Project 5-9046-03

NOISE ABATEMENT AND PERFORMANCE EVALUATION OF A NEXT-GENERATION DIAMOND GRINDING TEST SECTION IN HARRIS COUNTY

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16. Abstract

This research project compared the performance of a 0.68-mile long continuously reinforced concrete pavement test section before and after receiving next-generation diamond grinding (NGDG). The test section was tranversely tined and located on Loop 610 in Harris County, Houston, Texas. It included the two rightmost lanes on each traffic direction. TxDOT collected sound intensity, skid resistance, ride quality and macrotexture data before griding, as well as 3 and 6 months after construction. Equipment was: laser scanner, locked wheel trailer, and on-board sound intensity (OBSI). The analysis methodology consisted of a preliminary investigation to verify data consistency and homogeneity, followed by a comparative analysis of pre- and post-NGDG performance. The NGDG surface significantly improved the skid resistance as well as the ride quality of the test section analyzed. Macrotexture showed a less significant improvement; however, the macrotexture analysis was inconclusive due to some possible issues with the data. The test section overall sound intensity decreased from 107.6 to 101.7 dBA, a value that recent studies consider within the quietest possible range for non-porous concrete pavements. The mathematically correct subtraction of 107.6-101.7 dBA, when converted back to decibels, removes 106.3 dBA from the enviroment. This amount of noise between a sports event and a rock band. This is also equivalent to a 75% reduction in noise level. Under the simplifying assumption that all vehicles produce the same noise, the NGDG surface needs 4 times more traffic to generate the same noise as the previous surface.

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Report 5-9046-03-F
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Project Title: Pre-Construction and Next Generation Concrete Surface (NGCS) Noise Monitoring on US290 Harris County

Performed in cooperation with the
Texas Department of Transportation
and the
Federal Highway Administration

September 2016

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the FHWA or TxDOT. This report does not constitute a standard, specification, or regulation.

This report is not intended for construction, bidding, or permit purposes. The engineer(researcher) in charge of the project was Jose Weissmann, P.E. #. 79815.

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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This project was conducted in cooperation with TxDOT and FHWA. This study was developed in close collaboration with TxDOT personnel, who provided invaluable ideas, support and supervision, and collected all data used in this study. The UTSA team is very grateful to TxDOT and also acknowledges the flawless team work central to the successful completion of this study.

The authors thank all TxDOT's personnel involved in this project, whose collaboration was crucial for a timely and successful conclusion.

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The University of Texas at San Antonio
COLLEGE OF ENGINEERING

EXECUTIVE SUMMARY

Texas Department of Transportation

Research Report 5-9046-03-F
Noise Abatement and Performance Evaluation of a Next-Generation Diamond Grinding Test Section in Harris County

Acknowledgements

This study was developed in close collaboration with TxDOT personnel listed in the main report. They provided invaluable support and supervision, and collected all data used in this study. The UTSA team is very grateful to TxDOT and acknowledges the flawless team work central to the successful completion of this study.

Study Objectives and Scope

This research project analyzed the performance of a concrete pavement surface treatment called next-generation diamond grinding (NGDG), by comparing data collected before and after grinding on a test section located in Harris County, Houston, Texas. The study evaluated the NGDG performance in terms of macrotexture, ride quality, skid resistance and tire-pavement noise.

This study scope consisted of a 0.68-mile long test section of Loop 610 in Houston, Texas. The test section comprises the two rightmost lanes on each traffic direction between TC Jester and Ella Blvd. It does not include bridge structures, which did not receive NGDG. Figure. 1 shows the test section location, the lanes surveyed and the lane nomenclature, which follows TxDOT's Pavement Management and Information System (PMIS) nomenclature.

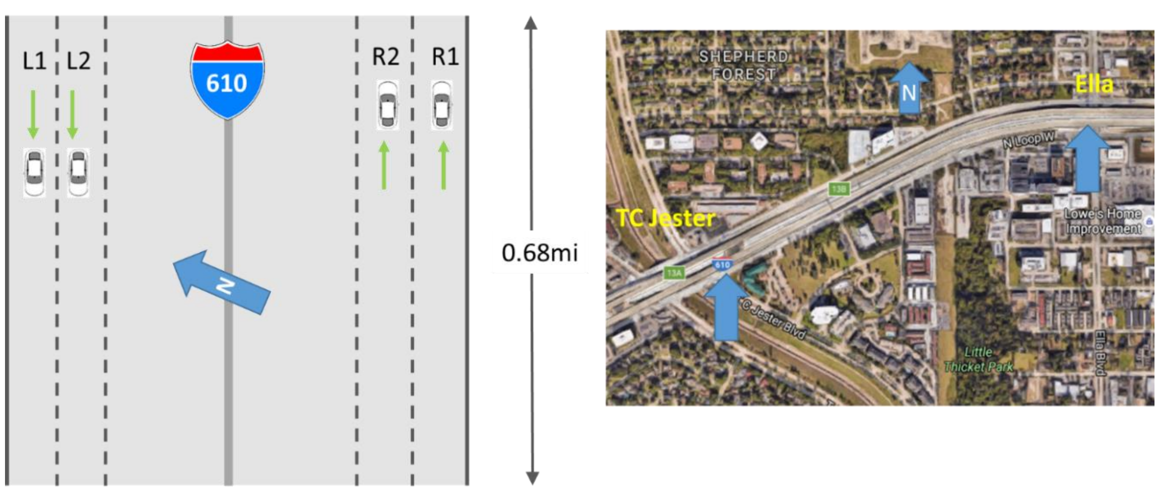

Figure. 1 Study Test Section

Satellite map source: Google Maps

The study was organized into the 5 tasks listed below. The contract did not specify a literature review task but it was necessary, was requested by TxDOT, and therefore was documented in this report. The contractual tasks were:

Task 1 Visit Project Sites

Task 2 Develop Monitoring Plan

Task 3 Perform Pre-Diamond Grinding Measurements

Task 4 Perform Post-Diamond Grinding Measurements

Task 5 Final report with comparative analysis.

What the research team did

During Tasks 1 and 2, the UTSA-TxDOT research team decided to collect sound intensity, skid resistance, ride quality and macrotexture data using TXDOT's equipment and personnel. The equipment used and the test specifications were:

- Laser texture scanner for texture and ride quality (ASTM E2157/ASTM E1845)
- Locked-wheel skid trailer, ASTM E274
- On-board sound intensity (OBSI) equipment mounted on a vehicle equipped with standard tires (AASHTO TP 76)
- Multi-purpose van.

TxDOT collected the types of data summarized in Table 1, before and after NGDG (next generation diamond grinding), in the following dates: Pre-NGDG, in November 2014; the first set of post-NGDG data in March 2016 and the last set of post-NGDG data in July 2016 (respectively 3 and 6 months after opening to traffic).

Table 1 Summary of Data Collected

Data Type	Data Description
Skid Resistance	Excel workbooks with 13 data points/lane, numbered from 0 to 12 for each lane, at 0.05-mile-long intervals. Each data point consists of: minimum, average, maximum skid number (SN), peak value and percent slip.
Ride Quality	Raw data files compatible with ProVAL. Continuous survey, left and right wheels. Surveyed section longer than test section. Approximately 45,000 data points/lane.
Macrotexture	Macrotexture data consists of mean profile depth (MPD) measured in millimeters (mm), reported at every 2ft.
Sound	Three 440-ft-long segments per lane, 3 data runs per segment, 2 microphones. 400 to 5000 Hz 1/3 octave band and narrow band (1/24 octave) provided for each data run, microphone and lane segment.

Once the TxDOT team delivered the data, the UTSA team analyzed it according to a methodology that basically consisted of a preliminary analysis to check data quality, consistency and homogeneity, followed by a comparative analysis between pre- and post-NGDG performance. The preliminary analyses included, but were not restricted to, data quality checks specified in the standards, homogeneity tests among lanes and sound data segments, and visual inspection of plots, histograms and boxplots. The raw ride quality data was pre-processed with ProVAL to obtain ride number (RN) and international roughness index (IRI). These indices were then further analyzed using SAS, a database management and statistical analysis package, also used to analyze the other types of data.

What the researchers found and recommended

Skid Resistance

NGDG significantly improved the overall skid resistance of the concrete pavement. Overall improvements for the aggregated data (entire test section) were:

- 95% confidence interval for the average pre-NGDG Skid Number (SN): 18.7 ± 2.2
- 95% confidence interval for the average post-NGDG SN: 33.7 ± 1.0
- Average SN improvement: 59.5%
- Smallest SN improvement: Lane R2, 15.2%
- Greatest SN improvement: Lane L1, 102.1%
- Overall percent slip improvement: 35%

Ride Quality

The post-NGDG data necessarily included non-treated segments, since the surveyed length was longer than the NGDG length. Due to lack of accurate information on the start and ending points of the survey, it was not possible to eliminate all non-NGDG data points from the analysis. Nevertheless, the overall improvements in ride quality were significant. Depending on the lane, improvements ranged from:

- 91% to 202% for IRI
- 35% to 64% for RN

The researchers also calculated the sum of all segments with IRI > 95, a threshold for re-grinding in construction quality control. There were significant improvements, as listed below.

- Pre-NGDG: 2,853ft or 80% of the surveyed length had IRI > 95.
- Post-NGDG: 480ft or 11% of the surveyed length. Note: it is reasonable to hypothesize that most if not all post-NGDG segments with IRI > 95 were outside the test section and therefore not ground.

Macrotexture

As explained in the ride quality section, the post-NGDG survey length was greater than the test section length, thus it included untreated segments. Macrotexture results were inconsistent, indicating a considerable mean profile depth (MPD) improvement for the 3-month post-NGDG but not for 6-month post-NGDG. The post-NGDG-3-mo MPD improvements are consistent with the improvements observed for the other parameters that evaluate the surface roughness. If one considers only the consistent measurements, the post-NGDG overall MPD improved 24.7% with respect to the pre-NGDG.

Sound

A 2006 study by the Iowa University's National Concrete Pavement Technology Center defined noise zones for pavements. Zone 1, "innovation zone," is the quietest, followed by Zone 2, "quality zone". Zone 1 has the following caveat: "It appears that conventional (dense) concrete may not have the ability to be built consistently in Zone 1. Research and innovation will therefore be required to develop solutions that consistently provide OBSI levels within the zone." The NGDG fell in Zone 2 which, according to this study, is as quiet as possible for today's dense concrete pavements.

The difference in pre minus post-NGDG decibels (direct subtraction) was always greater than 5.0, the threshold for "noticeable change," according to an evaluation table used by TxDOT. This was true for every lane and for the overall test section.

The actual sound intensity decreases ranged from 105.3 dBA (lane R1, segment A, post-NGDG-3mo) to a maximum of 107 dBA (lane R2, segment CC, post-NGDG-6mo). The test section overall average sound intensity decreased 106.3 dBA. Therefore, the NGDG removed from the environment an amount of noise between that of a sports event (about 105 dB) and a rock band (about 110 dB).

