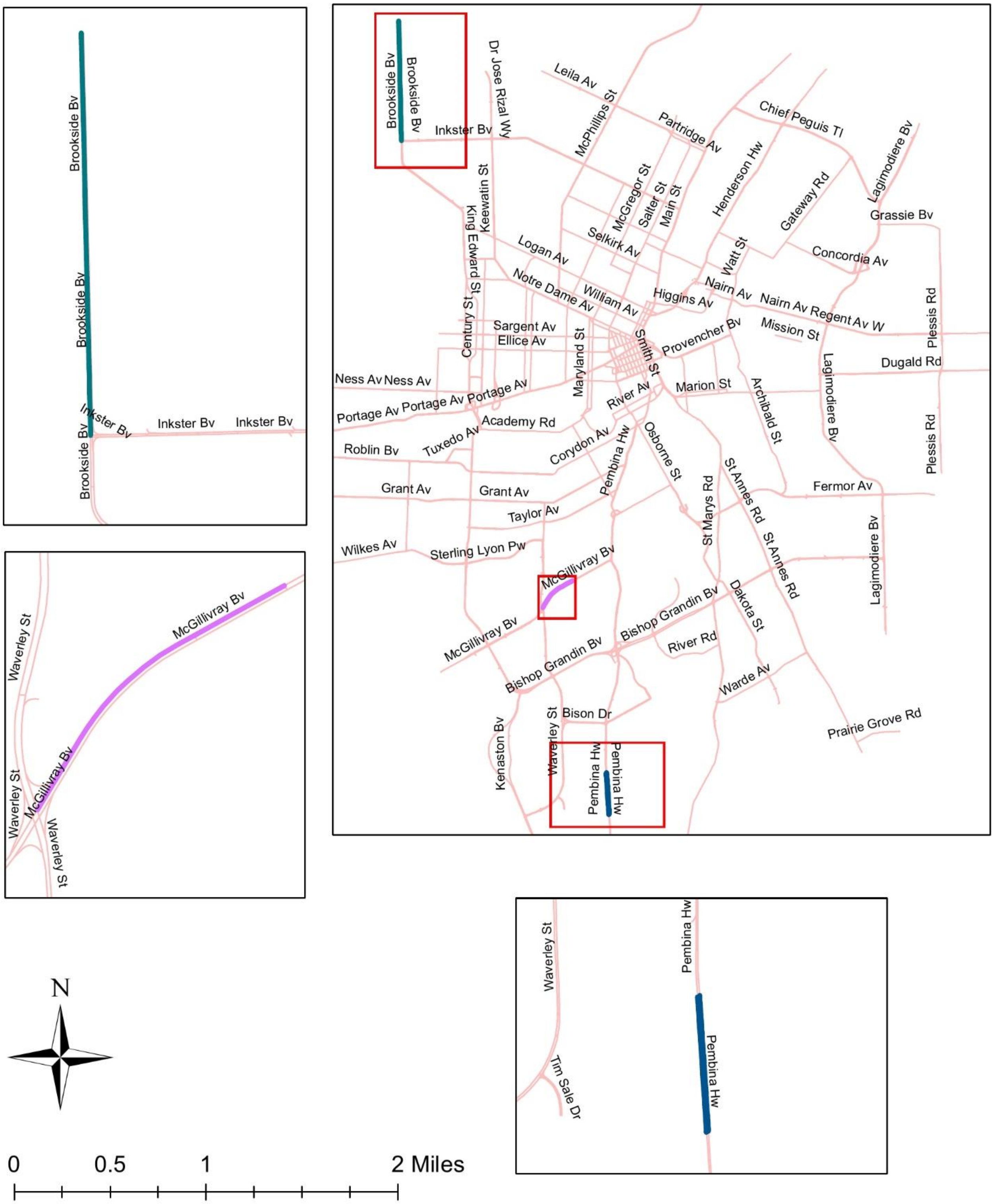


Figure 3-1: Location of Pavement Sections Scheduled for NDT in the City of Winnipeg.

Table 3-3: Residential Streets Test Sections Background Information.

Section*	Functional Class	Construction Year	Condition	Rate Year
Highgate Crescent	Local	1983	Good	2015
Bethune Way	Local	1987	Good	2014
Carmarthen Boulevard	Local	1965	Fair	2015
Oswald Bay	Local	1977	Fair	2015
Best Street	Local	1963	Fair	2015
Barbara Crescent	Local	1978	Good	2015

* AADT data is not available for these sections

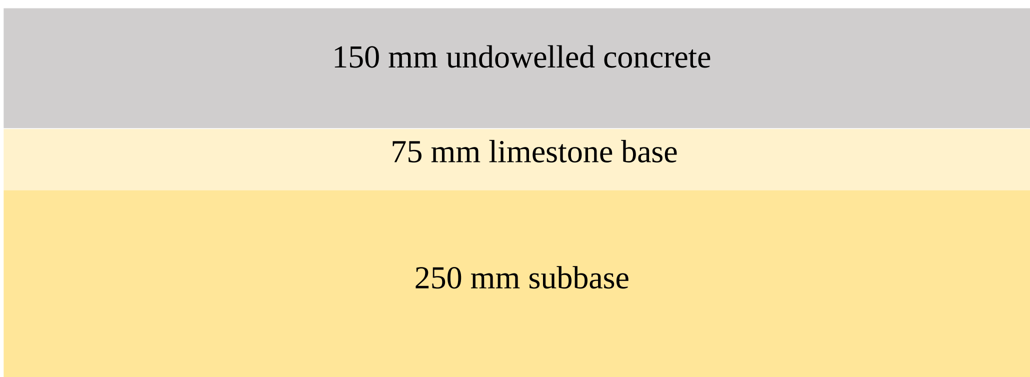


Figure 3-2: Typical Residential Street Pavement Structure.

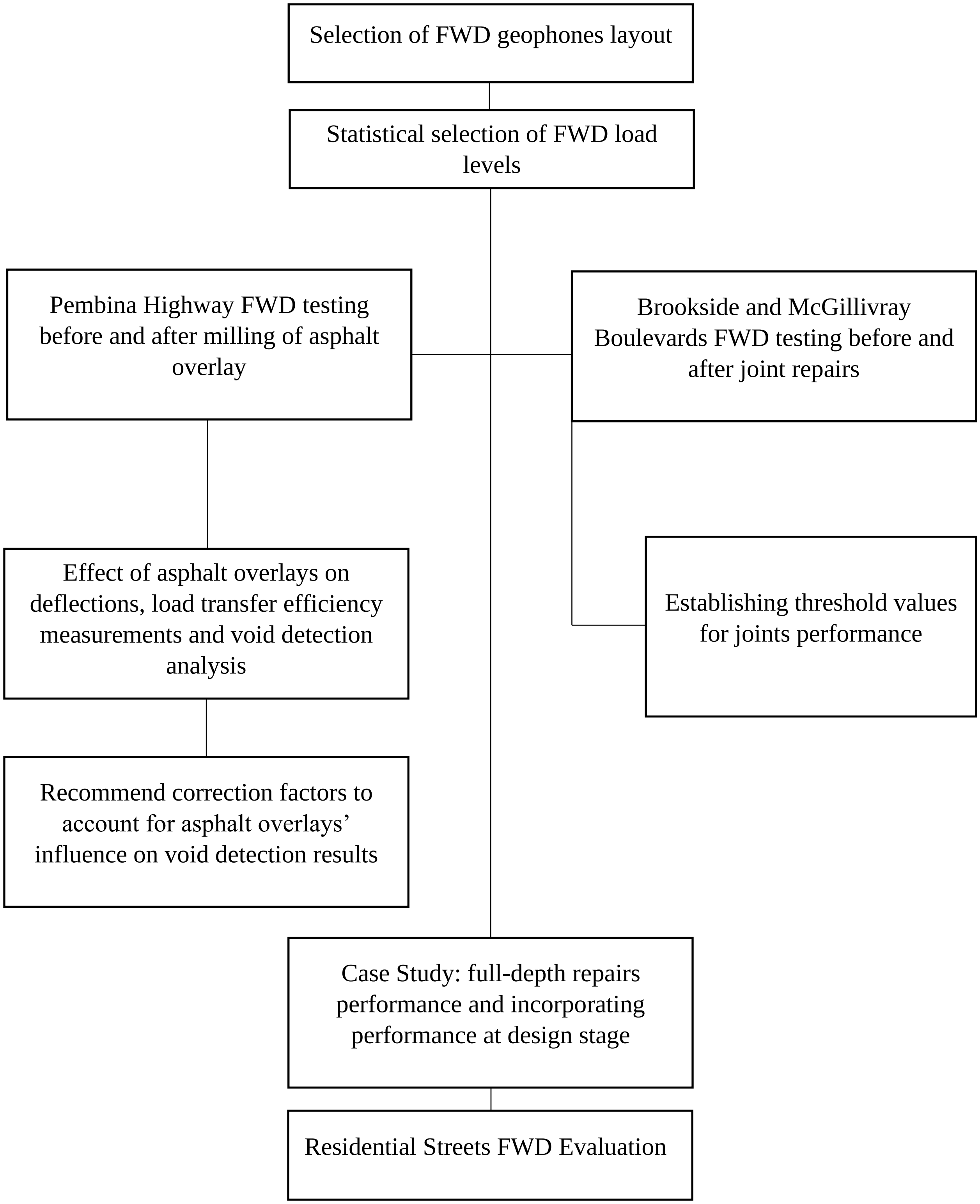


Figure 3-3: Outline of Methodology

3.2. Falling Weight Deflectometer Testing Program

Field testing was carried out using Dynatest Model 8002 truck mounted FWD shown in Figure 3-4. To supplement FWD's global positioning system (GPS), Real Time Kinematic (RTK) survey equipment was used to capture test locations coordinates for retesting after overlay milling and after full-depth repairs (FDRs). FWD applies an impulse load on the surface of the pavement by dropping a known weight from a certain height on a buffer system. The target load is then transferred to the pavement surface through a 150 mm radius plate. The response of the pavement structure is then recorded using deflection sensors, or geophones, placed at specific intervals. All deflection testing was carried out using four (4) load levels – 25 kN, 40 kN, 55 kN, and 70 kN – and the tests were carried out with two (2) drops per load level. Each applied load is acceptable only if it falls within ±10% of the target load level. Table 3-4 shows the range of acceptable loads during FWD testing for each drop height.

Figure 3-4: Falling Weight Deflectometer with RTK Survey Equipment.

Table 3-4: FWD Target Loads and Acceptable Ranges.

Drop Height	Target Load (kN)	Acceptable Range (kN)
1	25	22.5 to 27.5
2	40	36.0 to 44.0
3	55	49.5 to 60.5
4	70	63.0 to 77.0

Following the selection of geophones layout, FWD deflection testing was performed on Pembina Highway, Brookside Boulevard and McGillivray Boulevard and six residential streets in the City of Winnipeg. The structures of each roadway are outlined in Table 3-2 and Figure 3-2. The geophones layout adopted for joint testing on these sections is LTPP's layout. A total of sixty-three (63) joints were tested on Pembina Highway prior to and after the milling of the asphalt overlay to evaluate its effect on deflections, load transfer efficiency measurements, and void detection. A total of fifty (50) joints were tested on Brookside Boulevard and McGillivray Boulevard to select of geophones layout and evaluate the performance of joint repairs. For every joint tested, the leave and approach side of the joint were tested as shown in Figure 2-10. Figure 3-5 and Figure 3-6 show the geophones layout during field testing of approach and leave sides of joints on Pembina Highway and McGillivray Boulevard, respectively.

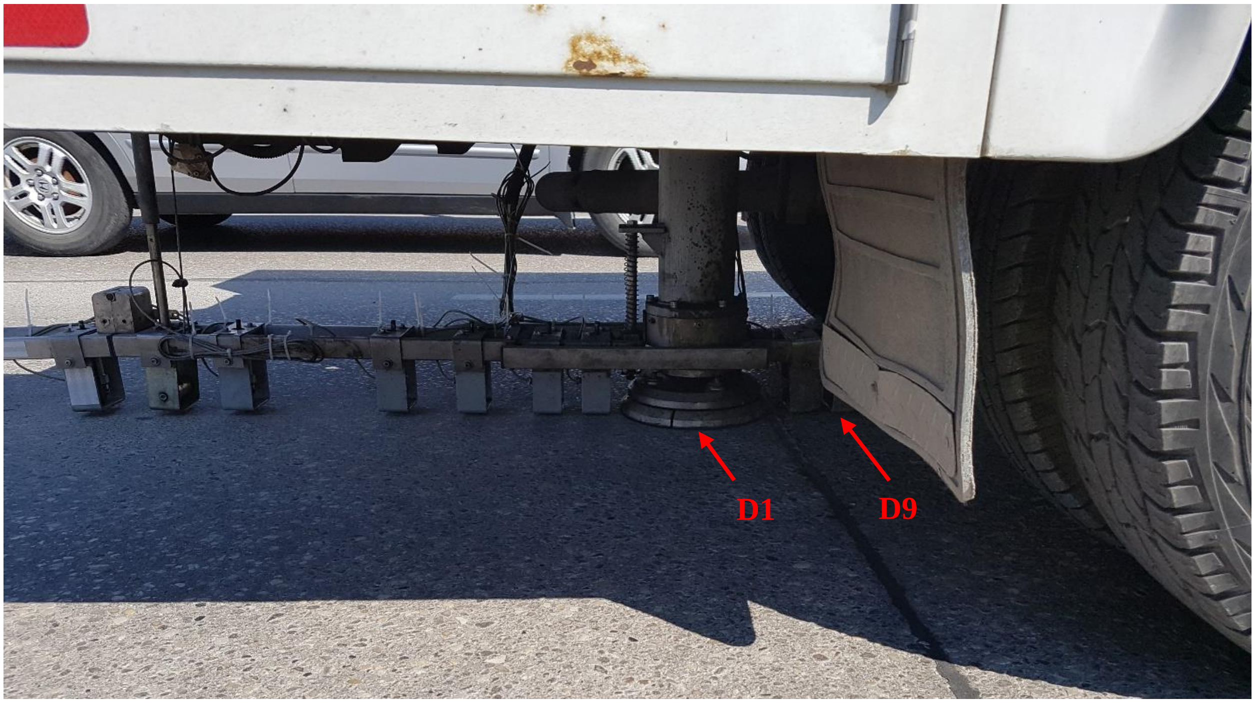


Figure 3-5: Approach Joint FWD Testing on Asphalt Overlay of Pembina Highway.

Figure 3-6: Leave Joint FWD Testing on Concrete Pavement of McGillivray Boulevard.