The abovementioned sound intensity reductions translate into in an overall 75% decrease in this test section's noise level.

The ratios of before / after sound intensities ranged from 3.12 (lane L2, segment FF, post-NGDG-3mo) to 4.79 (R2, CC, 6mo). The overall test section average reduction factor was 4.08. Assuming that all vehicles cause the same noise, these ratios can be interpreted as traffic reduction factors. According to the data, the post-NGDG test section would cause as much noise as the pre-NGDG section only when carrying 4.08 times more traffic.

The abovementioned noise level ratios can also be expressed as years of traffic growth. The overall test section noise reduction factor of 4.08 means that the post-NGDG surface would cause the same noise as the pre-NGDG does with today's traffic after 28.8 years of steady traffic growth at a 5% annual rate.

Recommendations

This study originally intended to analyze another NGDG test section on US 290, also in Harris County, TX; however changes in construction schedules precluded post-NGDG data collection. It is recommended to survey this test section after construction completion and perform the same analyses discussed in this study's main report (report number 5-9046-03-F).

This study's schedule did not allow proper durability evaluation. The two post-NGDG measurements were taken about 3 and 6 months after construction completion, not enough time for concrete pavement surface treatments to deteriorate. It is therefore recommended to collect post-NGDG data after 1 and 2 years of traffic. Annual data collection for 5 consecutive years would provide enough data for a time-series durability analysis, especially when coupled with data on traffic and heavy vehicle volumes.

What this means

The NGDG surface significantly improved the skid resistance as well as the ride quality of the test section analyzed. Macrotexture showed a less significant improvement; however, the macrotexture analysis was inconclusive due to inconsistencies with the other indices related to roughness as well as inconsistencies between the 3- and 6-month post-NGDG data. Some of these inconsistencies may have been due to the fact that the surveyed length included bridge segments which were not treated with NGDG.

The NGDG surface noise level decreased to values that are currently considered the quietest possible for dense concrete pavements. The sound intensity difference between pre- and post-NGDG corresponds to a 75% reduction in noise level, or an equivalent traffic reduction on the pre-NGDG pavement. It also corresponds to removing from the environment an amount of noise somewhere between a sports event and a rock band.

If the extra cost and inconvenience of grinding cured concrete as opposed to texturing uncured concrete are a consideration for future construction, it is recommended to develop a more comprehensive study comparing other types of treatments capable of performing similarly to NGDG. If possible and convenient, the study should include tests sections built with porous concrete pavements currently considered innovative.

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Chapter 1
Introduction

Project Motivation

Traffic noise has increasingly become a nuisance and an environmental concern for the general public, thus affecting transportation agencies all over the world. This concern has motivated the development of new methods to treat Portland cement concrete pavements (PCCP) to decrease traffic noise without sacrificing skid resistance. This research project evaluated the application of next generation diamond grinding (NGDG) on existing transversely tined PCCPs in Harris County, Texas.

Although the Federal Highway Administration (FHWA) allows the use of federal funds for noise reduction, it also stipulates that pavement type or texture cannot always be considered a noise abatement measure. Nevertheless, there are advantages to reducing the noise at the source rather than placing a barrier: drivers also benefit, in many cases it is more cost-effective to treat the pavement than to build barriers, barriers may cause aesthetic concerns and in many urban locations they adversely impact access to adjacent facilities. If a pavement is built to be quieter and is able to retain its quiet characteristics over time with reasonable maintenance, the FHWA may approve its use in the future as a noise abatement measure (FHWA, 1997).

Project Objectives

In order to address the previously stated issues, the Texas Department of Transportation initiated this study. Its purpose was to monitor next-generation diamond grinding (NGDG) recently implemented on existing transversally tined Portland Cement Concrete Pavements (PCCP) in Houston in order to improve noise, ride quality and skid resistance. This project measured the pavement performance before and after NGDG and analyzed the improvements. Sound, texture, skid resistance and ride quality data were collected on three occasions, termed as follows: pre-NGDG, post-NGDG, and mid-range-NGDG. The latter two are termed post-NGDG-3-months and post-NDGD-6 months in this report, referring to the fact that these data were collected approximately three and six months after construction completion. The specific data collection dates were as follows:

1. Pre-NGDG: November 3-4, 2014:
2. Post-NGDG (post-NGDG-3mo): March 14-16 ,2016, and
3. Mid-range-NGDG (post-NGDG-6mo): June 29-30, 2016.

Initially, the research included PCCP on the US290 expansion and on Loop 610 in the Houston District. Measurements taken in November 2014 reflect this initial objective. Due to construction schedule changes after the research contract was in effect, US290 was not completed in time for this project. Thus, the comparative analyses, which were the main objective of this research, could be performed only for Loop 610.

This project developed methodologies to analyze noise, skid, texture and ride quality data, as well as methods to report noise reduction in terms that public understanding of the fact that decreasing a few decibels translates into a significant traffic noise reduction.

Project Tasks and Report Organization

This project was organized into 5 tasks as follows:

Task 1 Visit Project Sites

Task 2 Develop Monitoring Plan

Task 3 Perform Pre-Diamond Grinding Measurements

Task 4 Perform Post-Diamond Grinding Measurements

Task 5 Final report with comparative analysis

This report is organized into the 5 chapters, executive summary and 3 appendices listed below. Chapter 1 is this introduction and Chapter 2, the literature review. Tasks 1 and 2 are documented in Chapter 3. Chapters 4 and 5 document tasks 3, 4 and 5, organized by type of data collected in this project. In lieu of a final Chapter (6) summarizing conclusions and recommendations, the report starts with a stand-alone 5-page executive summary.

- “Executive Summary.” A stand-alone 5-page summary report briefly discussing the project objectives, data collected, findings, conclusions, and recommendations. It also contains information on the authors and the sponsoring agency.
- Chapter 1, "Introduction," presents the project's motivation, objective and organization, followed by this report organization.
- Chapter 2, "Literature Review," discusses experiences of researchers and of transportation agencies in the United States and abroad. The contract did not stipulate a separate task for literature review, but it was necessary and its findings are summarized in this chapter.
- Chapter 3, "Monitoring Plan," provides an overview of the data collection methodology, schedules, devices used and test section locations. It covers Tasks 1 and 2 of the project, respectively titled "Visit Project Site" and "Develop Monitoring Plan," as well as Products 1 and 2, "List of Available Equipment" and "Documented Monitoring Plan."
- Chapter 4, "Comparative Analysis of Skid Resistance, Ride Quality and Macrotexture Data," discusses the analysis methodology, summarizes the data collected, presents the results of statistical analyses performed and discusses the conclusions about NGDG performance in terms of mean profile depth (MPD), skid number (SN) and percent slip, international roughness index (IRI) and ride number (RN).

- Chapter 5, "Comparative Analysis of Sound Data," discusses the analysis methodology, summarizes the data collected, presents the results of statistical analyses performed and discusses the conclusions about NGDG performance in terms of noise reduction and in terms of the traffic reduction necessary to reduce noise by the observed amount.
- Appendix 1 has the complete set of comparative plots of skid number (SN) and percent slip data for Loop 610.
- Appendix 2 has the complete set of comparative plots of international roughness index (IRI) and ride number (RN) for Loop 610.
- Appendix 3 has the complete set of comparative histograms of mean profile depth (MPD) data for Loop 610.

Chapter 2
Literature Review

This chapter summarizes the literature review, which focused primarily on the following subjects: (1) traffic noise and sound measurements, (2) Portland cement concrete pavement (PCCP) treatments' performance in terms of noise reduction, and (3) PCCP treatments' performance in terms of ride quality, skid resistance and texture.

Sound Measurements

The term "noise" refers to unwanted or unpleasant sound, but technically noise and sound are the same. Sound is an effect of change in air pressure, behaving like air ripples around the fairly constant local atmospheric pressure, detected by the ear as well as by microphone membranes. The human ear detects pressures ranging from 20×10^{-6} Pa to 20 Pa. Such extremely large range is poorly represented in linear units. All normal sounds would end up so close to the lower threshold of hearing that it would be impractical to plot sound measurements in normal environments. Therefore, the linear sound pressure is converted to a quantity termed "sound pressure level" (SPL or L_p) according to equation 2.1 below (Sandberg and Ejsmont 2002):

$$L_p = 10log\left(\frac{p}{p_{ref}}\right)^2 = 20log\left(\frac{p}{p_{ref}}\right) \quad (2.1)$$

Where:

L_p = sound pressure level, measured in decibels (dB)

p = sound pressure

p_{ref} = reference pressure of 2×10^{-5}Pa

L_p is measured in decibels (dB). The reference pressure of 2×10^{-5}Pa is the standard value for the lower threshold of human hearing. Decibels are defined so that the range of sounds between the lower threshold of human hearing and the threshold of pain is between 0 and 120 dB (Sandberg and Ejsmont 2002). In other words, decibels are the ratio between the sound pressure being measured and the threshold of human hearing.

Human hearing is not equally sensitive to all frequencies. For example, a high-frequency sound (shrill) can be more annoying that a low frequency one. When measuring sounds with the objective of analyzing how humans respond to the sources, it is necessary to filter frequencies. The "A" filter, considered to best mimic human perception of sound, was used in all sound measurements analyzed in this study. It is common to write the unit of sounds measured with this filter as dBA or dB(A).

Sound sources emit a large range of frequencies, or a frequency spectrum. Regardless of the measurement equipment, the signal will be distributed over a certain bandwidth. Commonly reported bands are 1/24 octave (narrow) and 1/3 octave. A 1/n octave bandwidth sets the band's highest frequency (fh) and lowest frequency (fl) so that fh = fl*$2^{1/n}$. Other common fractional octave analyses include 1/6, 1/12, and 1/24 of an octave.

The narrower the band, the better the resolution; on the other hand, narrow bands show too many details which are often of random origin and do not provide useful information. The vast majority of the pavement noise studies use the 1/3 octave band because, over the important frequency range, it resembles the human auditory system's own way to subdivide the sound into frequency bands (Sandberg and Ejsmont, 2002). This band is also recommended in AASHTO standard method 76 (2015)

This project used the on-board sound intensity (OBSI) equipment. Sound intensity is a vector with magnitude measured in W/m^2, which represents the sound power flow through a unit area. OBSI uses a probe with two microphones spaced apart by specified distance to determine the sound direction. Rasmussen et al. (2011) list three advantages of using sound intensity instead of sound pressure for measuring tire-pavement noise at the source. First, the directional characteristic of the probe makes it better suited for measuring a specific noise source, while attenuating sounds from other sources in other directions (such as engine or exhaust noise). Second, sound intensity is much less contaminated by "random" noise, such as wind noise generated as the vehicle is moving. Third, because sound intensity measures the acoustic energy propagating away from the source to the roadside, it correlates well with sound measured at the roadside (known as pass-by or wayside measurements).

Similarly to sound pressure (see equation 2.1), the sound intensity level is also measured as a ratio to a reference intensity in a decibel logarithmic scale, according to equation 2.3. The reference sound intensity I_{ref} is equal to 10^{-12} W/m^2, a value selected so that, in an acoustic-free field, one obtains the same dB when measuring pressure and intensity (Sandberg and Ejsmont, 2002).

$$L_I = 10log\left(\frac{I}{I_{ref}}\right) \qquad (2.3)$$

Where:

L_I = sound intensity level in dBA (often termed OBSI when measured with the on-board sound intensity equipment).

I = measured sound intensity

I_{ref} = reference intensity

Kohler (2010) reports a correlation between on board sound intensity (OBSI) measurements and vehicle speed developed in California. The correlation has a near-perfect R^2 of over 99%, and is depicted in equation 2.2:

$$n = 0.2228*S + 88.741 \qquad (2.2)$$

Where:

n = noise measured with OBSI in dBA

S = speed in mph

Although the author states that “more research is needed,” he also states that California recommends a correction factor of 0.22 dB per mph. The NCHRP recommends 0.28 dB per mph, and the author, 0.25 dB per mph.

Studies Comparing Noise Levels of PCCP Treatments

The Iowa State University’s National Concrete Pavement Technology Center (2006) developed a comprehensive study of PCCP noise reduction treatments. This FHWA-sponsored study concluded the following:

- The general population of concrete pavement textures’ average OBSI levels ranged from a low end of approximately 100 dBA to a high end of 113 dBA.
- The tire-pavement noise data ranked drag and grinding among the quieter textures and transverse tining among the loudest.

Of particular interest is this study’s definition of noise zones to interpret and evaluate PCCP noise levels. The three noise zones defined in this Iowa study (2006) are:

- Zone 1: low noise level or “innovation” zone, with OBSI values in the 99/100 dBA and below range. With the exception of some experimental pervious concrete pavements, there were no concrete solutions in Zone 1. It appears that conventional (dense) concrete may not have the ability to be built consistently in Zone 1. It has been demonstrated that in rare circumstances small portions of some in-service concrete pavements do fall within the Zone 1 range. Research and innovation will therefore be required to develop solutions that consistently provide OBSI levels within the zone.
- Zone 2: mid noise level or “quality” zone, with OBSI values approximately in the 99/100 to 104/105 dBA range. The target for both new and existing concrete pavements should be in this zone. It represents solutions that provide a balance of noise, friction, smoothness, and cost effectiveness. Grinding and burlap/turf drags often result in “quality” decibel levels and may provide the easiest method to attain zone 2 values.
- Zone 3: high noise level or “avoid” zone, with OBSI values in the range of approximately 104/105 dBA and above. This zone includes highly variable textured pavement, very aggressive transverse textures, and older pavements with serious joint deterioration. A significant amount of existing concrete pavements in the United States fall within this range.

Gharabegian and Tutle (2002) compared longitudinal tining to diamond grinding and found that “the average noise drop of the maximum measured single pass-by overall noise levels is approximately 6 dB at 7.5 m (25 ft) and 4 dB at 15 m (50 ft). Therefore, the single vehicle 'pass-by' measurements indicate that noise from tire/pavement interaction is likely to be perceptibly quieter for a diamond ground pavement versus a pavement with longitudinal grooves. This is especially applicable to a roadway where there is little truck traffic. At a roadway where there are large numbers of heavy trucks a noticeable noise reduction may not be achieved because the main truck noise comes from the engine and exhaust stack." The study also says that: "However, the 15 min measurements at 10 m (33 ft) indicate that there is about 3 dB noise reduction due to the diamond grinding for all vehicles, including heavy trucks. When heavy trucks are excluded, the noise reduction is about 4 dB."

An ACP (2006) study measured noise levels observed in the following types of surface textures: ground (diamond grinding), longitudinal tining, and transverse tining (random and uniform). This study's conclusions also favored diamond grinding over the other treatments, as depicted in Figure 2.1.

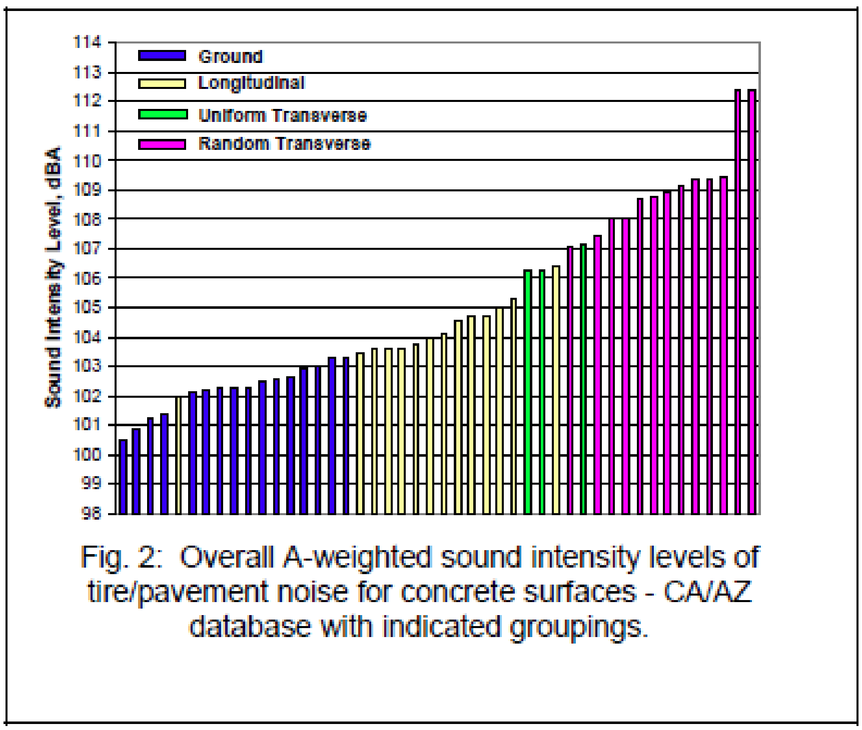


Figure 2.1 ACP Study Results

Source: ACP 2006

Donavan (2005) compared diamond grinding to longitudinal tining and found a considerable reduction in noise, as depicted in Figure 2.2. The same reference also reports that, in California, grinding of bridge decks and elevated structures reduced tire/pavement 3 by 10 dB. In Arizona, grinding of PCCP has reduced source levels up to 9 dB relative to some transversely tined surfaces. Measurements conducted in Europe using the same measurement methodology indicated a range of 11 dB including more novel porous PCCP surfaces.

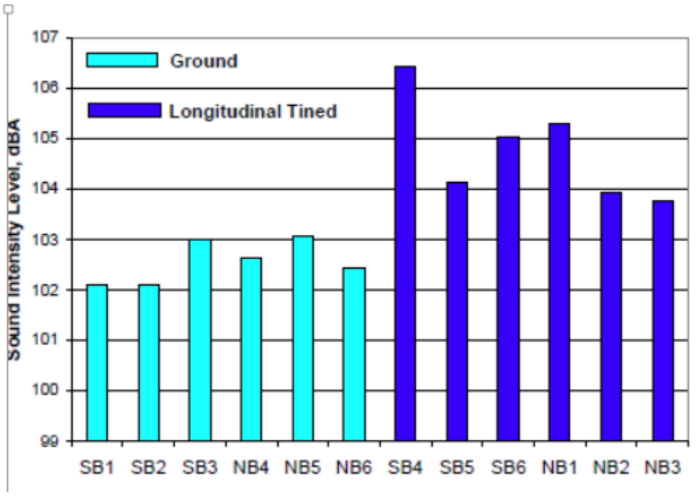


Figure 2.2 Diamond Grinding and Longitudinal Tining

Source: Donovan 2005

Rasmussen et al (2008) reported that the Concrete Pavement Surface Characteristics Program (CPSCP) evaluated nearly 1,500 concrete pavement textures worldwide and reported the statistical distributions of the noise levels depicted in Figure 2.3. This was the largest database we were able to find in the literature, and it indicates that diamond grinding is less noisy than the other PCCP treatments.

Scofield (2012), conducted a comprehensive study of next generation concrete surfaces (NGCS) for the American Concrete Pavement Association (ACPA). The study found that, at the time of construction, the NGCS is typically 99 dBA in noise level and has a typical range up to 101 dBA over time. This reference is very detailed and may be useful to TxDOT engineers in charge of selecting specific types of next generation concrete surfaces.

Izevbekhai (2007) compared before-and-after OBSI measurements in Minnesota on a new type of diamond grinding. The results showed that "the innovative grind achieved a high level of quietness surpassing previously known configurations of grinding. At 98.5 dB(A) the innovative grind was much quieter than both the conventional grind 102 dB(A) and the un-ground tie 104 dB(A)". This author also states that "a reduction of the sound intensity by 3 dB(A) is equivalent in effect to a traffic reduction to 50 % of original ADT."

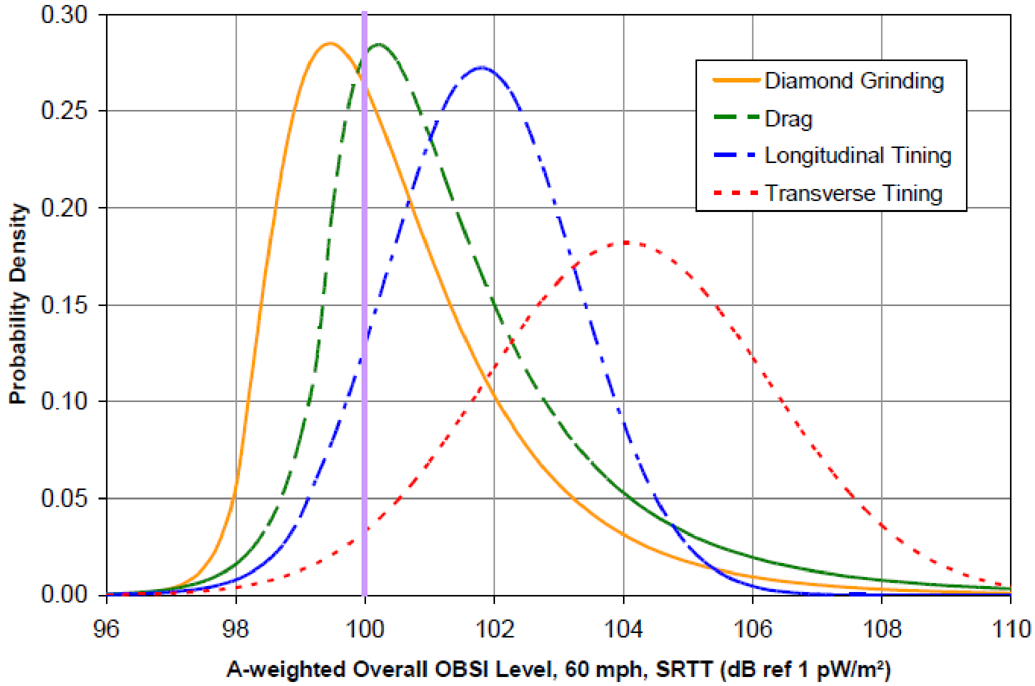


Figure 2.3 Normalized Distributions of Sound Levels

Source: Rasmussen et al. CPSCP 2008

Since decibels are a logarithmic scale, the above assertion "reduction of sound intensity by 3 dB" (3=102-99) is not mathematically correct. Logarithmic scales cannot be meaningfully added or subtracted. Each sound measurement in dB (in this example, 102 dB and 99 dB) must be converted back to their corresponding sound intensities, which should be subtracted then expressed back in decibels. Equation 2.4 shows how to add (or subtract) the sound intensity levels and convert the result back to the decibel scale. Actually, a drop from 102 dB to 99 dB is equivalent to almost 99 dB decrease in sound intensity. Conversely, adding these two sound intensities would result in 103.8 dB, not 201 dB.

$$dB_{total} = 10log\left[\sum_{i=1}^{n} 10^{\frac{dB_i}{10}}\right] \qquad (2.4)$$

Izevbekhai (2007) used this concept to estimate the traffic reduction necessary for the untreated pavement to cause the same noise level as the treated pavement, based on the simplifying assumption that all vehicles emit the same sound intensity. Under this assumption, the summation depicted in equation 2.4 becomes a multiplication by "n," and the ratio between intensities before and after the treatment is the equivalent traffic reduction. This concept was also used in this project to evaluate the NGDG noise reduction.