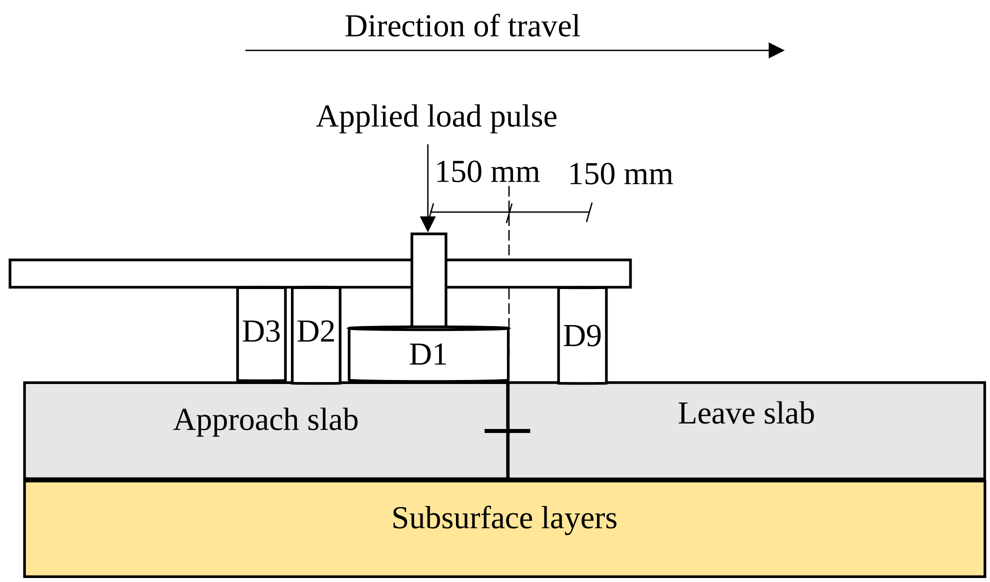
3.2.1. FWD Testing for Selection of Geophones Layout

To select the geophones layout used in subsequent testing, joints were tested at the same time using LTPP and Dynatest® recommended geophones layouts shown in Figure 3-7. In LTPP layout, the loading plate is placed tangential to the joint face and in the Dynatest® layout, the loading plate is placed away from the joint face with the joint being in between two geophones. LTE equation for each layout is outlined in equations (3-1) and (3-2) below:

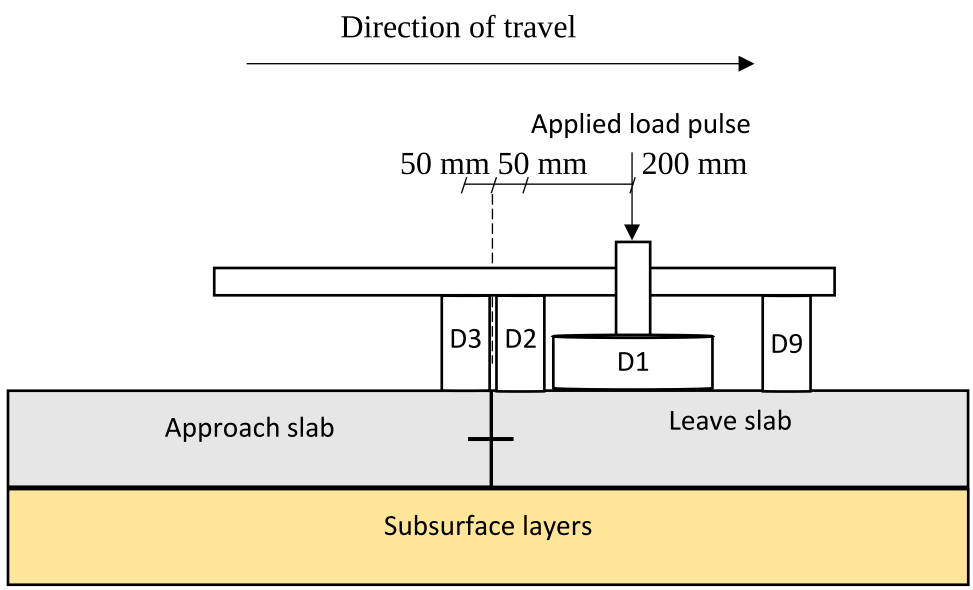
$$LTE_{LTPP} = \frac{D_9}{D_1} \times 100\% \quad (3\text{-}1)$$

$$LTE_{Dynatest} = \frac{D_3}{D_2} \times 100\% \quad (3\text{-}2)$$

Peak deflections, differential deflections, and LTEs obtained from each layout was evaluated. The layout that produced lower LTE and higher deflections was considered to represent the more critical joint loading condition and was, therefore, selected for subsequent testing. Different layouts apply the impulse load at different offsets from the face of the joint and compute LTEs using geophones at differing offset from each other and the face of the joint. Therefore, they are anticipated to simulate different loading situation on the joint. For instance, a layout applying the load 150 mm away from the joint face is considered to simulate a different loading condition compared to a layout applying the load 250 mm away from the joint face. The more critical layout for joint testing is considered to be the one that computes more adverse LTEs and deflections at joints.



a) LTPP Geophones Layout.



b) Dynatest® Geophones Layout

Figure 3-7: FWD Testing Layouts a) LTPP and b) Dynatest® Geophones Layouts.

3.2.2. Threshold Values for Joint Performance Parameters

It has been demonstrated that no threshold values were recommended for the peak and differential deflections and the LTE for a well-performing joint is recommended to be above 50%-70%. However, LTE value of 50% is considered too low for arterial roads and MTO recommends a full-depth repairs for such joints regardless of their visual condition. Therefore, a correlation between the computed LTEs from field testing and their corresponding peak deflections and differential deflections was carried out to investigate what deflection values corresponding to LTE value of 60% and 70%. Moreover, since the support condition and presence of voids under the concrete slab impact the deflections experienced by joints, a correlation between void detection x-intercepts and measured peak deflections was established to determine the peak deflection value corresponding to the x-intercept of 50 μm (or 2 mils). This is the deflection x-intercept value above which a potential void is identified under the concerete slab. Peak deflection and differential deflection values obtained from these correlations are then selected as threshold values that could trigger different treatments.

3.2.3. Statistical Selection of FWD Load Levels to Optimize FWD Testing

Paired student t-test statistical analysis of FWD results was performed in order to reduce the number of load levels used in each test from four load levels (25 kN, 40 kN, 55 kN, and 70 kN) to two load levels. Falling weight deflectometer's (FWD) 40 kN load is equivalent to a standard 80-kN (18000-lbs) axle load and so it is typically used in FWD testing for most highway pavement testing (K. Smith et al., 2017). Moreover, 40 kN load level is recommended to be used within the load range as part of the void detection methodology. Therefore, the 40 kN load level will be maintained for testing and a second load level which produces statistically similar results will be selected to maintain the level of variation in test results obtained from four load levels.

T-test results for LTE, peak deflections and differential deflections were tabulated in matrices to identify the loads under which each parameter is statistically similar to the other. Statistical testing was also completed for void detection x-intercept obtained from four load levels and three combinations of two load levels, namely 25 kN and 40 kN, 40 kN and 55 kN, and 40 kN and 70 kN. The combinations of two load levels which produce statistically different LTE, peak deflections, and differential deflections while producing statistically similar void detection results are selected to optimize FWD testing time and cost.

3.2.4. Effect of Asphalt Overlays

The effect of asphalt overlays on deflection measurements was evaluated by examining the difference in deflections prior to and after milling of the asphalt overlay. A correlation between deflections prior to and after milling was then established to predict the deflection of the pavement structure without milling the asphalt layer. Half of the deflection dataset obtained from FWD testing before and after milling was used for developing the correction factors and the other half was used to validate the factors recommended.

Deflections after milling would be estimated using a correction factor accounting for the asphalt layer, $F_{Asphalt}$, from deflections obtained prior to milling using the following equation

$$D_{after\ milling} = F_{Asphalt\ Layer} \cdot D_{before\ milling}$$

In turn, LTEs of the concrete layer and the concrete's condition, can be estimated prior to the milling of the asphalt overlay and void detection analysis can be performed to detect voids under the concrete slab before the milling of the asphalt layer. Improvements in estimating LTE, deflections and void detection was evaluated after the deflections are corrected.

3.2.5. Deflection Data Analysis

Each deflection testing was performed under four target load levels – 25kN, 40 kN, 55 kN, and 70 kN. Since the resulting peak loads vary slightly from the target loads due to friction in the hydraulic system dropping the load and dampers stiffness variation, deflections are normalized to each target load using Equation 3-3:

$$D_n = D_m \frac{L_t}{L_m} \quad \text{3-3)}$$

Where D_n = normalized deflection, D_m= measured deflection, L_t = target load, and L_m = measured load.

LTE is defined as a ratio of the deflection transferred from the loaded side of the joint to the unloaded side and can be calculated using Equation (3-4 as shown below:

$$LTE = \frac{D_u}{D_l} \times 100\% \quad (3\text{-}4)$$

Where LTE = deflection-based load transfer efficiency, D_u = deflection of unloaded side of the joint, and D_l = deflection of loaded side of the joint.

Load transfer can be achieved by aggregate interlock, support of underlying layers, mechanical devices, temperature, or a combination of these factors (K. Smith et al., 2017). Other performance parameters considered in this study are the peak deflection, D_1 or D_l, which is the absolute deflection under the loading plate, as well as the differential deflection (DD) which is defined in Equation (3-5 below:

$$DD = D_l - D_u \quad (3\text{-}5)$$

3.2.6. Backcalculation for Layer Stiffness

As discussed in section 2.3.5, ELMOD backcalculation software was selected to analyze FWD basin deflections and estimate pavement layer stiffnesses. The pavement structures modelled are according to the structure information outlined in section 3.1 for each pavement section.

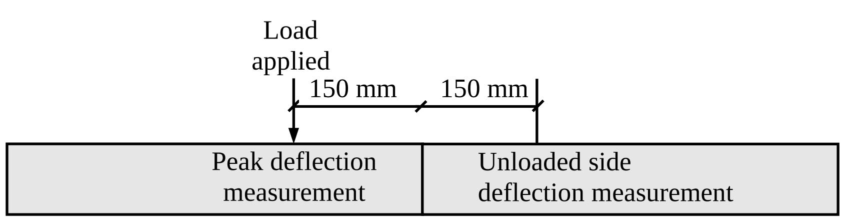
Chapter 4

FALLING WEIGHT DEFLECTOMETER TESTING RESULTS

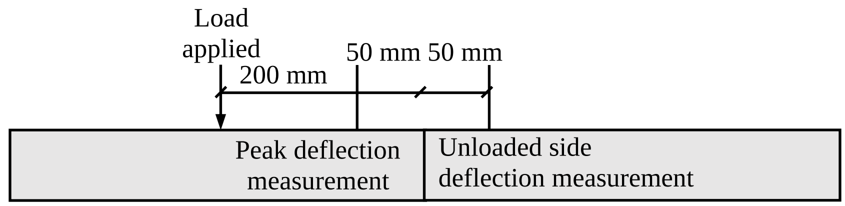
This chapter presents the results from FWD testing conducted on the City of Winnipeg regional roads and residential streets during 2017 and 2018. Testing was completed on Pembina Highway prior to and after milling of the asphalt overlay and on McGillivray Boulevard and Brookside Boulevard prior to and after rehabilitation.

4.1. Selection of Geophones Layout

Joint testing was completed using LTPP and Dynatest® geophone layouts as previously shown in Figure 3-7. In addition to dropping the loads at different distances from the edge of the slab, the deflections are also measured at different locations off the slab edge. For example, Figure 4-1a shows that the load application and peak deflection measurement are 150 mm from the slab edge in LTPP's layout. In Dynatest® layout, the load is applied 250 mm from the slab's edge and deflections are recorded 50 mm on either side of the joint face, Figure 4-1b. This also shows that LTEs, peak deflections and differential deflections are computed using geophones that are 300 mm apart for LTPP layout and 100 mm apart for Dynatest® layout.



a) LTPP Layout



b) Dynatest® Layout

Figure 4-1: Difference in Load Application and Deflections Measurement Locations Between a) LTPP and b) Dynatest® Layouts.

Each joint was tested under 25, kN, 40 kN, 55 kN, and 70 kN load levels. The testing aimed to evaluate the influence of geophones layout on the measured deflections and computed performance parameters to select one layout for subsequent testing.

Figure 4-2a shows that placing the load plate further from the edge of the slab leads to overestimating computed LTE wherein LTEs were generally higher when tests were conducted using Dynatest® layout compared to those using LTPP layout. Dynatest® layout computes LTEs using sensors only 100 mm away from each other, and so their deflections ratio is anticipated to be higher as opposed to when LTEs are computed using deflections 300 mm apart. This was reflected in the results of statistical analysis, in Table 4-1, wherein LTEs computed from the two layouts were found to be statistically different.

Peak deflections recorded 150 mm away from the slab edge in LTPP layout were similar to those recorded 50 mm away from slab edge in Dynatest® layout. The deflection directly under the load 150 mm away from the slab edge is closely similar to the deflection 50 mm away from edge when the load is 250 mm away, as in Figure 4-2b. This means using Dynatest® layout, deflections

50 mm away from the slab edge can be used to conduct void detection since the analysis is performed using peak deflections from LTPP analysis. Table 4-1 shows that peak deflections are statistically similar for both geophone layouts.

Differential deflections determined through the Dynatest® layout, shown in Figure 4-2c, were lower and statistically different than those determined through the LTPP layout. This is in line with LTE results wherein the difference in deflections determined from deflections 100 mm away from each other is less than the difference determined from deflections 300 mm away. Figure 4-3 further elaborates how LTPP layouts computes LTEs using D_1 and D_2 that are 300 mm away from each other and would, therefore, result in lower LTEs than Dyantest layout. Figure 4-4 shows the deflection bowl of Dyantest layout in joint testing. The Dynatest layout applies the load further away from the joint face than LTPP layout and uses D_2 and D_3 for LTE computation. Since D_2 and D_3 in Dynatest layout are closer than D_1 and D_2 in LTPP layout, the ratio of the deflections and computed LTEs are higher. Since the LTE and differential deflections are used to evaluate the performance of joints, the LTPP geophone layout, that presented the more critical loading condition and more critical performance parameters, is selected for subsequent testing.

Table 4-1: Summary of Statistical Testing of Performance Parameters for LTPP and Dynatest® Geophone Layout.

	Min. – Max.		Sample Size	*p-value*	Statistically Similar
	LTPP	**Dynatest**	**n**		
LTE (%)	87 – 98	92 – 98	120	1.85 x 10^{-6}	No
Peak Deflection (μm)	65 – 414	65 – 415	120	9.13 x 10^{-1}	Yes
Differential Deflection (μm)	2 - 26	2 - 22	120	5.78 x 10^{-4}	No

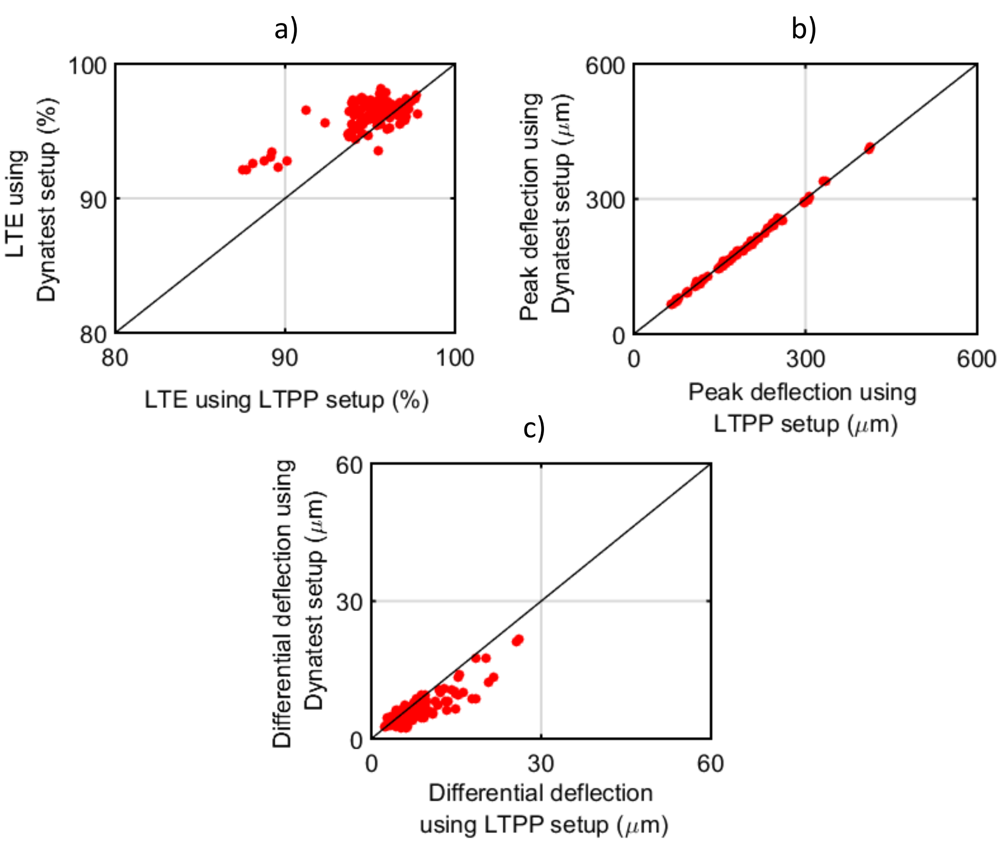


Figure 4-2: Dynatest vs. LTPP Recommended FWD Geophones Layouts a) LTE, b) Peak Deflection, and c) Differential Deflection.

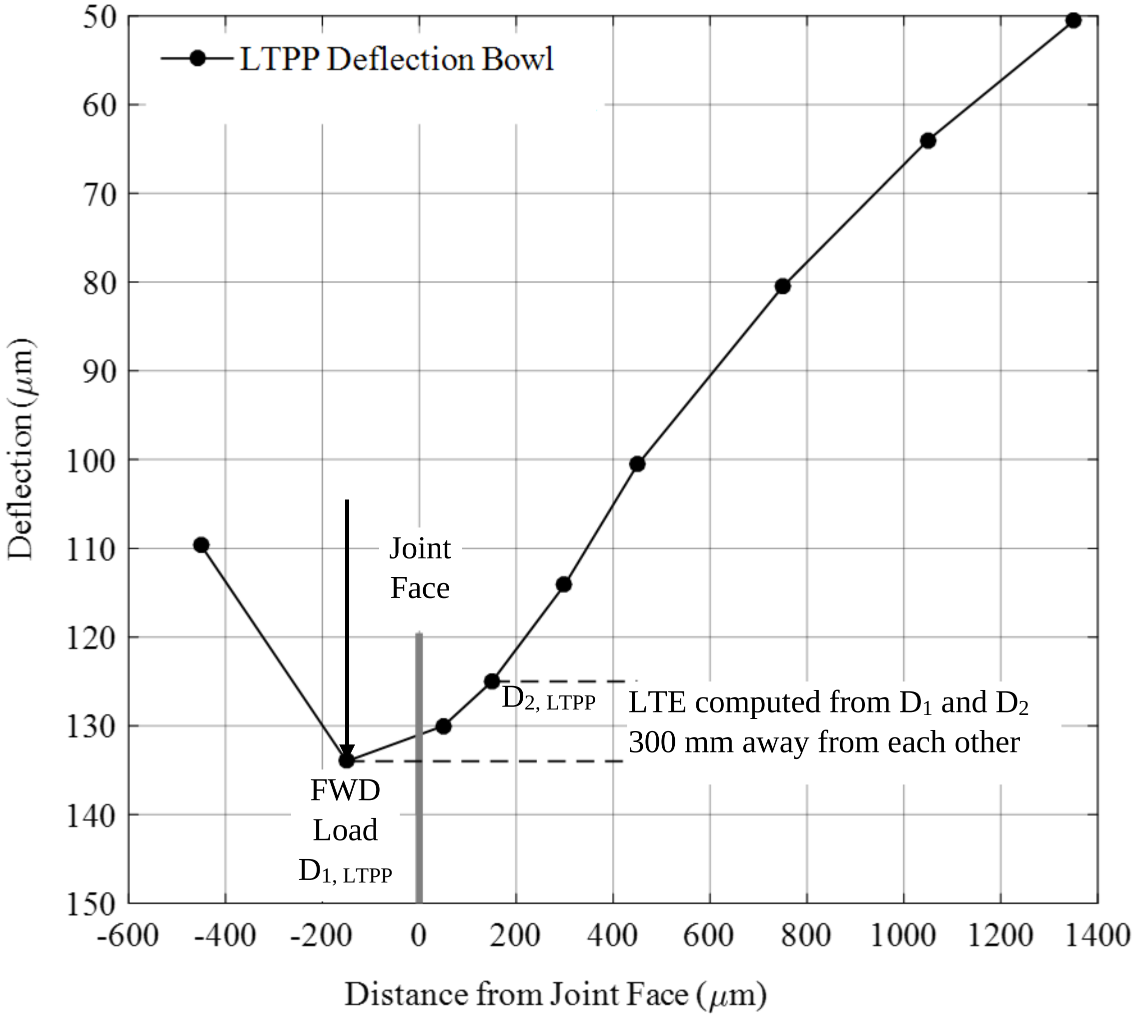


Figure 4-3: LTPP Layout Deflection Bowl at Joint.

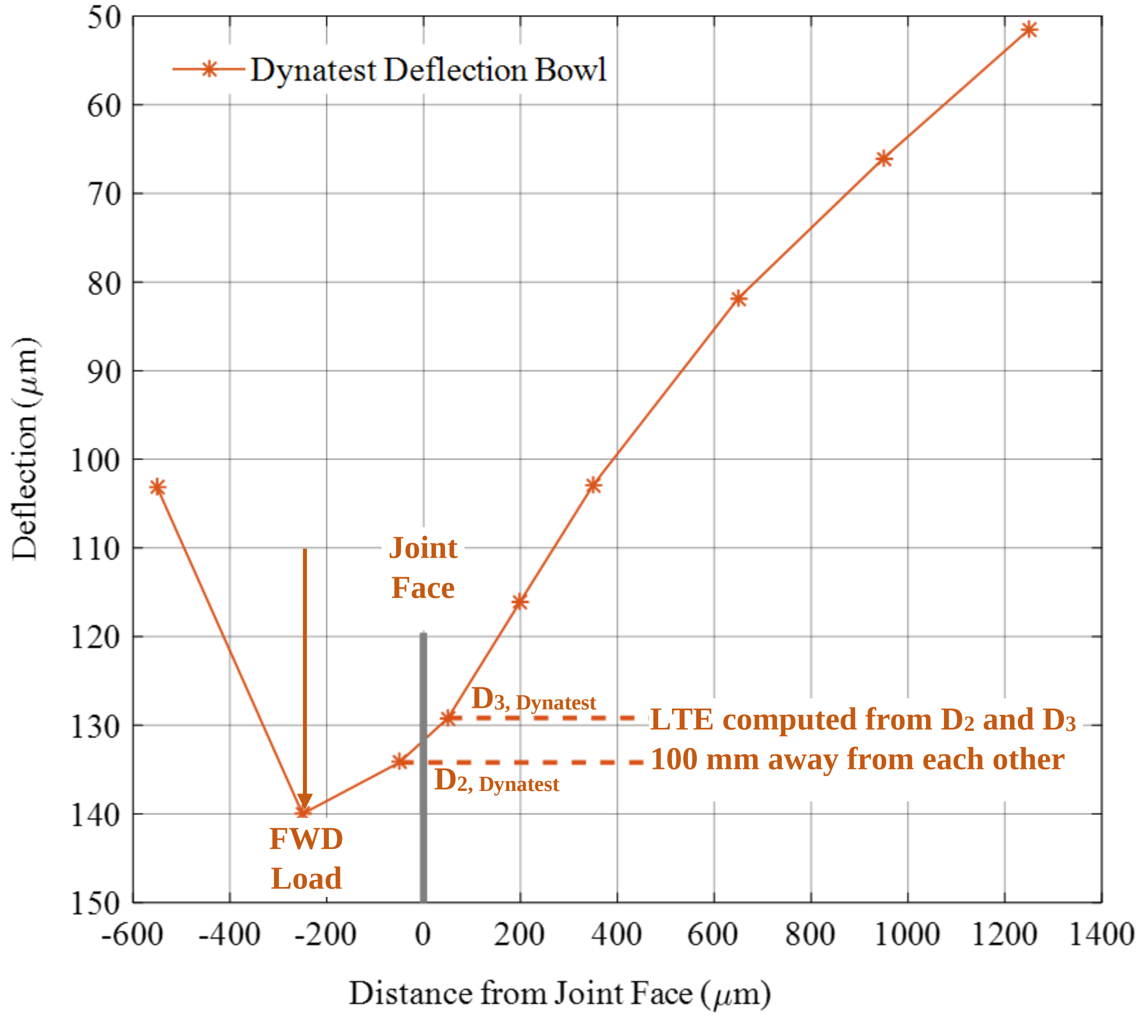


Figure 4-4: Dynatest Layout Deflection Bowl at Joint.

4.2. Threshold Values for Peak and Differential Deflections

As mentioned in Section 2.3.2, the lowest LTE for a well-performing joint ranged between 60% to 70%, while peak and differential deflections had significant variations. Therefore,a correlation between the computed LTEs from field testing and their corresponding peak and differential deflections was carried out to investigate the deflection values correspond to the LTE value of 60% and 70%. These correlations were established for as sample size, n, of 431.

Figure 4-5 shows the relationship between joint LTE values and their corresponding peak and differential deflections. In the linear regression between LTEs and deflections, 60% and 70% LTE thresholds correspond to approximately 487 μm and 400 μm peak deflections, respectively. Meanwhile, they correspond to 196 μm and 139 μm differential deflections, respectively. For deflection thresholds, the lower the threshold values that trigger a treatment, the more conservative the rehabilitation approach is. Therefore, a threshold value for differential deflection of 130 μm was used as this value is conservative as well as in line with the field performance and other agencies.

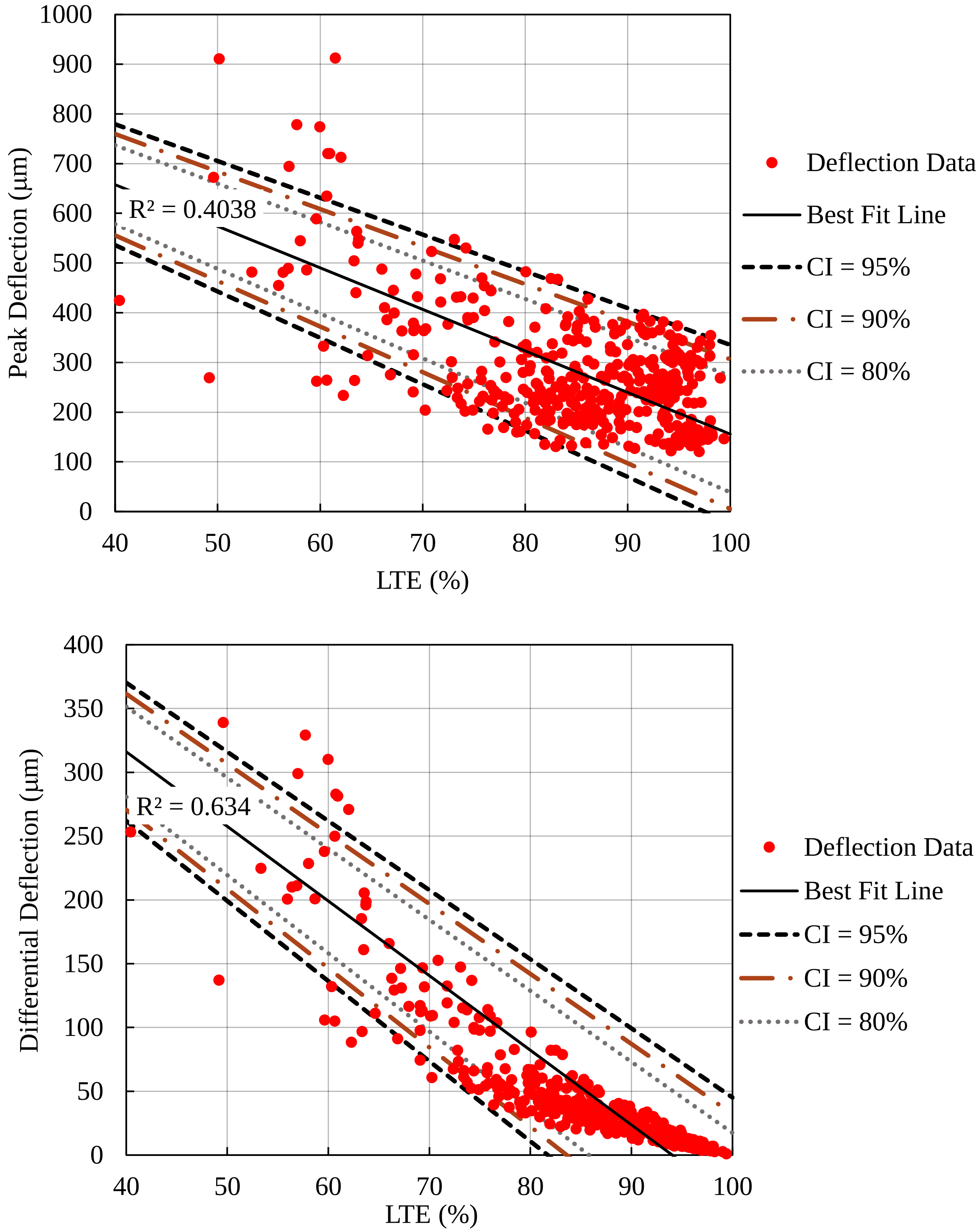


Figure 4-5: Relationship between Load Transfer Efficiency and Peak and Differential Deflections.

Peak deflections of 487 μm and 400 μm correspond to 60% and 70% LTE, respectively. The trigger value for slab stabilization for the Pennsylvania Department of Transportation is 500 μm so the observed deflection threshold in the field is comparable. Since peak deflections indicate support condition, they are anticipated to be strongly affected by the presence of voids under joints. As voids form and increase in size under joints, the slabs experience excessive deflections at the joints, which lead to pumping and cracking of the concrete. In selecting a threshold value for peak deflections, it is important to consider the correlation between peak deflections and void formation in terms of deflection intercepts. Figure 4-6 shows the relationship between deflection intercepts and peak deflections for the tested joints. As shown, larger x-intecept values, which indicate a higher probability of void existance, correspond to higher peak deflections. This is because as voids formulate under the concrete slab, the slab is allowed to deflect more under applied loads. The critical deflection intercept of 50 μm (2 mils) corresponds to a mean peak deflection of 466 μm which is very close to 487 μm that 60% LTE corresponds to. At 90% reliability (lower confidence interval), the peak deflection value is 401 μm. Differential deflection threshold values could be considered as 30% and 40% of the peak deflection value as it corresponds to 70% and 60% LTE, respectively. Differential deflections values of 120 μm and 160 μm correspond to 30% and 40% of the peak deflection value at 90 % reliability, respectively. These possible differential deflection values are in line with the conclusions from the correlation of differential deflections with LTEs. Therefore, the selected threshold value for peak deflection is recommended to range between 400 μm and 500 μm as it corresponds to threshold values of load transfer and to possible presence of voids. The selected threshold value for differential deflection is recommended to be 130 μm. Selected joint performance threshold values are summarized in Table 4-2.

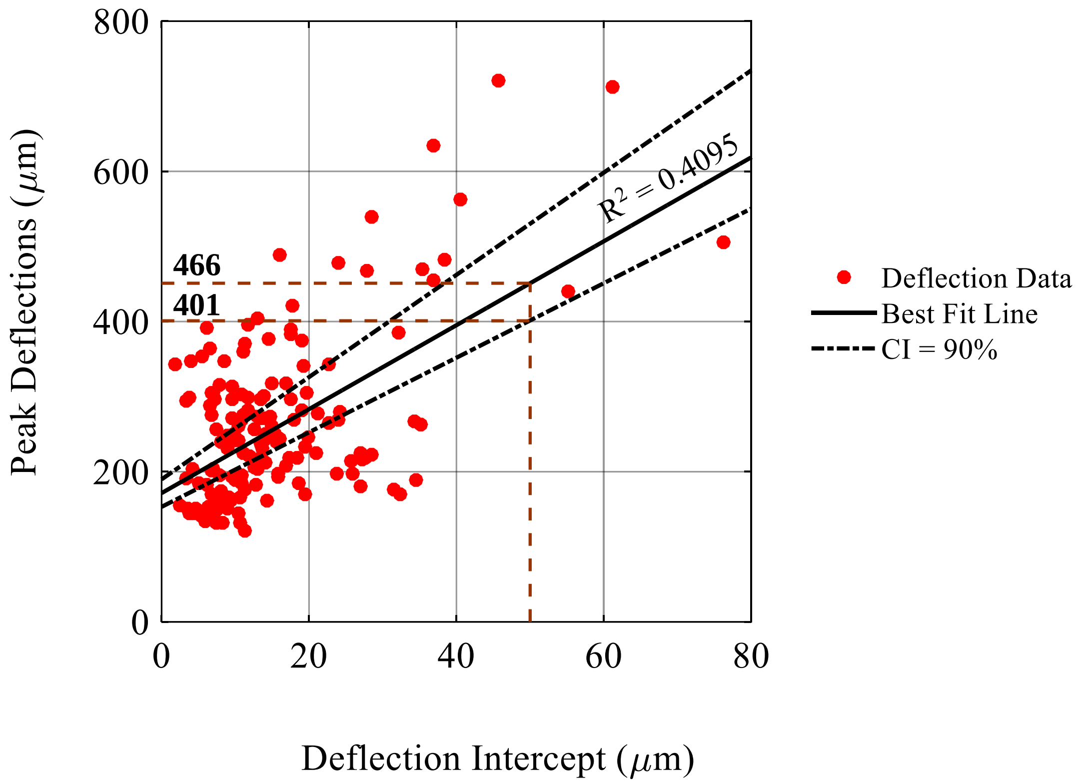


Figure 4-6: Relationship Between Deflection Intercepts and Peak Deflections.

Table 4-2: Summary of Selected Joint Performance Threshold Values at 40 kN.