Wirth (2008) presented a table to evaluate perception of sound in terms of the direct decibel subtraction/addition, as depicted in Table 2. 1. This table was used in this project in addition to intensity subtraction and Izevbekhai's (2007) method to evaluate the NGDG sound reduction.

Table 2. 1 Human Perception of Sound Reduction

Change in Decibels (dB)	Change in Loudness
1 to 3	Just perceptible
5	Noticeable
10	Twice or ½ as loud
20	Four times (or ¼) as loud

Pavement Surface Characteristics

Pavement texture, noise, skid resistance, and ride quality all depend on the pavement surface characteristics. Construction practices aim at providing balance between ride quality (smoothness), noise and safety, which requires some roughness to provide proper tire/pavement friction. NCHRP 291 (2000) appears to still be the most comprehensive literature review on those practices. However, numerous other studies have been developed more recently, which are also discussed in this section.

Macrotexture

Macrotexture is a function of aggregate size and shape, providing improved friction between the vehicle's tires and the pavement at high speeds. Among the available indices to represent macrotexture, the mean profile depth (MPD) was provided by TxDOT for all three data collection efforts discussed in this report.

Despite its name, MPD is not simply the mean, or average, of all profile depths measured in the field; rather, it is the average obtained after averaging peak levels. Figure 2.4 illustrates the mean profile depth (MPD) definition. NCHRP 191 (2000) provides a comprehensive overview of macrotexture indices and measurement techniques.

Several studies have shown an increase in vehicle crashes once macrotexture falls below a certain threshold. These studies agree on an MPD between 0.4 mm to 0.5 mm as the value below which the crash rate significantly increases. Figure 2. 5 illustrates these findings. It shows a relationship developed by Cairney (2006), which clearly suggests 0.5 mm as a threshold above which an increase in macrotexture has considerably less impact on the crash rate.

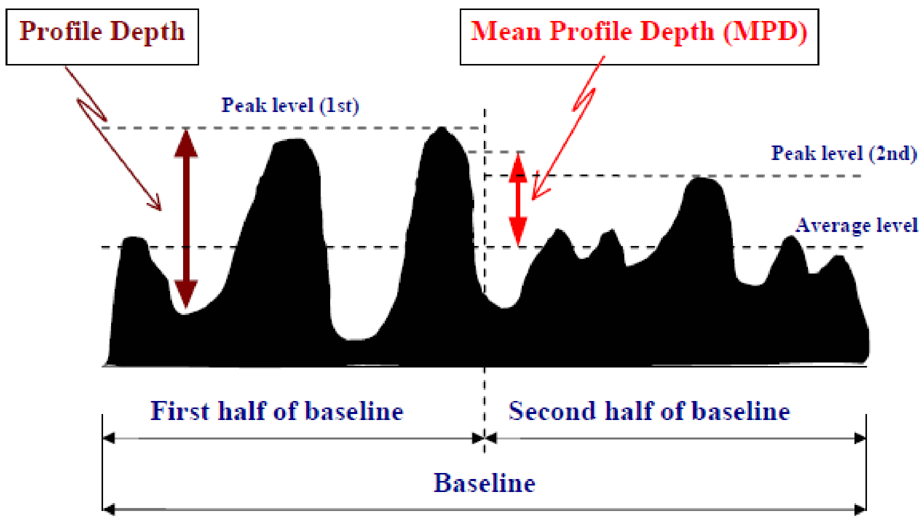


Figure 2.4 Mean Profile Depth Definition

Source: Flintsch, de León, McGhee, and Al-Qadi (2003)

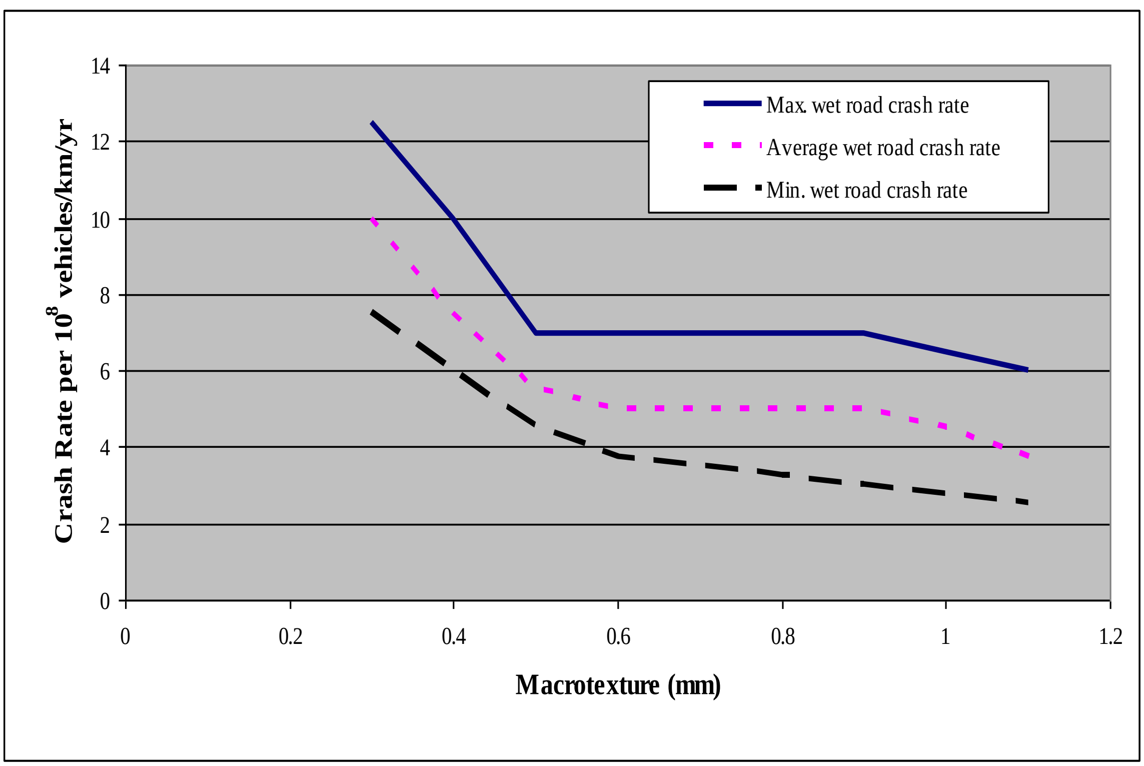


Figure 2. 5 Relationship Between MPD and Crashes

Source: Carney 2006

Carney (1997, 2006) reviewed several studies that agree with his own findings: Roe (1991) studied the relationship between MPD and macrotexture represented by an index termed sensor measured texture depth (SMTD), which is the average depth of the pavement surface macrotexture. MTD varies slightly from MPD. The relationship between

SMTD and MPD is depicted in equation 2.3. According to this equation, an MTD value of 0.5 mm is equivalent to an MPD value of 0.46 mm.

$$SMTD = 0.947*MPD + 0.069 \qquad (2.3)$$

Where:

SMTD = Mean Texture Depth (mm), sensor-measured

MPD = Mean Profile Depth (mm)

The study compared macrotexture at crash sites with macrotexture for the entire road. The number of crashes almost doubled (with respect to rest of the crash sites) when SMTD was less than 0.4 mm.

Two further aspects of this study are very important. First, all crashes were classified into skidding crashes with a wet pavement, skidding crashes with a dry pavement, non-skidding crashes with a wet pavement, and non-skidding crashes with a dry pavement. The relationship between these categories of crash and macrotexture was similar. This suggests that the wet pavement aspect of macrotexture may not be relevant. Second, there was a concern that the observed relationship might have been the result of crashes occurring for other reasons where macrotexture was already low. In order to account for this possibility, crashes where divided between those that occurred near intersections and those that occurred elsewhere. The four macrotexture relationships to crashes were found to be very similar. These findings reinforced the relationship between low macrotexture and crashes (Cairney, 2006).

Gothie (1993) reports a study involving wet-road crashes and macrotexture. The study covered 215 km of national roads in the Alpine region of France with an average daily traffic of approximately 10,000 vehicles. The study included 201 wet-road crashes over a period of almost five years. The crash rate increased considerably when macrotexture dropped below 0.5 mm. The consensus thus appears to be 0.5 mm as a threshold for sharp increase in crash rates.

Skid Resistance

The data TxDOT collected for this study consists of maximum, average, and minimum skid number (SN), peak friction and percent slip for Loop 610 test section, collected with the skid trailer (see Chapter 3 for equipment). As such, this literature review concentrates primarily on criteria to evaluate the skid resistance.

The skid number (SN) is the friction coefficient between pavement and tire, multiplied by 100 (in other words, expressed as a percentage). Detailed discussions about SN can be found in most pavement engineering textbooks. This review concentrates on practical uses of this index in pavement management.

NCHRP 291 (2000) reports a survey of state DOT practices regarding skid resistance. Among the 41 states that responded, 10 had either suggested or formally established minimum acceptable “intervention levels.” The suggested SN thresholds for taking maintenance actions ranged from 28 to 41 for interstates, 25 to 37 for primary roads, and 22 to 37 for secondary roads. Texas reported 30, 26 and 22, respectively. For new construction and surface restoration, minimum values were reported by Maine, Minnesota, Washington, and Wisconsin, varying are from 35 to 45.

Long et al (2014) was the most recent study found that researched the relationship between crash risk and skid resistance using Texas-only data. Based on statistical analyses of 3 years of Texas crash data, they developed criteria for intervention and corresponding thresholds for skid number (SN). Table 2. 2 summarizes the recommendations from this study.