Parameter	LTE (%)	Differential Deflection (μm)	Peak Deflection (μm)
Threshold	70	130	500

4.3. Statistical Selection of FWD Load Levels

Since the non-destructive testing was applied in urban areas, it was important to reduce testing time in order to minimize the disruption to traffic resulting from FWD testing. Therefore, statistical analysis of FWD results was performed to reduce the number of load levels used in each test from four load levels (25 kN, 40 kN, 55 kN and 70 kN) to two load levels. The aim was to omit FWD load levels that result in statistically similar LTE since they utilize testing time but effectively produce the same result. Moreover, the statistical analysis aims to determine the combination of two load levels that result in statistically similar deflection x-intercept values to those obtained from four load levels. This would effectively result in avoiding to compromise the reliability of void detection in the process of reducing testing load levels. FWD 40 kN load is equivalent to a standard 80-kN (18000-lbs) axle load and so it is typically used in FWD testing for most highway pavement testing (K. Smith et al., 2017). Therefore, the 40 kN load level was maintained for testing and the second load level was to be selected to produce statistically similar void detection results. The statistical test used was paired student t-test. Table 4-3 summarizes the results of the statistical t-test for LTE, wherein the matrix shows the statistical significance of LTE values obtained from each load level to all other load levels. Since statistical testing can not be performed between two sets of the same data, the diagonal of the matrix is made blank as it relates to the statistical significance of LTE of each load level. The red colour in the matrix denotes that LTE values from the two loads (in the row and column) are significantly different, while the green indicates that the LTE values are significantly similar. Therefore, LTE values obtained from 40 kN and 55 kN are statistically similar to those obtained from the 70 kN load level. Table 4-4 shows the statistics of the LTEs obtained from different load levels. Although the statistics may not strongly reflect similarlity or differences, a paired t-test is more likely to reflect these changes as it evaluates the mean of the differences of each sample. LTE statistics were found to be minimally affected by the load level with the minimum, maximum and mean of LTEs at 25 kN being almost the same for 40 kN. The minimum LTE computed decreased as the load applied increased. This could be as lower loads do not mobilize and activate the load transfer mechanism to its fullest.

Table 4-3: LTE Statistical Testing Results (*p-value*) of Four Load Levels.

Sample Size, n = 962				
Load Level (kN)	**25**	**40**	**55**	**70**
25	-	3.49×10^{-6}	7.17×10^{-11}	1.30×10^{-4}
40	3.49×10^{-6}	-	7.07×10^{-3}	5.72×10^{-2}
55	7.17×10^{-11}	7.07×10^{-3}	-	3.09×10^{-1}
70	1.30×10^{-4}	5.72×10^{-2}	3.09×10^{-1}	-

* Red indicates *p-value* is < 0.05 and the means are significantly different.
Green indicates *p-value* is > 0.05 and the means are not significantly different.

Table 4-4: Statistics of LTEs at Different Load Levels.

Load Level (kN)	**Minimum (%)**	**Maximum (%)**	**Mean (%)**	**Standard Deviation (%)**
25	36	98	84	13
40	38	99	85	12
55	26	100	85	12
70	14	100	85	13

Since LTE is a ratio, it could be unaffected by deflections variation resulting from the applied load as was found in the statistics of Table 4-4. This is especially the case for higher load levels, such as the statistical significance found between 40 kN and 55 kN with 70 kN. At higher loads, the concrete slab would be experiencing high deflections which lead to the full mobilization of the load transfer mechanism, resulting in statistically similar LTE values. However, the void detection methodology does not rely on full activation of the load transfer mechanism, but rather on the progressive increase of applied loads and experienced deflections even with partial mobilization of the load transfer mechanism. Figure 4-7 shows a typical variation of LTEs with the four load levels. It can be seen that LTE values, being a ratio, are not significantly affected by the load level applied. Peak deflections are fundamentally expected to increase with the magnitude of the applied load as higher contact stress results in higher deflections of the slab edge as shown in Table 4-5. However, the ratio of loads to one another is similar to the ratio of the average deflections obtained from these loads as shown in Table 4-6. Since differential deflections are the absolute difference of deflection values, they increase with higher loads similarly. Therefore, it is concluded that the results of the LTE statistical testing cannot be used to select two load levels as the LTE statistcs do not show considerable variation under different load levels.

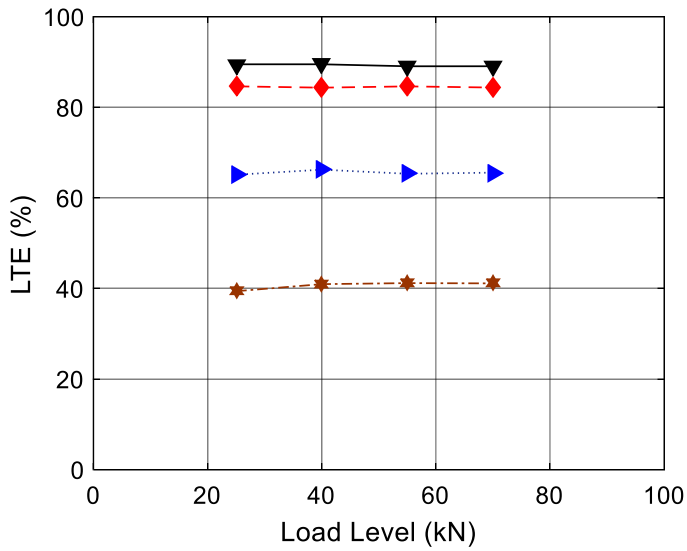


Figure 4-7: LTEs Variation with Load Levels.

In order to reduce the number of load levels during FWD testing, statistical analysis was completed to determine the statistical similarity of deflection intercept values for void detection between the four load levels and a combination of two load levels. The void detection methodology recommends having 40 kN within the range of load levels applied. Therefore, Table 4-7 shows the statistical testing completed by coupling 40 kN with other load levels to determine if their deflection intercepts were statistically similar to those obtained from four load levels. The combination of 25 kN and 40 kN loads offer statistically similar deflection intercepts as the four load levels for void detection and provide statistically different LTEs so that FWD testing time is not spent to obtain statistically similar values. Therefore, the combination of 25 kN and 40 kN loads are recommended to be used instead of four load levels (25 kN, 40 kN, 55 kN, and 70 kN) to optimize FWD testing time and cost.

Table 4-5: Statistics of Peak Deflections at Different Load Levels.

Load Level (kN)	Minimum (μm)	Maximum (μm)	Mean (μm)	Standard Deviation (μm)
25	49	745	128	68
40	82	1134	202	106
55	111	1512	273	147
70	141	1800	346	213

Table 4-6: Load Ratios and Mean Peak Deflection Ratios.

Loads	Load Ratio	Mean Deflection Ratio
25 kN / 40 kN	0.6	0.6
25 kN / 55 kN	0.5	0.5
25 kN / 70 kN	0.4	0.4
40 kN / 55 kN	0.7	0.7
40 kN / 70 kN	0.6	0.6
55 kN / 70 kN	0.8	0.8

Table 4-7: Results of Statistical Testing (*p-value*) for Deflection Intercepts Obtained from Four Load Levels and Different Combinations of Two Load Levels.

T-test Results between Four Load Levels with	***p-value***	Statistically Similar
25 kN and 40 kN	0.91	Yes
40 kN and 55 kN	0.01	No
40 kN and 70 kN	< 0.05	No

4.4. FWD Testing in the Presence of Asphalt Cement Overlays

Joints deflection testing was performed on the approach and leave sides of a composite urban arterial section using the LTPP layout selected in Section 44.1. The FWD testing on Pembina Highway was conducted on the asphalt overlay of the PCC and then on the PCC layer after the overlay was milled. Reflection cracks were used to determine the location of joints prior to milling and real-time kinematic (RTK) survey equipment was used to capture test location coordinates for retesting after milling of the asphalt overlay. Figure 4-8 shows the same joint being testing before and after milling of the asphalt overlay.

1) Prior to Milling

2) After Milling

Figure 4-8: FWD Test 1) Prior and 2) After Milling.

4.4.1. Effect of Asphalt Overlay on Joint Performance Parameters

FWD joint testing was carried out to evaluate the effect of the overlay on the measured deflections and computed load transfer efficiencies and differential deflections. Table 4-8 shows the results of statistical t-tests for LTE, peak deflections and differential deflections of joints prior to and after milling at 40 kN. The sample size, n, is 640 for all load levels and 160 for 40 kN.

The results show that these parameters are all statistically different prior to and after milling. Therefore, it is difficult to correctly predict the performance of joints when pavement characterization is conducted prior to milling as the recorded deflections would be influenced by the presence of the overlay. In addition to asphalt compression, the difference in deflections might be attributed to the way the asphalt overlay distributes forces and stresses across its depth to the surface of the concrete structure, as shown in Figure 4-9. The load applied on the asphalt overlay gets distributed across its thickness so that that the concrete surface receives a lower force and produces a lower deflection. The distributed stress also induces force on the unloaded side of the joint resulting in inaccurate deflections and LTEs. Figure 4-10 shows the probability distribution of measured deflections before and after milling. Overlay milling produces larger joint deflections whereby the mean deflection increases from 150 μm to 200 μm after asphalt overlay milling. This increase in deflections is the result of the load being applied directly on the concrete slab without the overlay resulting in higher contact stress on the concrete surface after milling.

Table 4-8: Statistical Test Results of Joint Performance Parameters Prior to and After Asphalt Overlay Milling at 40 kN.

		Mean	***p-value***	**Statistically Similar**
LTE (%)	Before Milling	86	7.77×10^{-9}	No
	After Milling	79		
Peak Deflection (μm)	Before Milling	152	1.19×10^{-7}	No
	After Milling	207		
Differential Deflection (μm)	Before Milling	21	9.68×10^{-6}	No
	After Milling	34		

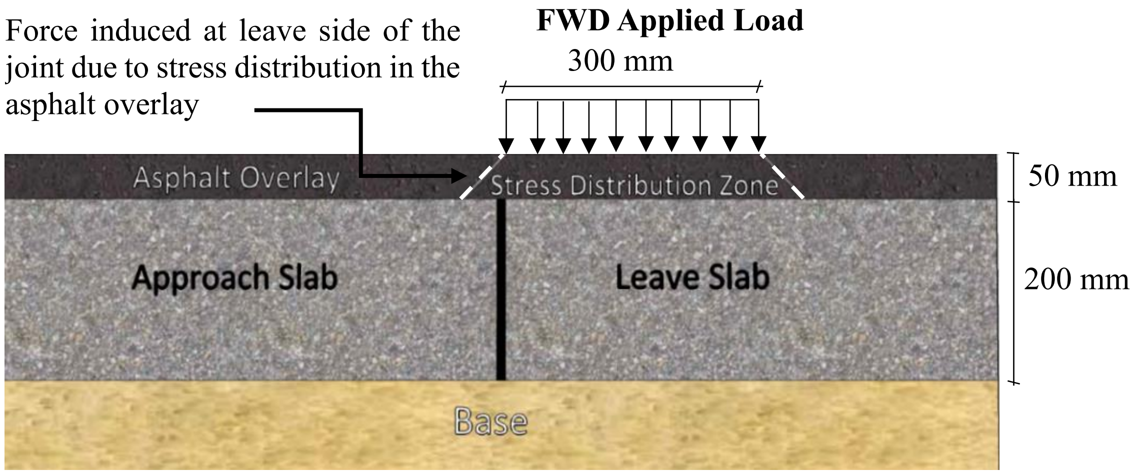


Figure 4-9: Stress Distribution Across Asphalt Overlay Causing Overestimated Joint Performance.

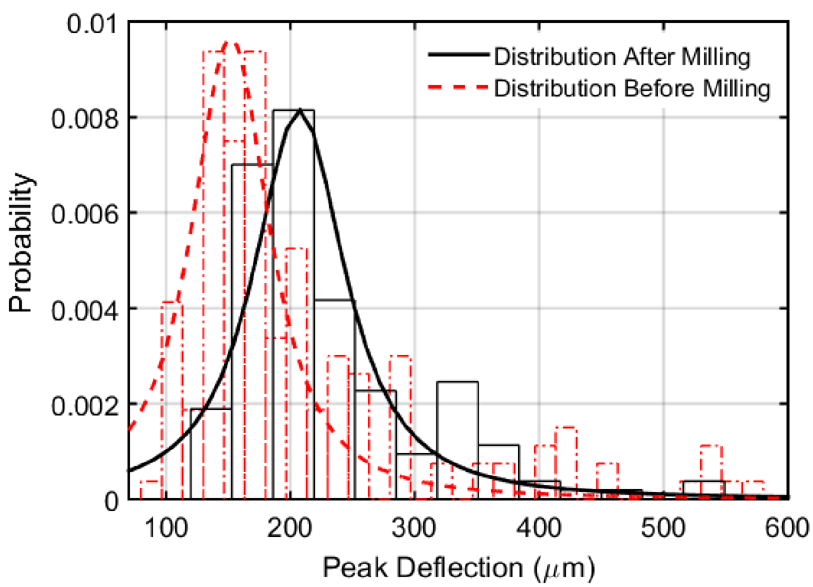


Figure 4-10: Probability Distribution of Peak Deflections Before and After Milling.

A similar trend is observed in differential deflections, as shown in Figure 4-11, whereby the mean differential deflection increased from 20 μm to 35 μm when the asphalt overlay was milled. This effect can also be seen in Figure 4-12 wherein the computed LTE reduces after the milling of the asphalt overlay from a mean of 85% to 78%. This reduction is due to the absence of the asphalt overlay and its stress distribution to the unloaded side of the joint producing lower deflections on the unloaded side, as explained in Figure 4-9.

When joints are tested prior to the commencement of rehabilitation for planning, bidding, and selection of appropriate repair strategy, the performance of joints may be overestimated when deflection testing is conducted in the presence of the asphalt overlay. The effect of the asphalt overlay on peak deflections is also expected to influence the results of void detection analysis which is performed using peak deflections. Therefore, it is important to be able to account for the presence of the asphalt overlay and correct deflections measured prior to its milling.

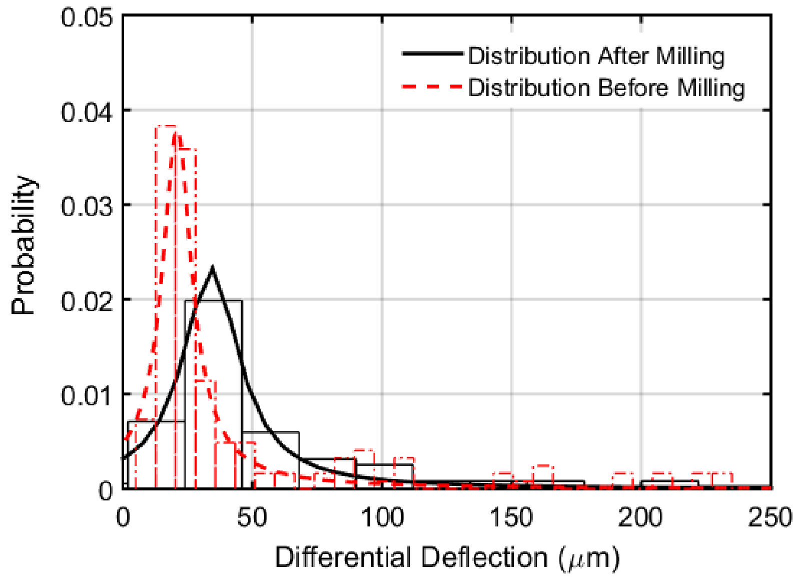


Figure 4-11: Probability Distribution of Differential Deflections Before and After Milling.