Table 2. 2 Recommended Thresholds for Skid Resistance Improvement

SN Range		**Recommended Action**
All Weather	*Wet Weather*	
SN<14	SN<17	Potential for short term action(s)
14<SN≤28	17<SN≤29	Detailed project-level testing
28<SN≤73	29<SN≤73	Vigilance
SN>73	SN>73	Increasing SN may have little effect on crash rate

Source: Long et al. 2014

Ride Quality

TxDOT provided longitudinal profile data compatible with ProVAL, an FHWA supported software that calculates, summarizes and plots two standard ride quality numbers from raw data: International Roughness Index (IRI) and the Ride Number (RN) (Transtec Group 2015).

The ride number (RN) was developed in the early 1980’s under the National Cooperative Highway Research Program (NCHRP), and later revised and standardized by the University of Michigan for the FHWA. RN predicts the human panel ratings of ride comfort from profile data and is a number between 0 (poor) and 5 (excellent) (Sayers et al., 1998).

The international roughness index (IRI) was developed by the World Bank in the 1980s to capture a standard vehicle’s accumulated suspension motion divided by the distance traveled by the vehicle during the measurement. The originally recommended units were meters per kilometer (m/km) or millimeters per meter (mm/m). IRI is measured in/mi in US customary units. The smaller the IRI, the smoother the pavement. FHWA classifies pavement smoothness based on the IRI as follows:

- Very good: <60 in/m
- Good: 61 to 95 in/mi
- Fair: 96 to 120 in/mi

- Poor: 121 to 170 in/mi
- Very poor: >170 in/mi

Smith et al (2002) measured IRI in 1239 CRCP test sections nationwide in four climatic conditions, finding averages ranging from 82 to 105 in/mi, depending on the climatic region. They also reviewed state agencies' smoothness specifications for concrete pavements acceptance. South Dakota was the only state reporting direct use of the IRI.

The Federal Highway Administration (2015) publication on recommendations for diamond grinding JCP, JRCP, and CRCP surfaces define trigger and limit values for diamond grinding application. Trigger values indicate when a highway agency should consider diamond grinding and rehabilitation to restore rideability. Limit values define the point when it is no longer cost-effective to grind. Table 2. 3 provides examples of trigger and limit values for diamond grinding for CRCP (FHWA 2015).

Table 2. 3 FHWA Recommended Thresholds for CRCP Diamond Grinding

	Value	**Traffic Volumes**		
		High	*Medium*	*Low*
IRI (m/km)	Trigger	1.0	1.2	1.4
	Limit	2.5	3.0	3.5
IRI (in/mi)	Trigger	63	76	90
	Limit	160	190	222

Volumes: High ADT>10,000; Med 3000<ADT<10,000; Low ADT <3,000

Source: FHWA 2015

Conclusions and Recommendations

Noise

- There appears to be a consensus that longitudinal diamond grinding is the best treatment to reduce noise in concrete pavements, that it is also durable and cost-effective. New generation treatments are usually better than the conventional diamond grinding.
- A comprehensive study of PCCP worldwide found that it is reasonable to expect PCCP treated with diamond grinding to stay in the 99/100 to 104/105 dBA range.
- Some studies report highway noise abatement in terms of the equivalent traffic reduction. A small reduction in the number of decibels translates into a significant equivalent traffic reduction.

- The website “www.igga.net/resources/technical-information/noise” is very useful to researchers studying pavement noise. It has links to a plethora of recent studies on the subject.

Texture, Skid Number, and Ride Quality

- NCHRP 291 (2000) reported a comparison between accident rates on dry, wet, and snow/ice conditions, on diamond ground and tined concrete pavements. It found that diamond grinding reduced the crash rate by 42% or both dry and wet pavements, and by 16% under snow or ice.
- Studies reviewed agreed that when MPD falls below the 0.4 to 0.5mm range, the crash rate increases very significantly for all surface conditions (dry, wet, snow and ice)
- This literature review did not find thresholds for macrotexture depths standardized by state DOTs. The UK uses a threshold of 0.5 mm for interventions.
- Studies report a wide range of skid number (SN) thresholds as warranting interventions. Long et al. (2014) are the most recent and also the most comprehensive skid resistance study found that analyzed Texas crash data.
- Ride quality data criteria vary considerably from agency to agency. IRI threshold recommendations were found for Texas as QC/QA recommendations for contractors’ incentives and disincentives. IRI ≥ 95 is the threshold for corrective action.

Chapter 3
Monitoring Plan

This chapter provides an overview of the data collection methodology, schedules, equipment used and test section locations. It covers Tasks 1 and 2, respectively titled "Visit Project Site" and "Develop Monitoring Plan," as well as Products 1 and 2, respectively titled "List of Available Equipment" and "Document Monitoring Plan."

Equipment

On August 20, 2014, José Weissmann, UTSA professor and this study's principal investigator, met with Magdy Mikhail, Director of TxDOT's Pavement Preservation Section, to discuss this project's data collection and equipment availability at TxDOT. It was decided to collect the data with TxDOT in-house equipment operated by TxDOT personnel. The equipment used in this research consisted of:

- Laser texture scanner (LTS) for texture and ride quality (note: see ASTM E2157/ASTM E1845)
- Standard skid trailer (Note: one-channel locked-wheel skid trailer, see ASTM E 274)
- Multipurpose van used for texture and ride quality.
- On-board sound intensity (OBSI) equipment mounted on a vehicle equipped with standard tires as depicted in Figure 3. 1 (Note: see AASHTO TP 76)

Figure 3. 1 TxDOT's Dual-Probe On-Board Sound Intensity

Source: Wirth 2009

Test Sections

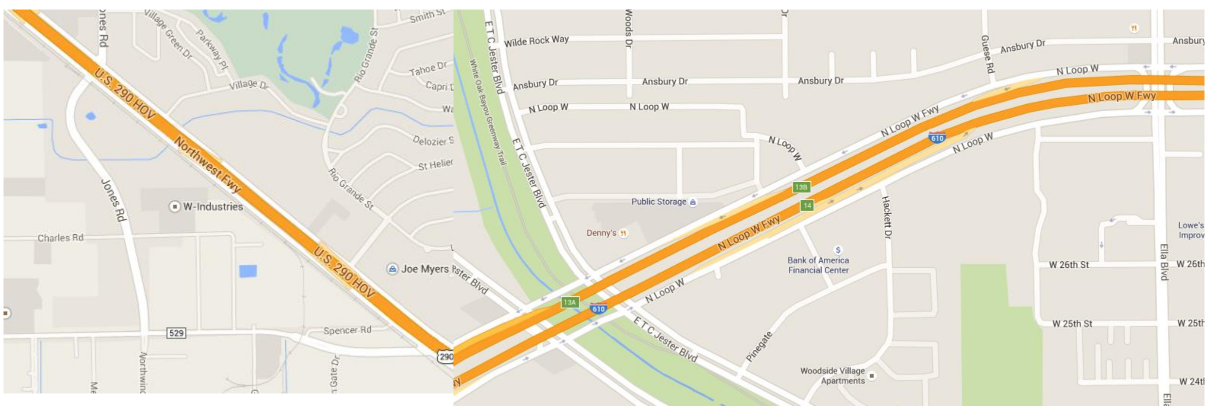
The selection and prioritization of test sections was decided in concert with TxDOT in a September 12, 2014 meeting held in the Houston Area Office located at 14838 Northwest Freeway. The agenda included the following:

- Construction and research schedules.
- Survey sections identification.
- Equipment coordination: OBSI, texture, ride, skid.
- Measurements schedule.

Test section locations and prioritization were a function of construction schedules available at that time, relevance to noise impacts to the community and ability to obtain accurate measurements at a constant speed. The team selected the following locations:

- US 290 between Jones and FM259 (1.2 miles)
- Loop 610 between TC Jester and Ella (0.7 miles)

These test sections were both transversely tinned and were scheduled to undergo several renovations. Their locations are depicted in Figure 3.2.



US290 Loop 610

Figure 3.2 Test Sections Selected at the Beginning of the Project

Source: Google Maps

Texture and ride quality were measured over the entire length of the test sections, on two lanes in each direction. On-board sound intensity (OBSI) test section length is standardized at 440ft (AASHTO 2015). Lanes measured were R1, R2, L1 and L2, designated according to TxDOT's Pavement Management Information System (PMIS) nomenclature. Lanes R1 and R2 are in the north or eastbound traffic direction and lanes L1 and L2, south or westbound. Lanes are numbered from right to left in each traffic direction. Figure 3. 3 illustrates the lane nomenclature.

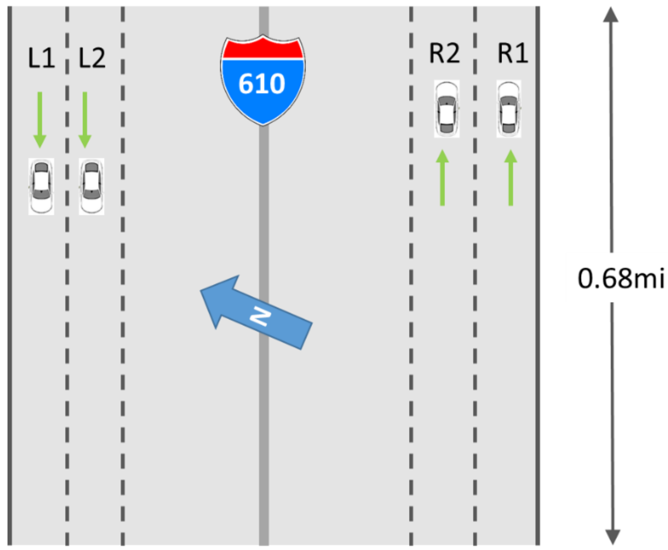


Figure 3. 3 Lane Designations on Loop 610

There were three 440-ft segments on each lane, designated as follows:

- Segments A, B, C: lane R1 (rightmost lane, northbound on Loop 610, eastbound on US290).
- Segments AA, BB CC: lane R2 (second-to-right lane, northbound on Loop 610, eastbound on US290).
- Segments D, E, F: lane L1 (rightmost lane, southbound on Loop 610, westbound on US290)
- Segments DD, EE, FF: L2 (second-to-right lane, southbound on Loop 610, westbound on US290)

Figure 3.4 Example of Sound Test Sections on Lane R1

Picture source: Google Earth

Table 3.1 and Table 3. 2 show the coordinates and the landmark description of the sound segments on both lanes of each test section. Figure 3.4 helps visualize the sound segments in each test section, using as example lane R1 of Loop 610. The segment starting points were located based on the coordinates depicted in Table 3.1, which were a bit off and had to be adjusted based on comparing TxDOT's picture and description of the landmark with the picture from Google Earth Tour. Figure 3.5 illustrates the comparison used to refine the segment location. All sound segments were marked on Google Earth in this manner on the rightmost lanes (R1 and L1). Segments on the "2" lanes are parallel with matching starting and ending points.