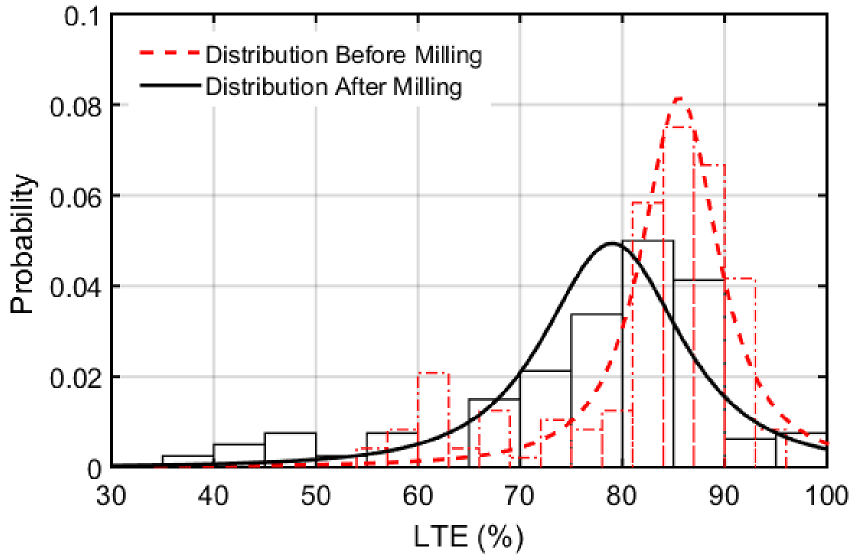


Figure 4-12: Probability Distribution of LTEs Before and After Milling.

4.4.2. Effect of Asphalt Overlay on Void Detection

Since the asphalt overlay affects the measured deflections at joints as shown in Figure 4-10, they are anticipated to affect deflection x-intercept values obtained in void detection analysis. When deflection testing is performed on asphalt overlays, the lower deflections recorded are expected to cause lower deflection x-intercepts and, consequently, prevent locating potential voids under the concrete slab. Figure 4-13 demonstrates how the load-deflection plot of the same joint shifts to the right after milling as deflections increase for the same applied loads. This, in turn, would affect the calculated deflection x-intercepts as part of the void detection analysis and may influence rehabilitation decisions.

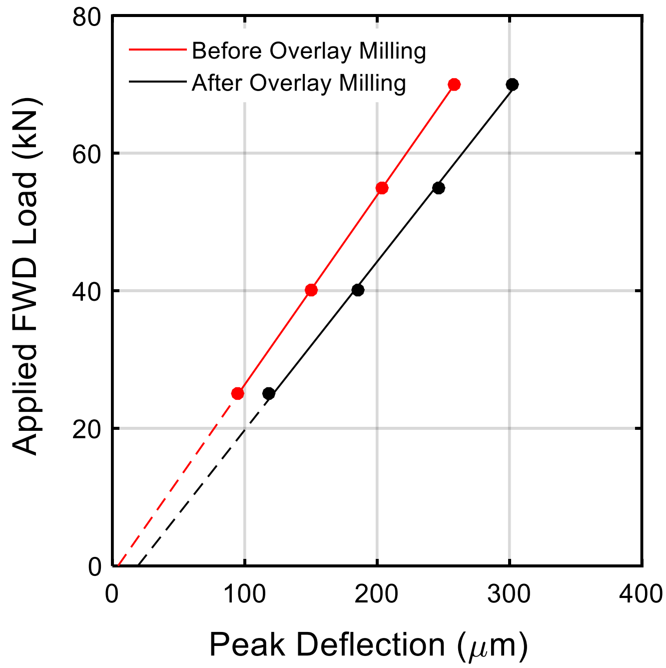


Figure 4-13: Joint Deflection-Load Plot Before and After Milling.

Void detection analysis was performed with deflection data obtained prior to and after the milling of the asphalt overlay. Figure 4-14 shows that there is no correlation between the outcome of the void detection analysis, in terms of deflection x-intercepts, using deflection tests before and after milling. Therefore, when a joint is tested before milling the asphalt overlay, void detection analysis is not recommended to be performed using raw deflection values without accounting for the presence of the asphalt overlay.

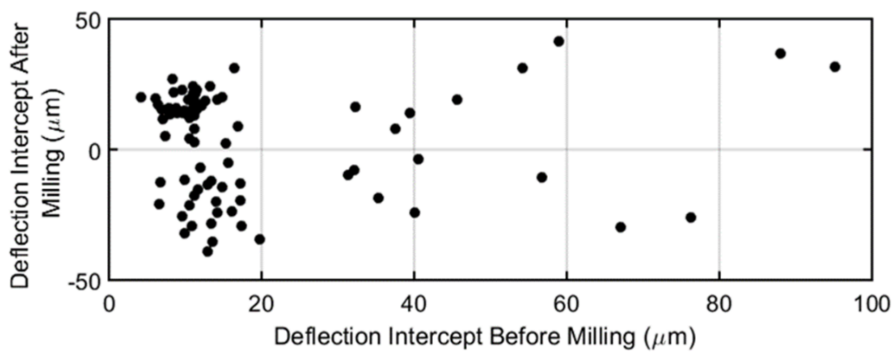


Figure 4-14: Void Detection Analysis Using Deflections Before and After Milling.

4.4.3. Correlating Deflections Before and After Milling

Section 4.4.1 and 4.4.2 demonstrated changes in performance parameters of concrete after milling of the asphalt overlay due to changes in the recorded deflection from that recorded prior to milling. To account for the presence of the asphalt overlay, deflections prior to milling need to be corrected to estimate deflections after milling to compute joints performance more reliably.

Peak deflections are used in computing the three performance parameters as part of joints evaluation as well as void detection analysis. Moreover, deflections 300 mm away from the load application, D_{300}, is also used in determining LTE and differential deflections. D_{300} is the deflection measurement on the unloaded side of the joint. Therefore, relationships between peak deflections (D_0) and deflections 300 mm away from the load plate (D_{300}) prior to and after milling were established as shown in Figure 4-15 and Figure 4-16.

Table 4-9 and Table 4-10 summarize the results of these correlations. Deflections after milling are estimated using a correction factor accounting for the asphalt layer, $F_{Asphalt}$, from deflections obtained prior to milling using the following equation

$$D_{after\ milling} = F_{Asphalt} \cdot D_{before\ milling}$$

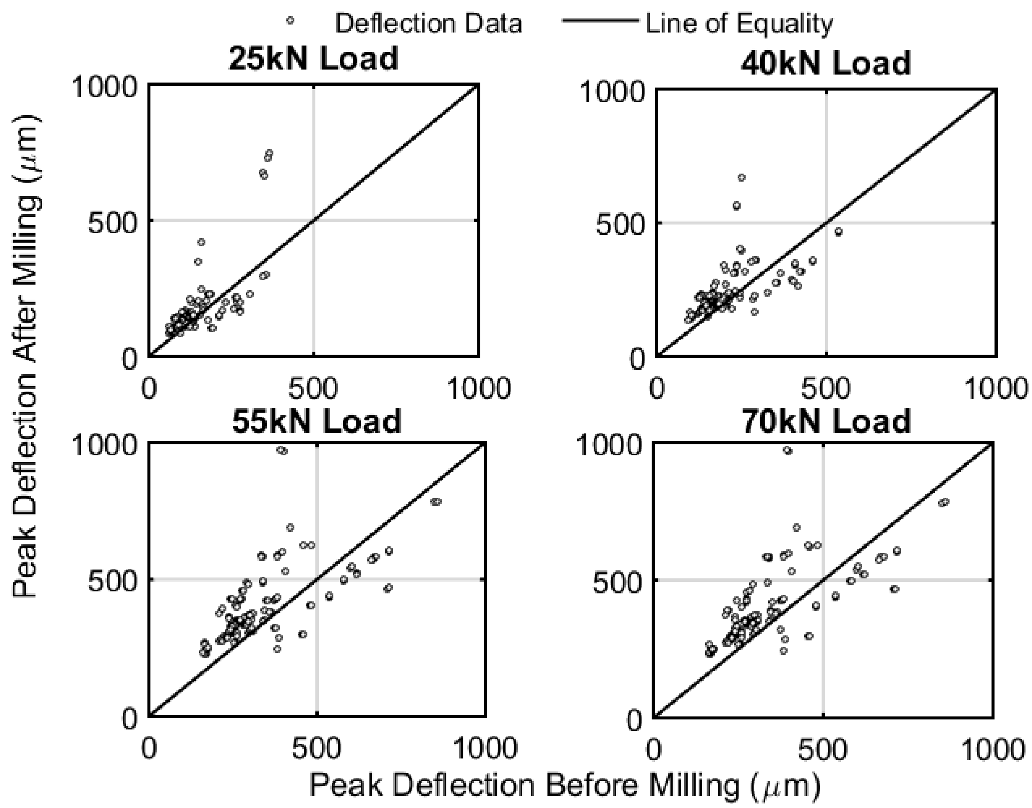


Figure 4-15: Relationship Between Peak Deflections Before and After Milling at Different Loads.

Table 4-9: Correlation Summary of Peak Deflections Before and After Milling.

Load Level (kN)	$\mathbf{F_{Asphalt}}$	$\mathbf{R^2}$
25	1.175	0.496
40	1.204	0.508
55	1.117	0.515
70	1.266	0.556

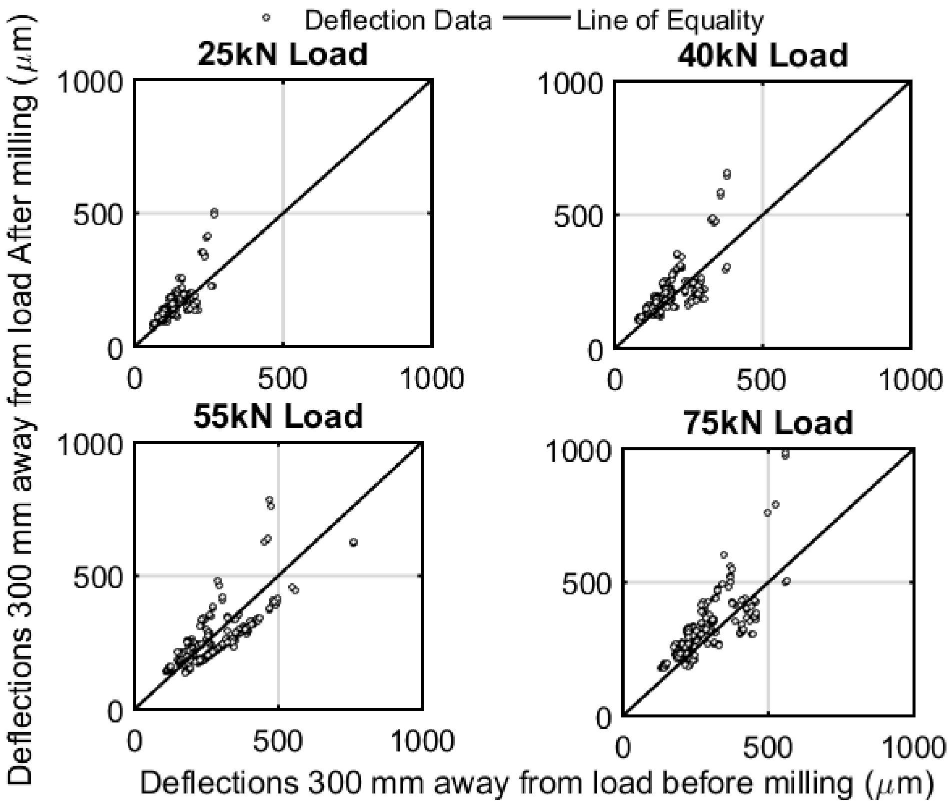


Figure 4-16: Relationship between Deflections 300 mm Away from Load Plate Before and After Milling at Different Loads.

Table 4-10: Correlation Summary of D_{300} Deflections Before and After Milling.

Load Level (kN)	**$F_{Asphalt}$**	**R^2**
25	1.166	0.621
40	1.103	0.563
55	1.108	0.627
70	1.166	0.625

Table 4-9 and Table 4-10 show that deflections may increase by up to 27% after milling, and such a high increase will affect measured LTEs and the decisions based on these readings. Figure 4-15 and Figure 4-16 show that deflections prior to and after milling correlate well wherein the least R^2 value is approximately 0.5. This indicates a marginal correlation between PCC deflections measured in the presence and absence of an asphalt overlay. Since the magnitude of deflections is affected by the magnitude of load applied, the correlations were established for different load levels. The $F_{Asphalt}$ factors determined for each load level could potentially be combined into one factor for all loads levels. However, this is currently not suggested until further verification is conducted. It is worthy to note that the intercept values for all relationships were set to zero to establish a rational relationship between the two deflections. The relationships presented may vary with asphalt thickness, mix designs, asphalt overlay temperature and temperature gradient of the PCC at the time of testing. Factors affecting the deflections relationship explain why the R^2 values of these correlations were limited to a maximum of 0.63. The application of the correction factor, $F_{Asphalt}$, will be shown in the following section.

4.4.4. Corrected Joint Performance Parameters

To measure the load transfer efficiency of joints more reliably, deflection values before asphalt overlay milling should be corrected to account for changes in deflection incurred by the milling of the asphalt overlay. Correlation between the two deflection values for each joint at every load level was performed so that the deflection values after milling can be predicted from those prior to milling.

Figure 4-17 shows the distributions of corrected peak deflections prior to milling and peak deflections after milling of the asphalt overlay. In comparison with the distributions shown in Figure 4-10, correcting deflections obtained prior to milling provides a good estimate of deflections after milling. This, in turn, provides a more reliable tool to evaluate deflections and LTEs to improve the decision-making process in rehabilitation planning. Similarly, Figure 4-18 shows an improvement in differential deflection distribution compared to Figure 4-11, wherein accounting for the presence of the asphalt layer results in deflection values that better match those obtained from testing after milling of the asphalt overlay.

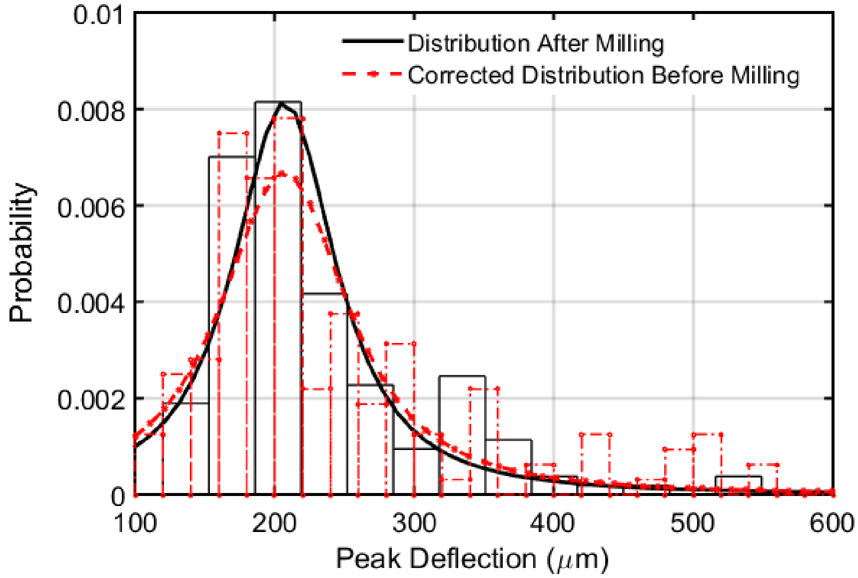


Figure 4-17: Probability Distribution of Corrected Peak Deflections Before Milling and Peak Deflections After Milling.

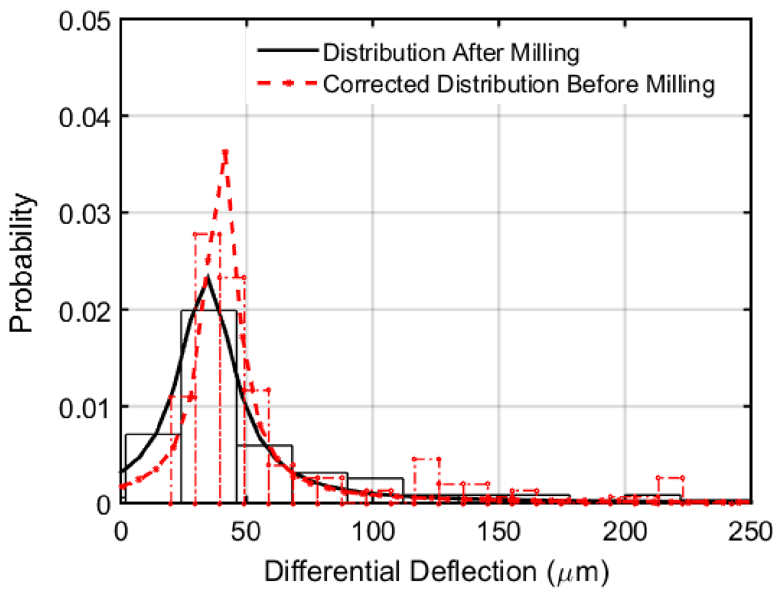


Figure 4-18: Probability Distribution of Corrected Differential Deflections Before Milling and Differential Deflections After Milling.

Figure 4-19 shows LTE distribution obtained from corrected deflections prior to milling and LTE distribution after milling. Accounting for the presence of the asphalt overlay has improved LTE predictions when deflection testing is performed prior to the milling of the overlay. Although accounting for the asphalt overlay has reduced the mean LTE readings to match those after milling, there was some variation in the frequency of LTE values in the 40 % to 60 %.

Table 4-11 shows the results of statistical t-tests for LTE, peak deflections and differential deflections obtained from corrected deflections prior to the milling of the asphalt overlay and from deflections obtained after milling. Peak deflections and differential deflections are statistically similar after the asphalt overlay has been accounted for, as opposed to Table 4-8. For LTE, the statistical test shows that they are still statistically different. This is believed to be due to the difficulty in estimating the frequency of LTE values at the extreme ends of the distribution, within 40% and 70%. However, correcting for the presence of the asphalt overlay has overall significantly improved measured peak deflections, differential deflections, and LTEs to closely reflect these measured after milling the asphalt overlay. This is expected to increase the reliability of FWD testing prior to overlay milling to undertake rehabilitation decisions.

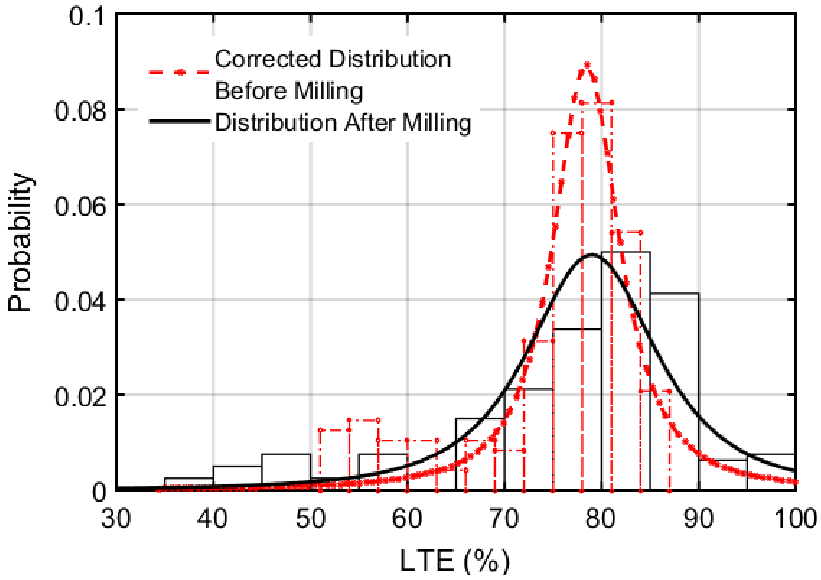


Figure 4-19: Probability Distribution of Corrected LTE Before Milling and After Milling.

Table 4-11. Statistical Test Results of Corrected Performance Parameters Prior to Milling and After Milling at 40 kN.

		Mean	***p-value***	**Statistically Similar**
LTE (%)	Corrected Before Milling	86	0.0134	No
	After Milling	78		
Peak Deflection (μm)	Corrected Before Milling	207	0.6934	Yes
	After Milling	207		
Differential Deflection (μm)	Corrected Before Milling	40	0.894	Yes
	After Milling	34		

4.4.5. Corrected Void Detection Results

Void detection analysis depends on peak deflections measured on the surface of the concrete slab. Since asphalt overlays result in smaller peak deflections, void detection analysis was found, Figure 4-14, to not identify potential voids correctly if FWD testing was done on the overlay surface. Deflections prior to milling were corrected using the determined coefficients in Table 4-9 to predict peak deflections after milling of the asphalt overlay so that void detection analysis results can be more reliable. Figure 4-20 shows the corrected deflection-load plot of the same joint shown in Figure 4-13. After correction, the deflections of the joint prior to milling are closer to those obtained after milling, and they can be used in the void detection analysis as they result in closer deflection intercepts. The deflection intercept was corrected from 4 μm to 17 μm compared to the 19 μm intercept calculated from deflections after milling.

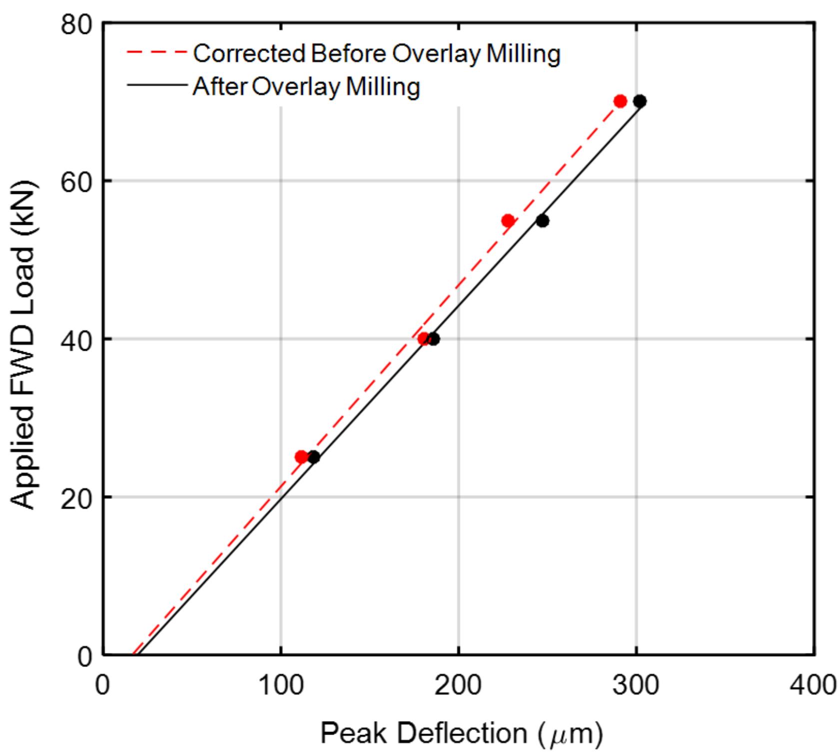


Figure 4-20: Corrected Joint Deflection-Load Plot Before and After Milling.

Figure 4-21 compares the deflection intercepts obtained from using the corrected pre-milling deflections and post-milling deflections. It is clear that the results of void detection analysis using the corrected pre-milling deflections correlate better with those from post-milling deflections when compared to Figure 4-14 in which void detection analysis was performed on deflection data collected prior to milling without further processing. This analysis highlights a need for processing deflection data obtained by joint testing on asphalt overlays as well as a need for a more comprehensive void detection method that takes the overlay into account since many urban PCC pavements are now overlaid.

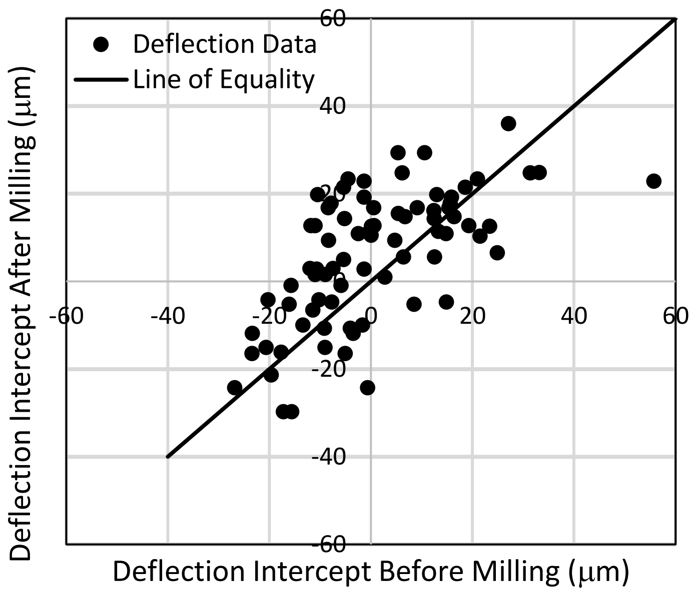


Figure 4-21: Comparison of Void Detection Analysis Results from Corrected Pre-Milling Deflections and Post-Milling Deflections.

Chapter 5

CASE STUDY: FULL-DEPTH REPAIRS PERFORMANCE AND INCORPORATING PERFORMANCE AT DESIGN STAGE

Part of this study aimed at evaluating the joint performance, in terms of LTE, peak deflection, and differential deflections, achieved by Full-depth repairs (FDRs). Moreover, the study examined the benefits of incorporating FWD testing and joint performance at the design stage of roadway rehabilitation projects.

FWD deflections and LTEs were utilized to examine the performance of FDRs in restoring the load transfer capacity of joints and evaluating the benefits of incorporating NDT at the design stage in terms of decision-making.

5.1. Performance of Full-Depth Repairs

FDRs at joints are considered to be one of the most common rehabilitation strategies applied to rigid pavements since joint performance has a significant impact on pavement performance (Snyder & Darter, 1990). When FDRs are performed properly, they can result in improved roadway smoothness, pavement structural integrity, and overall service life (ACPA, 1995).

The performance of joints prior to FDRs are not presented as the aim is to investigate whether FDRs are providing good load transfer capacity. Moreover, FDRs result in two new joints on either side of the old deteriorated joint and so comparing their load transfer capabilities would not be comparing the performance of the same joint. Figure 5-1 summarises the computed LTE, peak deflections and differential deflections for the approaches and leaves sides of 49 joints receiving FDR.

Six joints on the approach and 10 joints on the leave side of the joints, shown in Figure 5-1, have load transfer efficiency lower than the threshold value of 70%. However, none of the joints receiving FDR exhibited peak deflections higher than the 500 μm, indicating good support condition upon constructing the FDRs. One joint with 42% LTE experienced differential deflections of 135 μm and 132 μm and peak deflections of 233 μm and 228 μm for the approach and leave sides, respectively. This joint indicates good support condition but possibly loose dowels and is expected to deteriorate in a short period of time due to the lack of load transfer capacity. Fifty-seven percent (57%) of joints receiving FDRs had LTE values over 80% and the average peak deflection for these joints was 155 μm while their average differential deflection was 16 μm. Joints not meeting LTE criteria had an average peak and differential deflections of 212 μm and 88 μm, respectively. It is clear that there is a correlation between LTE values computed and differential deflections. This is anticipated as differential deflection is typically computed to provide information about dowel looseness which would affect the load transfer efficiency of a joint. Moreover, these results highlight the significance of having a peak and differential deflections as parameters to evaluate joints performance along with LTE.

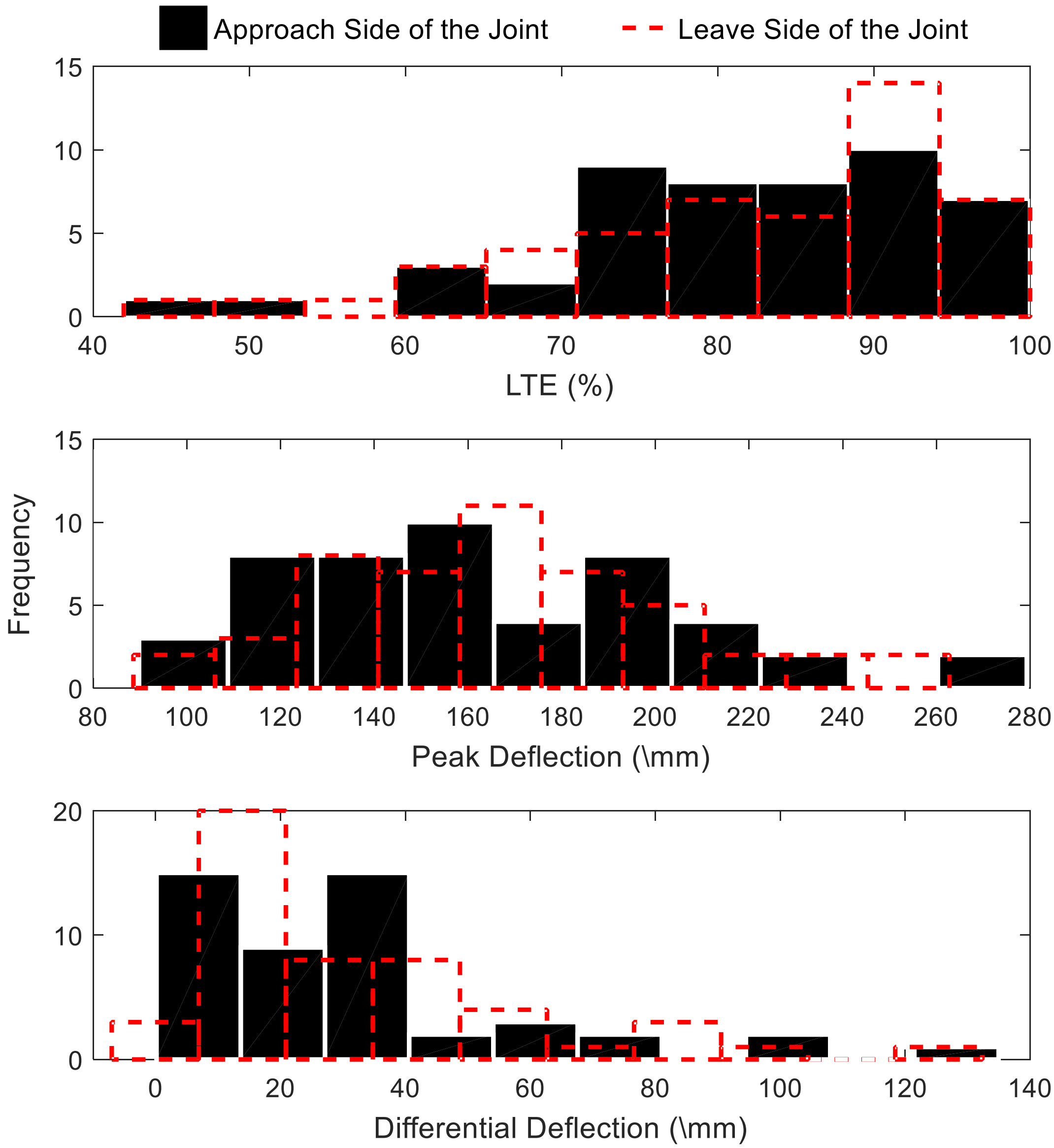
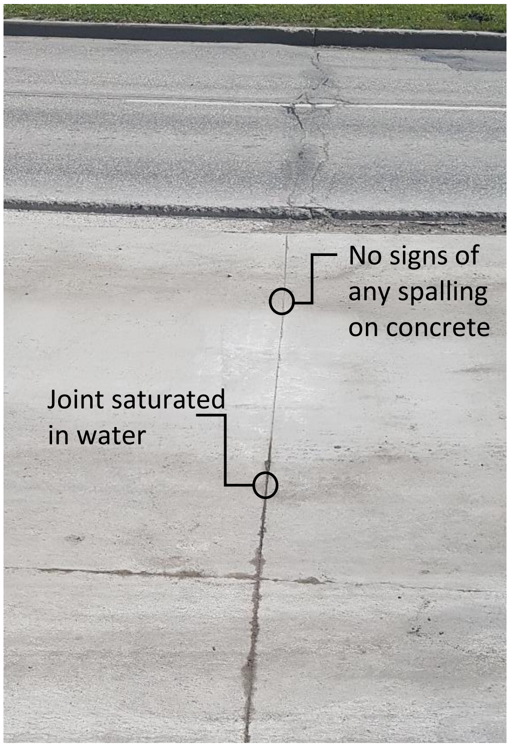


Figure 5-1: Histograms of Load Transfer Efficiency, Peak Deflection, and Differential Deflection of Joints After Full-Depth Repair.