Field Picture, November 2014

Google Earth Tour Picture, 2015

Figure 3.5 Field Landmarks: Field Picture and Google Earth's View
Location: Start of Segment A

Table 3.1 Loop 610 Sound Segments on Lanes R1 and L1

Test Section	Segment	Starting Point		Field landmark description
		Latitude	**Longitude**	
Northbound TC Jester to Ella	A	29°48'33.95"N	95°26'20.96"W	Just after the bridge deck pavement change, used a right exit arrow as reference point
	B	29°48'37.87"N	95°26'12.25"W	Full roadbed width overhead sign, 3 green signs
	C	29°48'41.22"N	95°26'04.40"W	End of on-ramp bridge Girders, start of Concrete retaining wall
Southbound Ella to TC Jester	D	29°48'46.49"N	95°25'49.45"W	Barrel before lamp post after road straightens
	E	29°48'43.65"N	95°26'02.62"W	1 Lane Green overhead Exit Sign
	F	29°48'40.54"N	95°26'09.72"W	Full roadbed width 3 green overhead signs

Source: TxDOT

Table 3. 2 US290 Sound Segments on Lanes L1 and R1

Test Section	Segment	Starting Point		Field Landmark
		Latitude	Longitude	
Westbound FM 529 to Jones Rd	A	29°52'58.36"N	95°34'24.57"W	Exit Sign for Exit to Jones Rd
	B	29°53'3.77"N	95°34'32.26"W	Merge Left - Yellow Caution Sign
	C	29°53'10.37"N	95°34'41.76"W	US 290 Sign on Frontage Rd
Eastbound Jones Rd to FM 529	D	29°53'16.06"N	95°34'51.59"W	Jones Rd to FM 529 EB
	E	29°53'6.30"N	95°34'37.88"W	1 Lane Green overhead, FM 529 - Bltwy 8 Frntg - Senate Ave, Exit 1/2 mile Sign
	F	29°53'1.37"N	95°34'30.68"W	1 Lane Green overhead, Sam Houston Tollway, Exit 1 mile Sign

Source: TxDOT

Figure 3.6 shows the data structure: two lanes per test section, and three sound segments per lane. The analysis data base is also organized with the same data hierarchy using the same lane and segment nomenclature. This structure was used for all three data runs: before construction, 3 months after completion, and 6 months after completion.

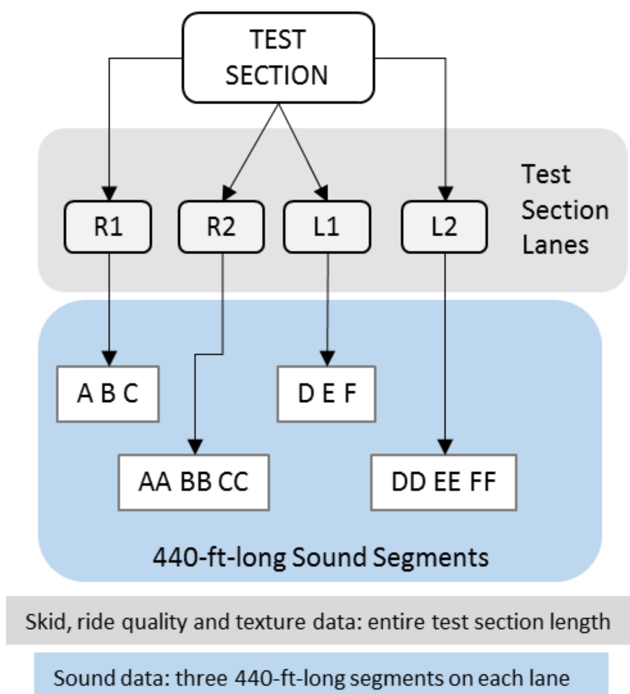


Figure 3.6 Data Structure

Data Collected

Summary of Data Collected

The ideal PCCP surface treatment would provide a quiet surface, a smooth ride and a high friction (skid resistance). However, while a smooth ride requires a surface, the skid resistance requires high friction (rough surface). Therefore, PCCP must achieve a balance between rough enough for safety (skid resistance), smooth enough ride comfort. A quiet concrete pavement requires predominantly negative texture (no peaks) among other characteristics. Moreover, as the surface treatment wears off, it loses skid resistance, so durability is also a concern. Given these facts, following data were collected by TxDOT and provided to UTSA for analysis:

- <u>Texture</u>: mean profile depth (MPD) and estimated profile depth (EDT) recorded every 1.8 ft over the entire length of both lanes on each traffic direction (measured in mm).
- <u>Skid number</u>: files with the minimum, average, and maximum skid numbers, peak and percent slip recorded every 0.05 mi over the entire length of both lanes on each traffic direction.
- <u>Ride quality</u>: a raw data file compatible with ProVAL containing data recorded over the entire length of the test section lanes on each traffic direction. These data were processed with ProVAL to obtain the international roughness index (IRI). ProVAL is an engineering application that allows users to view and analyze pavement profiles. ProVAL was developed by the Transtec Group for FHWA/LTPP, originally released in 2001 and periodically updated (Transtec Group, 2015).
- <u>On-board sound intensity</u> (OBSI). Excel files containing 1/3 octave 1/24 octave (narrow band) sound pressure and sound intensity levels at the trailing and leading microphones. 3 runs per sound segment. The excel files also contain data summaries by test segment.

Data Collection Schedule and Data Obtained

Based on construction schedules and equipment availability anticipated in 2014, it was decided to take four sets of measurements: pre-NGDG (next generation diamond grinding), immediately after opening to traffic, 2 months after and 4 months after. Due to subsequent changes in the construction schedule, US290 was not completed in time for this project and the comparative analyses were performed for Loop 610. Moreover, TxDOT equipment and personnel availability issues precluded measurements immediately after opening to traffic. The final measurement schedule is listed below. TxDOT delivered the data between 1 and 3 weeks after the collection date.

1. Pre-NGDG: November 4 and 5, 2014, on US290 and Loop 610.
2. Post-NGDG 3 months after construction completion (Loop 610 only):
    - Skid: March 10, 2016

- Sound: March 21, 2016
- Texture: March 28, 2016

3. Post-NGDG 6 months after construction completion (Loop 610 only):
    - Skid: March 10, 2016
    - Sound: March 21, 2016
    - Texture: March 28, 2016

Conclusion

Despite changes in construction schedules and in TxDOT equipment and personnel availability, this project collected and analyzed enough data to meaningfully compare pre- and post-NGDG performance on Loop 610. Mid- and long-term performance, however, would require annual measurements starting one year after opening to traffic.

Chapter 4
Comparative Analysis: Skid Resistance, Ride Quality and Macrotexture

This chapter discusses the analysis of the skid, macrotexture and ride quality data collected during this project, explaining the data analysis methodology and presenting the results, conclusions and recommendations. This chapter covers Tasks 3 and 4, respectively titled "Perform Pre- and Post-Diamond Grinding Measurements," and "Perform Mid-Range Post-Diamond Grinding Measurements" for all except the sound data, which is documented in the Chapter 5.

As previously stated, Loop 610 data were collected on three occasions: before construction (pre-NGDG), and 3 and 6 months after completion, respectively termed in this chapter as “post-NGDC-3mo” and “post-NGDG-6mo.” Pre-NGDG data is also available for US290. Since no post-NGDG data exists for US290 due to changes in construction schedules, a comparative analysis was possible only for Loop 610.

Skid Resistance

Available Data

As explained in Chapter 3, TxDOT used ASTM E274-06 skid trailer with a reported speed averaging 50mph. There are 13 data points for each lane, at 0.05-mile-long intervals, on 2 lanes in each direction (lanes R1, R2, L1 and L2). Data points are numbered from 0 to 12. Test 12 was missing for lane R1 in the pre-NGDG data. In the analysis, it was substituted for the lane average in order to obtain a complete factorial.

Skid data was provided as minimum, average and maximum skid numbers (SN), peak value and slip percentage for each of the 13 points. Temperature was also provided. Average test temperatures were:

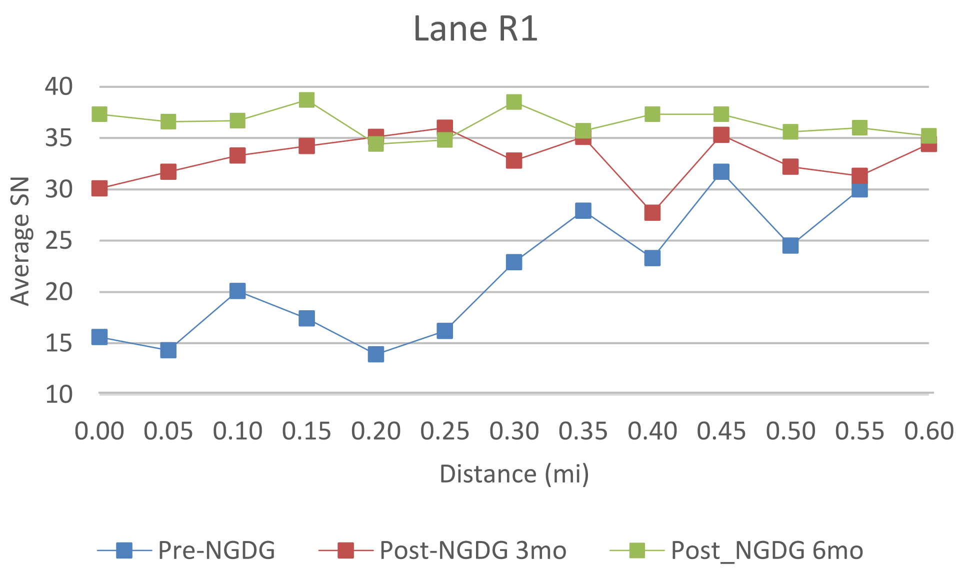
Pre-NGDG............................87.1^{o}F

Post-NGDG-3mo..................75.9^{o}F

Post-NGDG-6mo................111.5^{o}F

Preliminary Analysis

The initial steps in the preliminary analysis were to visually inspect the data then verify the need to correct SN for seasonal variations. Figure 4. 1 (lanes R1 and R2) and Figure 4. 2 (lanes L1 and L2) compare the average SN for the three data collection efforts. Improvement is quite obvious; all lanes except R2 showed considerable improvement after NDGD. It is noteworthy that pre-NGDG R2 measurements were better than those of the other lanes. Appendix 1 contains the full set of comparative plots of all SN data obtained for Loop 610.



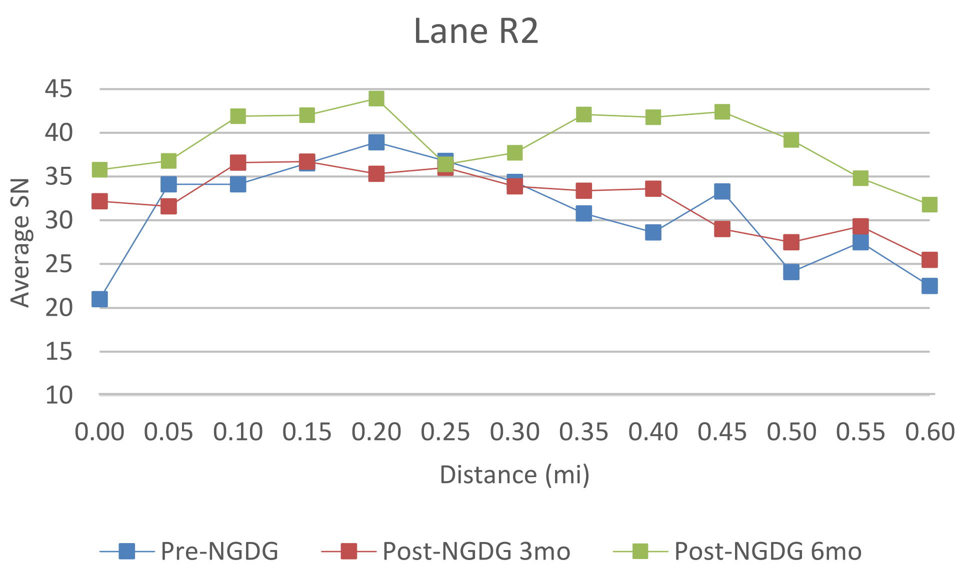


Figure 4. 1 Comparison among Average SNs— Northbound Loop 610

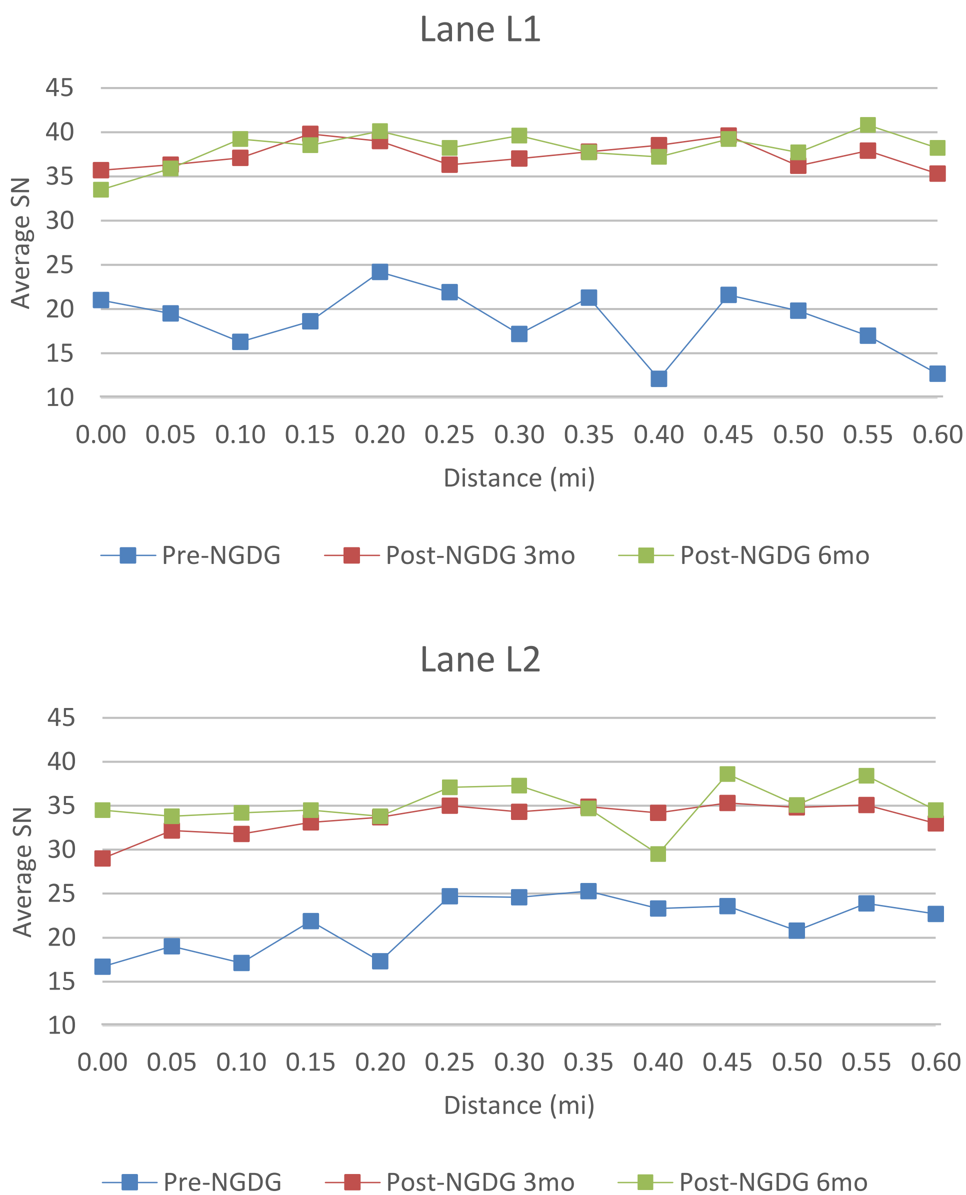
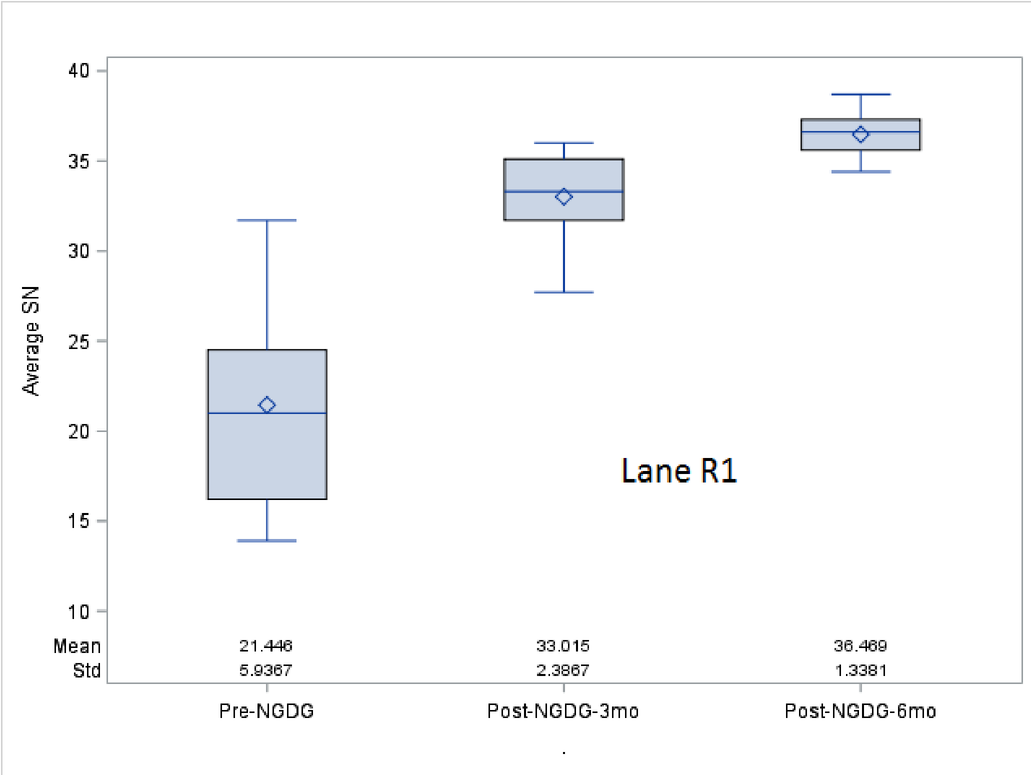


Figure 4. 2 Comparison among Average SNs — Southbound Loop 610

Figure 4. 3 and Figure 4. 4 depict the boxplots (or box-and-whiskers plots) of the average SN, respectively for north and southbound Loop 610. The top whisker's endpoints represent the minimum and maximum values; the lower and upper edges of the box are the first and third quartile; the line inside box is the median (second quartile), and the symbol marker is the mean. The dots are data outliers.