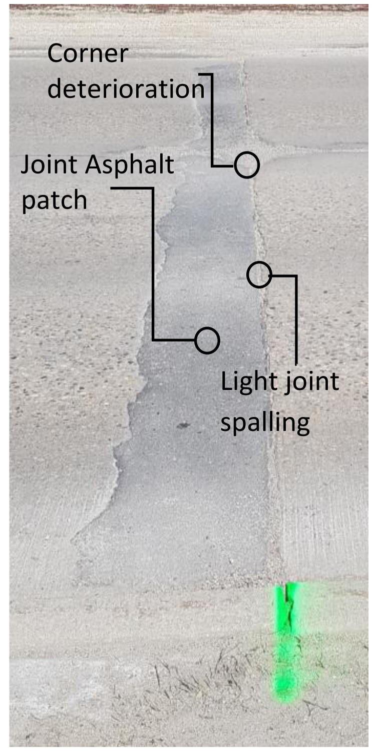
The variation in FDRs performance might be attributed to construction method and quality. Differential deflections were generally very low, indicating similar deflection levels across the joint and well-anchored dowels. Only a few joints showed high differential deflections. It is believed that this is because the grout injected in the dowel hole may have oozed out of the hole or that not enough time was given for the grout to cure. Furthermore, for joints with LTE less than 70%, their peak deflections were comparable to those with LTE values over 80%. This indicates that, generally, support condition may not be as alarming as dowel looseness and grouting for new FDRs. The joints with higher peak and differential deflections are expected to deteriorate earlier than the other joints and call for maintenance work in the near future. Only two joints had a peak and differential deflections higher than 250μm, indicating weaker support at these two joints than the other joints. FDRs have displayed their capability to achieve load transfer efficiency and low peak and differential deflections across joints.

5.2. Incorporating Performance at Design Stage

Visual inspection is the predominant state-of-practice in deciding the areas and types of repairs in rehabilitation projects. However, the performance of joints may prove hard to predict in the field with surface distresses or patches. Figure 5-2 below shows an example of two joints with the first having 65% LTE and 280 μm and 99 μm peak and differential deflections, respectively. The second joint, Figure 5-2b, has 95% LTE, 235 μm, and 11 μm peak and differential deflections, respectively. It appears that the first joint is experiencing lower performance than the second even if it may appear unpatched and well saw-cut. With visual inspection as the only decision-making tool, both joints may have received full-depth repair, either one or none – potentially incurring the cost of an additional FDR or missing the chance to repair the low-performing joint which triggers early deterioration in the rehabilitated structure. With the availability of FWD joint information, the joint in Figure 5-2a would have received an FDR while that in Figure 5-2b would have either received no treatment or a partial-depth repair if seen warranted by the project engineer. This would result in an optimized rehabilitation with optimized budget allocation to apply treatments in the identified areas.



a) LTE = 95%



b) LTE = 65%

Figure 5-2: Joints with Different Visual Condition and a) LTE = 95% and b) LTE = 65%.

To optimize the project cost, the budget should be allocated appropriately. FWD performance information can supplement visual inspection in the decision-making process to improve the selection of the type of repair for better use of the available rehabilitation budget. In this study, FWD testing was carried out concurrently with the rehabilitation work to investigate the effectiveness of the current visual decision-making process. Table 5-1 summarises the number of joints tested for two projects, the number of joints not meeting the set performance criteria, the number of joints receiving FDRs, and the success rate of applying the FDRs where they are needed based on the selected criteria.

Table 5-1: Joint Performance Test Summary and Their Respective Visual-Based Decisions.

Tested Lane	Number of Joints Tested	Number of Joints Not Meeting Performance Criteria	Number of Joints Receiving Full-Depth Repair	Number of Full-Depth Repairs Applied Where Performance Criteria Were Not Met
Median	40	4	9	4
Center	25	2	3	0
Curb	48	8	12	1
Total	**113**	**11**	**24**	**5**

Out of 113 tested joints in the two projects, only 11 (10%) did not meet the set performance thresholds for LTE, peak and differential deflections. A total of 24 FDRs was carried out within the tested sections only 4 joints met the set criteria. This means that the cost of 20 FDRs could have been reduced, or eliminated, and optimized with the availability of joint performance information. Moreover, the 10 joints which required joint performance improvement but received none will continue to deteriorate to cause further damage to the existing concrete and the newly laid asphalt layer, leading to the reduction of the rehabilitation's lifespan.

Performance information can be used not only to help decide which joints receive treatment, but also the type of treatment. Table 5-2 shows the performance parameters for the joints which do not meet the set criteria. It can be noted that joints 2, 5, 8 and 10 have very weak support compared to joints 1,3 and 6. Therefore, they would be ideal candidates for FDR. Joint 11 has high differential deflection and so it may have loose dowels or concrete cracking under the dowel, and so dowel bar retrofit may be the selected treatment. Joint 7, on the other hand, has met the set criteria and no action is recommended for it.

The current visual inspection method provides inputs related to surface distresses and drainage conditions, but it should be supplemented with performance information about the load transfer capacity of joints, dowel looseness and support condition, when possible. With the availability of such information at the design stage, improved construction planning and allocated budget can be achieved.

Table 5-2: LTE, Peak and Differential Deflections of Joints Not Meeting Set Performance Criteria.

Joint	LTE (%)	Peak Deflection (μm) at 40 kN	Differential Deflection (μm) at 40 kN	Potential Repair Decision
1	67.31	285.59	93.36	Dowel Bar Retrofit
2	60.10	566.67	226.10	Full-Depth Repair
3	68.81	280.53	87.50	Dowel Bar Retrofit
4	62.11	423.02	160.30	Full-Depth Repair
5	57.90	457.82	192.75	Full-Depth Repair
6	58.30	259.19	108.07	Dowel Bar Retrofit
7	70.56	355.52	104.65	No Action
8	62.27	536.52	202.43	Full-Depth Repair
9	64.07	401.69	144.33	Full-Depth Repair
10	61.66	472.10	181.00	Full-Depth Repair
11	62.35	338.18	127.33	Dowel Bar Retrofit

Chapter 6

RESIDENTIAL STREETS FWD TESTING RESULTS

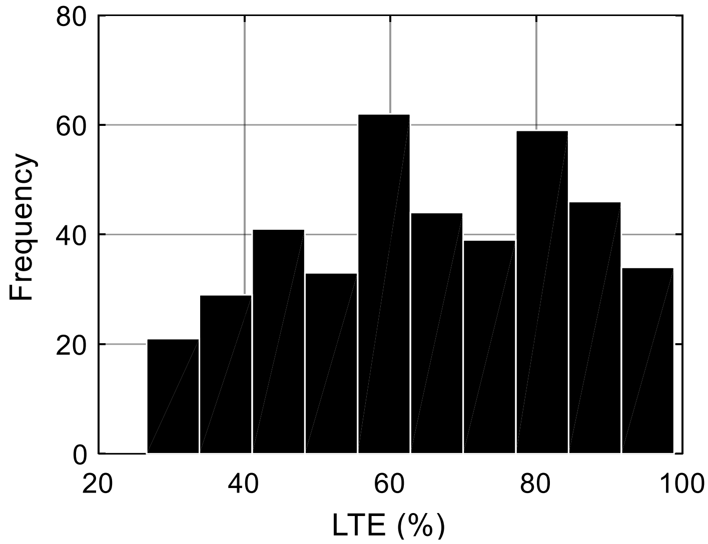
Mid-slab and joint testing were completed on six (6) residential streets in the City of Winnipeg to evaluate the load transfer and pavement structural capacities and to compare the performance of residential streets to regional roads. The typical residential pavement structure consists of 150 mm undowelled jointed plain concrete, 75 mm limestone base and 250 mm subbase. The tested roads are outlined in Table 3-3.

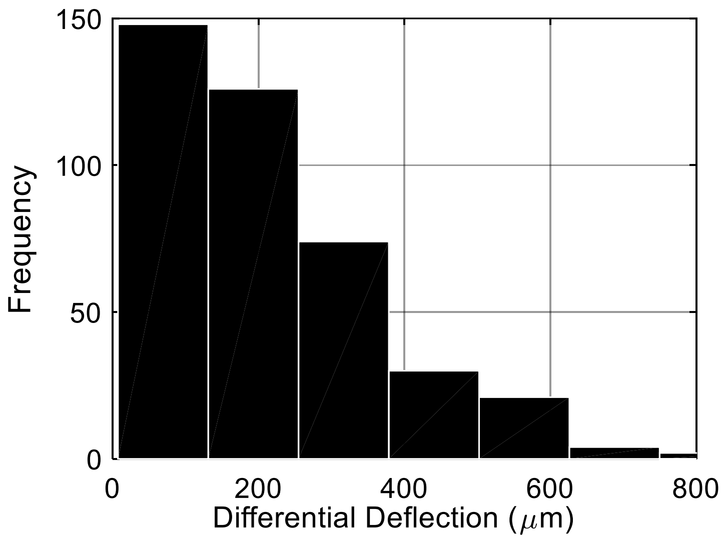
6.1. Load Transfer Efficiency of Residential Streets

LTEs, differential deflections, and peak deflections were computed for residential street joints to determine joints performance and evaluate their capacity compared to regional roads. This would allow for different rehabilitation actions and possibly different threshold values depending on a road's functional class. Fifty-six percent (56%) of joints did not meet the set criteria of 70% LTE, and 64 % experienced higher than 130 μm differential deflections while 67% of the joints exceeded 500 μm set criteria for peak deflections. Table 6-1 summarizes the joint performance statistics for each residential street. It is noted that Highgate Crescent has the most deteriorated joint performance compared to the rest of the streets with the lowest LTE and highest deflections. Camarthen Boulevard, on the other hand, has the best performance with joint paramters passing the threshold criteria set for regional roads. Figure 6-1 shows the distribution of LTE, differential deflections and peak deflections for the joints tested on residential streets. Low LTEs and high differential deflections indicate poor transfer of loads across the joints due to the absence of dowels. In addition, the base support is weak in the majority of the joints experiencing high deflections. A thicker base layer may reduce the percentage of joints deflecting higher than 500 μm by providing higher stiffness and support to the concrete slab. Moreover, such excessive deflections may call for slab stabilization to remedy potential voids under the concrete slab.

Table 6-1: Residential Streets LTE, Peak Deflections and Differential Deflections Statistics at 40 kN.

Parameter	Highgate Crescent	Bethune Way	Camarthen Boulevard	Oswald Bay	Best Street	Barbara Crescent
LTE (%)	42	63	84	59	63	77
LTE Standard Deviation (%)	12	14	12	12	16	17
Differential Deflection (μm)	268	139	73	186	233	110
Differential Deflection Standard Deviation (μm)	80	62	48	114	156	95
Peak Deflection (μm)	456	367	139	427	577	449
Peak Deflection Standard Deviation (μm)	51	62	62	167	171	61


80
60
40
20
0
Frequency
20
40
60
80
100
LTE (%)


150
100
50
0
Frequency
0
200
400
600
800
Differential Deflection (μm)

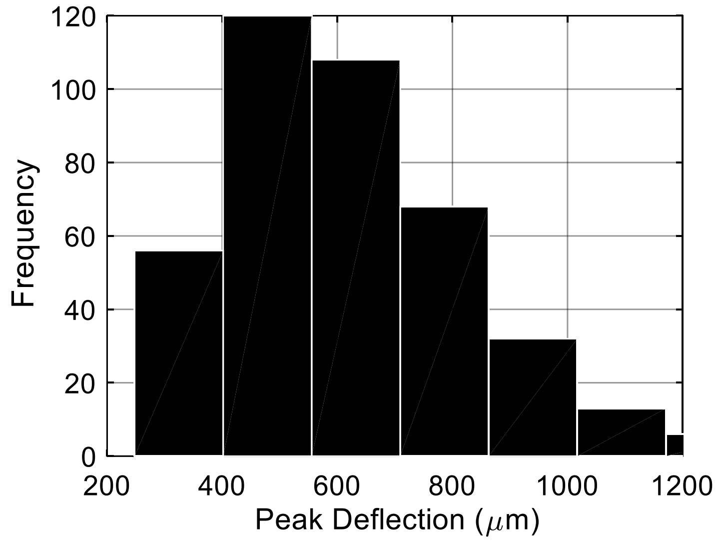
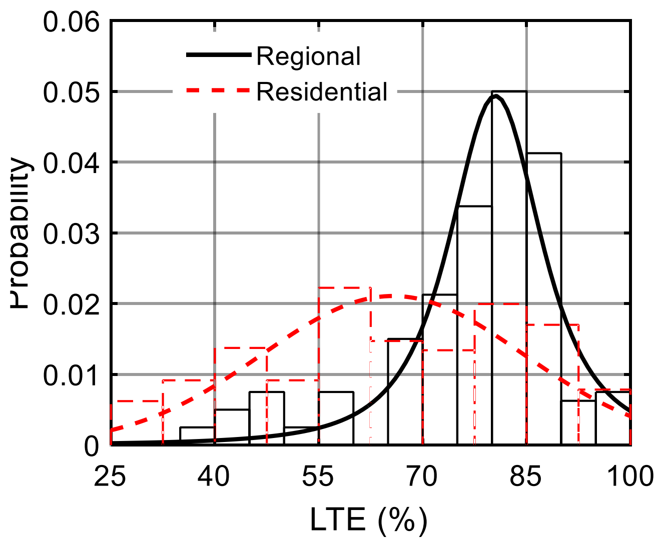


Figure 6-1: Histogram Distribution of LTE, Differential Deflections and Peak Deflections for Residential Streets.

Joints load transfer capacity was found to be considerably less for residential streets compared to regional roads. Figure 6-2 shows LTE, differential deflections, and peak deflections distributions compared to regional roads. These findings are summarized in Table 6-2. Despite being in fair to good condition, joints capacity for residential streets was lower than the set threshold values for LTE, differential deflections and peak deflections. On the contrary, regional roads were in poor to good condition and displayed joints capacity that is higher than the set thresholds for all parameters. Moreover, the joints performance of residential streets was generally more variable than regional roads for all parameters. This reflects a need for more targeted rehabilitation strategies for residential streets, as opposed to a blanket rehabilitation treatment for the full stretch. The lower load transfer capacity of residential streets is attributed to a less thick pavement structure as well as the absence of dowels. However, residential streets carry less traffic loads than arterial regional roads. Therefore, they may not be required to provide a similar load transfer capacity to regional roads. Moreover, maintenance trigger values applied were developed from regional roads and may not accurately reflect maintenance trigger values for residential streets. Developing such trigger values for low traffic roads should, hence, be explored.

Table 6-2: Residential Streets and Regional Roads Average LTE, Differential Deflections and Peak Deflections.

Parameter	Residential Streets	Regional Roads
LTE (%)	66	77
LTE Standard Deviation (%)	19	13
Differential Deflection (μm)	223	71
Differential Deflection Standard Deviation (μm)	175	99
Peak Deflection (μm)	628	258
Peak Deflection Standard Deviation (μm)	231	154



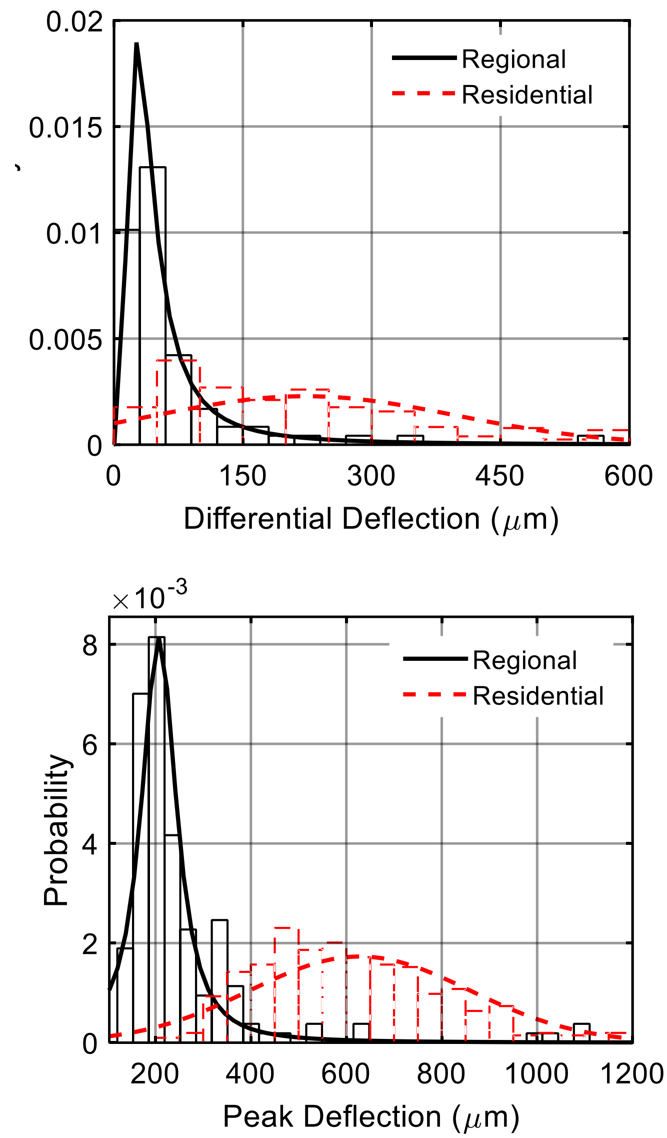
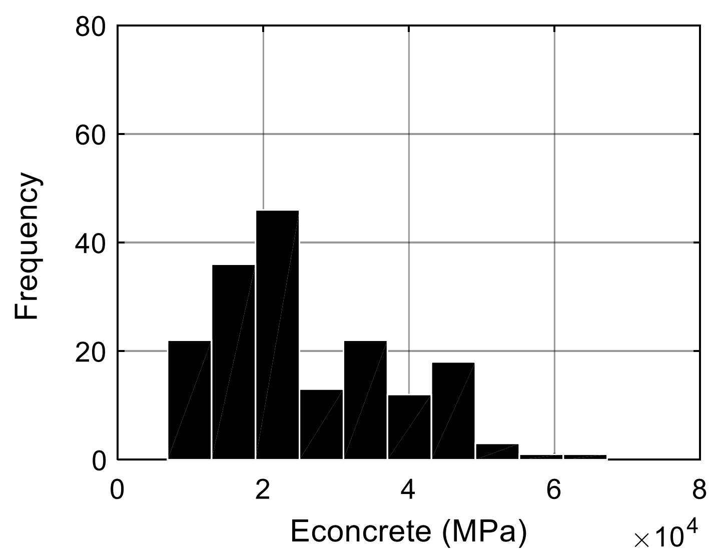
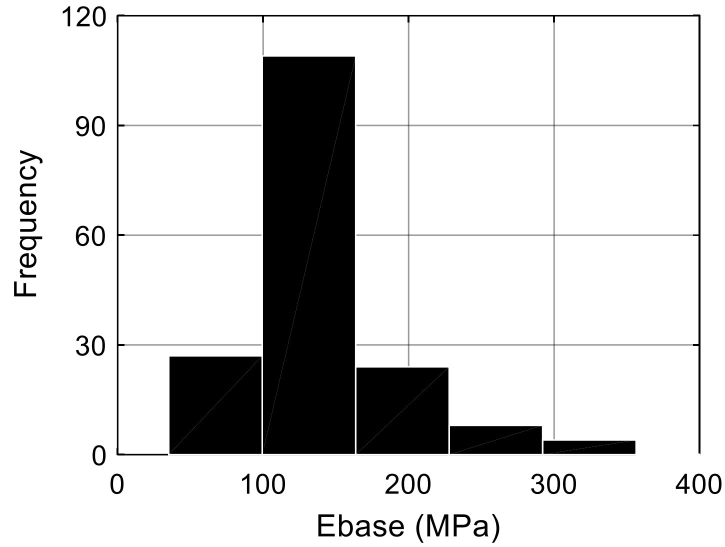


Figure 6-2: Residential Streets vs. Regional Roads Joints Performance.

6.2. Structural Capacity of Residential Streets

Layer moduli for the concrete, base, and subgrade layers were backcalculated for residential streets and regional roads. The backcalculation was completed using the widely used Elmod software. The output of the backcalculation is the stiffness of each pavement layer, namely concrete, base and subgrade. Layer moduli values are often used in assessing the structural capacity of a pavement layer, identify weak areas as well as design inputs. Figure 6-3 and Table 6-3 show the backcalculated average stiffness of concrete, base and subgrade are within anticipated values. This analysis does not, however, consider the effect of temperature slab curling on the concrete as no method has received widespread recognition to accurately capture such effect. Factors such as the presence of moisture and actual layer thickness have a direct impact on the base and subgrade backcalculated stiffness. Areas of weak subgrade and base could be candidates for strengthening as they show deteriorated structural capacity of the pavement structure.





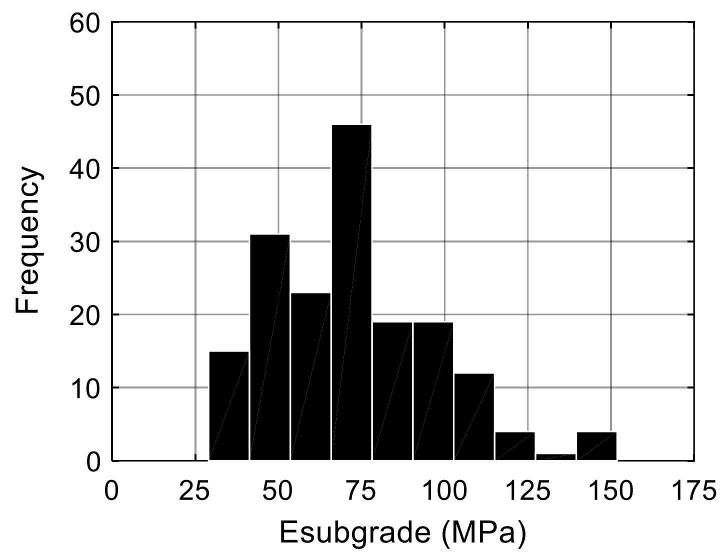


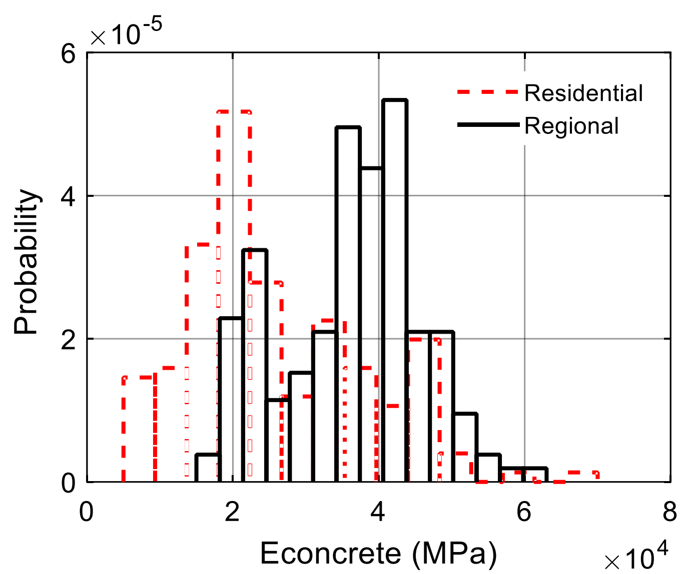
Figure 6-3: Histogram Distribution of Concrete, Base and Subgrade Stiffness for Residential Streets.

Residential streets layer moduli were lower than those for regional roads despite having a better visual condition rating, as shown in Table 6-3 and Figure 6-4. This might be attributed to an overall less thick pavement structure for residential streets. For instance, the concrete thickness for regional roads ranges between 200 to 250 mm wherein it is 150 mm for residential streets. The average backcalculated base layer moduli was also higher for regional roads due to thicker layer thickness. The average backcalculated subgrade moduli were found to be similar as all these sections were within the City of Winnipeg with a clayey subgrade material of a similar nature. However, it is noted that the stiffness of residential streets layers was more variable than regional roads. This highlights a need for a cross examination of the construction methods and materials as well as the applied specifications in constructing and rehabilitating residential streets compared to regional roads. Moreover, different design inputs for layer moduli should be considered depending on the roadway's functional class as the materials properties of each class are different.

In relating the backcalculated layer stiffnesses, it is noted that the concrete and base layer moduli are lower for residential streets as they have material with different properties specified for them. These results provide further explanation, in addition to different layer thicknesses, for the lower LTE and higher peak deflections that residential streets experienced. Having less stiff support and no dowels, residential concrete slabs experience higher deflections at the joints compared to regional roads and they do not share the applied loads onto adjacent slabs. In case higher traffic is anticipated to travel on residential streets, care should be given to their structural and load transfer capacitates.

Table 6-3: Residential Streets and Regional Roads Average LTE, Differential Deflections and Peak Deflections.

Parameter	Residential Streets	Regional Roads
Econcrete (MPa)	2.60×10^4	3.63×10^4
Econcrete Standard Deviation (MPa)	1.23×10^4	9.54×10^3
Ebase (MPa)	145	185
Ebase Standard Deviation (MPa)	72	45
Esubgrade (MPa)	72	71
Esubgrade Standard Deviation (MPa)	25	9



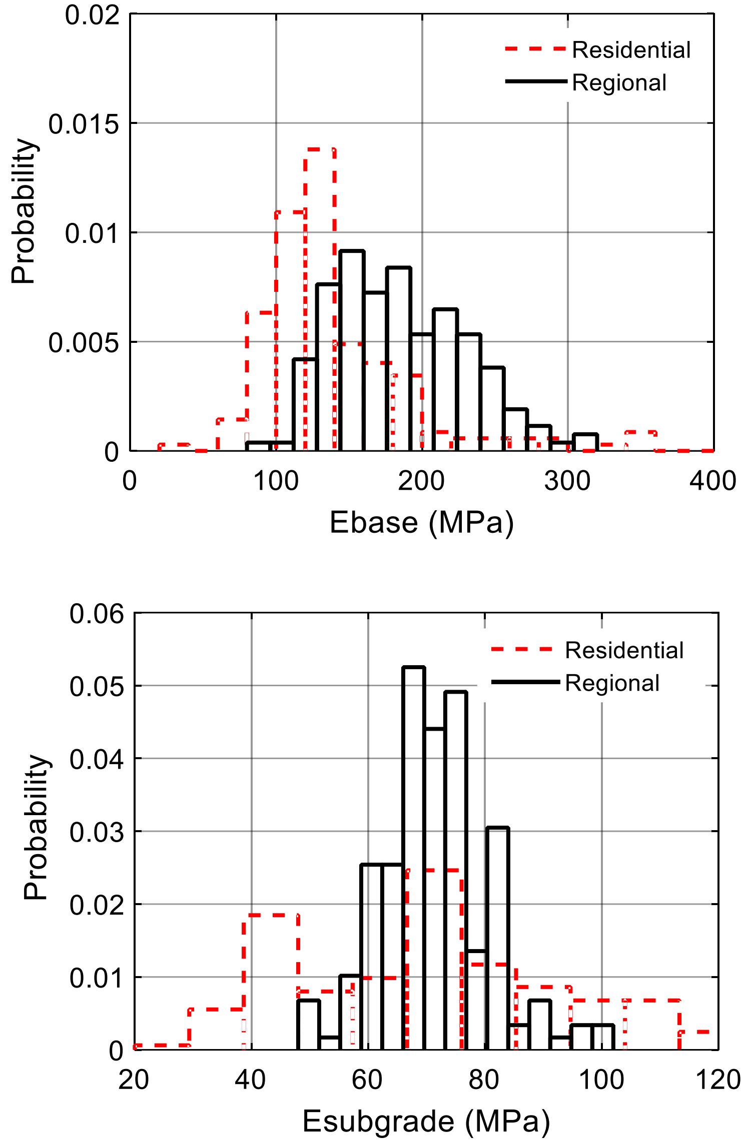


Figure 6-4: Residential Streets vs. Regional Roads Backcalculated Layer Moduli.

Chapter 7

SUMMARY, CONCLUSIONS, and RECOMMENDATIONS

7.1. Summary

Falling weight deflectometer (FWD) testing was completed on three urban arterial sections prior to and after rehabilitation as well as six residential streets. Two of the regional sections and all residential streets are concrete pavements while the third one is a composite structure. The planned rehabilitation treatments of interest were milling of the asphalt layer and full-depth repairs of concrete joints for the regional sections. The residential streets were tested but were not scheduled for rehabilitation. The primary objective of the testing was the application of non-destructive testing (NDT) in rehabilitation decision making process.

FWD testing was completed using two different geophones layouts in order to determine the more critical loading condition for further testing. Correlations were established between joint performance parameters to establish threshold values for triggering repairs. Statistical testing of collected data was completed to optimize FWD testing time and cost by reducing the number of loads required in FWD testing from four to two. Deflection information from FWD testing prior to and after milling of the asphalt overlay was used to determine the effect of the overlay on peak deflections, differential deflections, and LTEs to develop correction factors improved the reliability of FWD tests performed at the commencement of rehabilitation projects, and prior to the milling of the asphalt overlay for Pembina Highway. The correction factors are site specific and are considered to be the first step in realzing the influence of asphalt overlays on FWD measurements. Further studies are recommended to quantify the role of different structures, material properties and environmental conditions on the noted affect of overlays.

The results of FWD tests were used to evaluate the performance of full-depth repairs in restoring the load transfer capacity of concrete joints, and the benefit of incorporating FWD testing and joint performance evaluation at the design stage of rehabilitation projects. Lastly, FWD testing completed on residential streets was used to determine the joint performance and structural capacity of local roads and compare them to regional roads. Backcalculated layer moduli were obtained for each functional class to demonstrate the prospect of using FWD and the backcalculated moduli in the design stage.

7.2. Conclusions

1. LTEs were generally higher when tests were conducted using Dynatest® layout compared to those using LTPP layout. Peak deflections and differential deflections determined through the Dynatest® layout were lower than those determined through the LTPP layout. LTPP layout presented the more critical loading condition and more inferior joint performance for the same joint tested at the same time using the two layouts. LTPP layout was recommended to be used for joint testing.
2. Selected joint performance threshold value to trigger maintenance for LTE is 70% as it is the most widely used by agencies while the threshold value for peak deflections was selected to be 500 μm as it correlated to LTE as well as the presence of voids under the concrete slab. Lastly, selected threshold value for differential deflections was 130 μm. These threshold values are recommended for regional roads and it is recommended that

different threshold values are recommended for residential streets as they have different materials, structure and traffic loading.

3. FWD load levels of 25 kN and 40 kN are proposed to be used instead of four load levels (25 kN, 40 kN, 55 kN, and 70 kN) to optimize FWD testing time and cost as they result in statistically similar LTE values and void detection results.
4. LTE, peak deflections, and differential deflections are statistically different prior to and after milling wherein peak deflections and differential deflections increase while LTEs decrease after milling. Moreover, void detection analysis is not recommended to be performed using raw deflection values without accounting for the presence of the asphalt overlay.
5. Recommended correction factors improved void detection results by accounting for the presence of the asphalt overlay when FWD testing was completed on top of the asphalt. For instance, a deflection intercept was corrected from 4 μm to 17 μm compared to the 19 μm intercept obtained through FWD testing after milling of the asphalt overlay.
6. Correlation of deflections before and after milling, and the resultant correction factors, may be affected by asphalt overlay and concrete thicknesses, specific overlay mix design and concrete material properties, testing temperature and temperature gradient across the PCC. This study did not investigate the effect of these parameters.
7. None of the joints receiving FDR exhibited peak deflections higher than the 500 μm which indicates good base support condition under the concrete slab of the newly rehabilitated joints. However, low LTEs observed and corresponding high differential deflections indicate possible dowel looseness that may develop during construction. FDR construction method is recommended to be reviewed, including the dowel installation method.

8. FWD testing can optimize rehabilitation projects by applying treatments at joints that have high deflections and low LTEs. It was found that FDRs performed using visual inspection alone missed joints with low performance which could lead to faster deterioration in the future. FWD results can also be utilized to select the type of treatment of each joint, such as dowel bar retrofit or FDR based on measured peak deflections, differential deflections and LTEs.
9. Residential streets joints were found to have lower load transfer capacity compared to regional roads despite having a similar, or better, visual condition rating which highlights the need for non-destructive testing in addition to visual inspection to determine pavement structure performance.
10. LTEs, peak deflections and differential deflections for maintenance trigger (threshold) values are recommended to be established for low volume local roads exclusively due to their differing structure, material properties and performance compared to arterial regional roads.
11. Weaker residential pavement structure, and base layer specifically, contribute to lower and more variable joints load transfer capacity compared to arterial regional roads.
12. Average backcalculated layer moduli of 2.60×10^4 MPa for concrete, 145 MPa for base and 72 MPa for subgrade were determined for the six residential streets tested. Average backcalculated layer moduli of 3.63×10^4 MPa for concrete, 185 MPa for base and 72 MPa for subgrade were determined for the regional roads tested.

7.3. Recommended Future Work

1. Studying the effect of different asphalt overlay and concrete layer thicknesses on LTEs and void detection analysis.
2. Coupling the effect of asphalt temperature and PCC temperature gradient on deflections in the presence of asphalt overlays.
3. Further FWD testing and modelling for the development of correction factors as functions of pavement structure and condition.

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