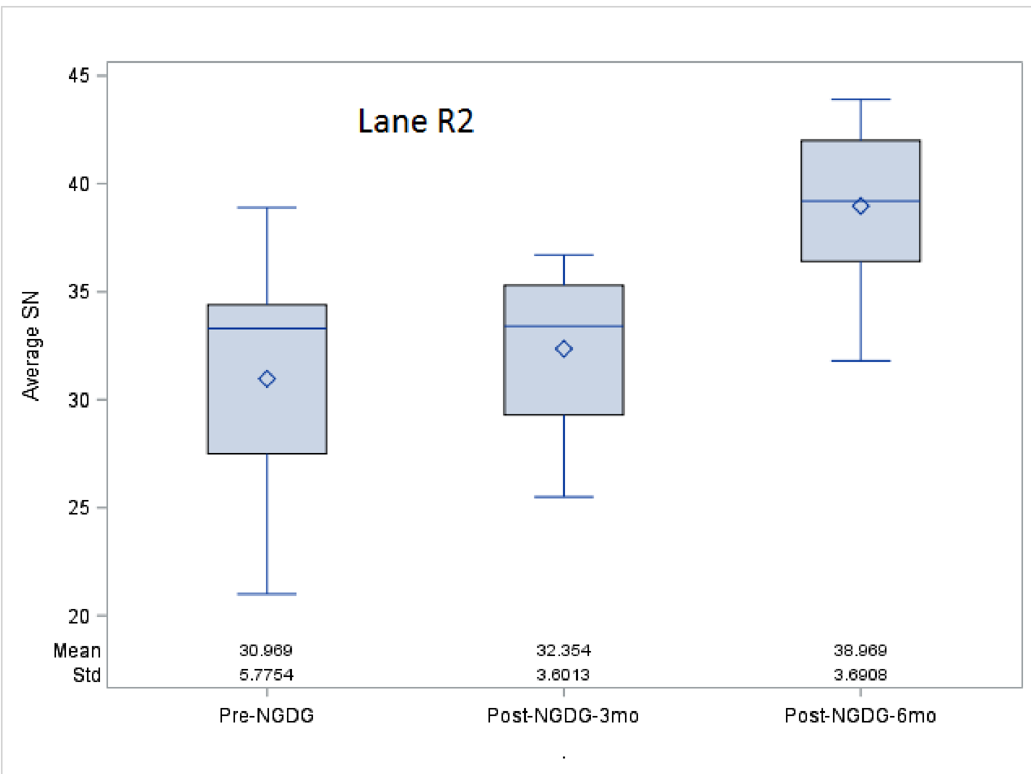


Figure 4. 3 Boxplots of Average SNs — Loop 610 Northbound

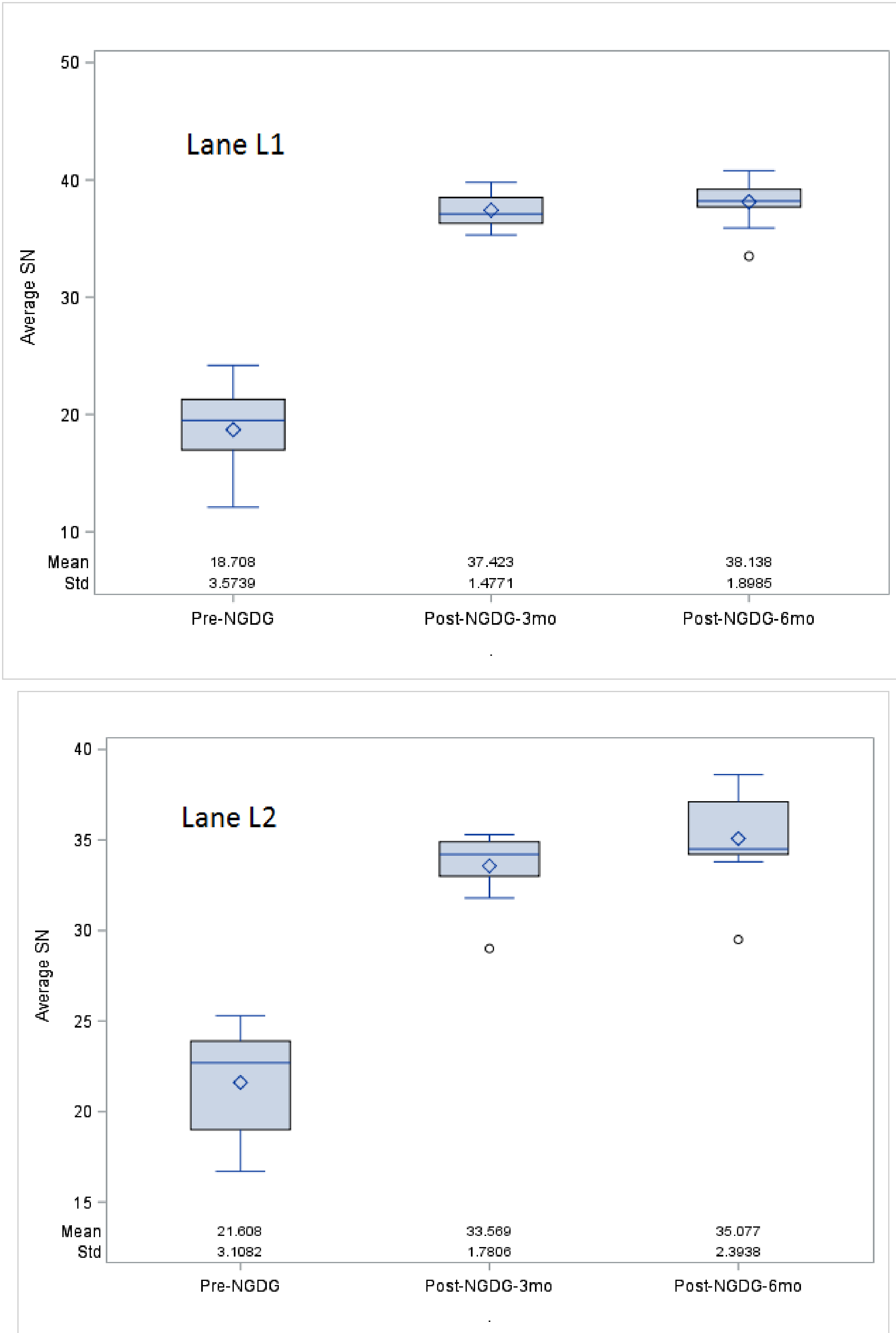


Figure 4. 4 Boxplots of Average SNs — Loop 610 Southbound

Desirable results would show both post-NGDG boxes located above the pre-NGDG box; the further above, the better. Ideal results for post-NGDG 3 and 6 month data set would have post-NGDG boxes positioned above the pre-NGDG top whisker (minimum post-NGDG value greater than the maximum pre-NGDG); and also have short boxes (small random variations within each data set) located at approximately the same height in the graph (no change after 3 months of traffic). Combined inspection of Figure 4. 1 through Figure 4. 4 indicate that post-NGDG results were desirable for lanes R1 and R2, and ideal for lanes L1 and L2. These preliminary conclusions were verified with the statistical tests discussed later.

The literature usually reports a drop in SN when the pavement is hot (Shahin 2005, Burchett et al. 1979). It also reports more significant SN seasonal variations in flexible pavements than in concrete pavements. For example, Burchett et al (1979) studied SN seasonal variations in Kentucky, finding a maximum SN change of 5 from summer to winter in concrete pavements. Five is the magnitude of the overall standard error of the SN data collected in this project. In Kentucky, the temperature varies approximately 50^{o}F between summer and winter. For this project data, the highest variation was 35^{o}F. Figure 4. 1 and Figure 4. 2 indicate that the 6-month post-NGDG SN is on the average higher than the 3-month post-NDGD SN, while the literature reports that SN usually decreases as the pavement temperature increases. Therefore, there are no detectable SN seasonal variations in these data. Adjusting SN for seasonal variations with models found in the literature would only add modeling errors to the intrinsic SN random variations.

The third step in the preliminary analysis consisted of checking if the data could be aggregated by traffic direction before performing the comparisons. This was done with homogeneity tests, which check whether or not two data sets come from the same population. Complete homogeneity tests were not necessary. Testing performed early in this project, when only pre-NGDG and post-NGDG-3mo data were available, already indicated that SN data must be compared for each lane individually, as documented in the results discussed below.

Table 4. 1 shows the results of the two homogeneity tests that compare pre-NGDG to post-NGDG-3mo data. The Wilcoxon test is based on a normal approximation, while Kruskal-Wallis is a non-parametric test and as such does not rely on assumptions about the distribution of the underlying populations.

Table 4. 1 results are the significance levels of the tests, reported as percentages and interpreted in this table as an answer to the question of whether or not the data should be pooled by traffic direction. The percentages correspond to the probability of being wrong when assuming that data for parallel lanes come from different populations (i.e., assuming that data cannot be aggregated by traffic direction). The maximum acceptable significance level of a statistical test is 5%. As depicted in Table 4. 1, the data could be pooled in some individual cases, but never for the same set of parallel lanes in both the pre- and the post-NGDG data. Conclusion: the comparative analysis must be made for each traffic lane individually.

Table 4. 1 Homogeneity Tests of Parallel Lanes SN

PRE-NGDG	*Min*		*Average*		*Max*		*Peak*	
Question	R1=R2?	L1=L2?	R1=R2?	L1=L2?	R1=R2?	L1=L2?	R1=R2?	L1=L2?
Wilcoxon normal approximation	1.09%	16.94%	0.25%	4.28%	0.16%	5.07%	0.04%	11.19%
Kruskal-Wallis	0.53%	14.95%	0.23%	4.02%	0.14%	4.70%	0.03%	10.62%
Pool the before-NGDG data?	NO	YES	NO	NO	NO	BL*	NO	YES
	NO	YES	NO	NO	NO	BL*	NO	YES

POST-NGDG 3 Months	*Min*		*Average*		*Max*		*Peak*	
Question	R1=R2?	L1=L2?	R1=R2?	L1=L2?	R1=R2?	L1=L2?	R1=R2?	L1=L2?
Wilcoxon normal approximation	75.69%	0.09%	85.75%	<0.001%	77.64%	0.32%	100%	0.12%
Kruskal-Wallis	69.77%	0.02%	83.74%	<0.001%	75.42%	0.10%	97.95%	0.02%
Pool the after-NGDG data?	YES	NO	YES	NO	YES	NO	YES	YES
	YES	NO	YES	NO	YES	NO	YES	NO

* BL=borderline, yes at 5% significance, no at 1%

Comparative Analysis

The first part of this comparative checked if there was a change in the post-NGDG data after 3 months of traffic wear and tear. The expected finding would be "no;" concrete surfaces do not wear out that quickly. If the “no” holds, the comparative analysis becomes pre-NGDG data versus the average of 3 and 6 month post-NGDG data points. Interestingly enough, Figure 4. 1 through Figure 4. 4 suggest increases in SN after 3 months, especially for the northbound direction (lanes R1 and R2).

The statistical significance of this apparent difference was checked for all lanes using the Wilcoxon and Kruskal-Wallis tests previously described. Table 4. 2 depicts the results. In this case, the percentages (significance levels) should be interpreted as the probability of being wrong when assuming a difference between 3 and 6-month post-NGDG data. In the table, results were translated into the answer to the question of whether or not the 3- and 6-month post-NGDG data could be aggregated.

For the southbound direction (L1 and L2), the 3 and 6-month post-NGDG SNs were statistically the same in all cases. The only exception was the percent slip on both L1 and L2. Conclusions: (1) Average the 3 and 6-month post-NGDG data sets and compare post-NGDG to pre-NGDG for all SN data. (2) Perform separate comparisons for the percent slip.

For the northbound direction (R1 and R2), only the peak value and the percent slip were statistically the same for 3 and 6 month data. All other values were different, and visual inspection of the data (see Appendix 1) indicates that skid resistance appears consistently better after 3 months of traffic. This counterintuitive result is probably due to unknown external factors influencing post-NGDG data collection. This influence should be minimized. Conclusions: average 3- and 6-month post-NGDG Sn and peak value data in all cases for the northbound direction before comparing to pre-NGDG. Investigate percent slip separately in order to verify the discrepancy in homogeneity.

Table 4. 2 Homogeneity Test for 3 and 6 Month Post-NGDG Skid Resistance

Question:
Pool the 3-month and 6-month post-NGDG data?

Minimum SN	**R1**	**R2**	**L1**	**L2**
Wilcoxon normal approximation	0.74%	0.31%	19.75%	8.35%
Kruskal-Wallis	0.33%	0.10%	17.71%	6.75%
Answer:	NO	NO	YES	YES
Average SN	**R1**	**R2**	**L1**	**L2**
Wilcoxon normal approximation	0.12%	0.21%	21.11%	13.60%
Kruskal-Wallis	0.02%	0.05%	19.05%	11.74%
Answer:	NO	NO	YES	YES
Maximum SN	**R1**	**R2**	**L1**	**L2**
Wilcoxon normal approximation	0.34%	0.15%	100.00%	73.73%
Kruskal-Wallis	0.11%	0.03%	97.00%	71.49%
Answer:	NO	NO	YES	YES
Peak	**R1**	**R2**	**L1**	**L2**
Wilcoxon normal approximation	47.92%	9.23%	5.82%	7.60%
Kruskal-Wallis	45.69%	7.57%	5.37%	6.06%
Answer:	YES	YES	YES	YES
Percent Slip	**R1**	**R2**	**L1**	**L2**
Wilcoxon normal approximation	4.81%	11.18%	0.97%	0.56%
Kruskal-Wallis	3.53%	9.42%	0.47%	0.22%
Answer:	NO	YES	NO	NO

Pre and post-NGDG values as well as "post-pre" differences (termed "ΔSN," Δpeak "ΔPctSlip" in all tables and graphs) were quantified as 95% confidence intervals for the SN, peak and percent slip values and for their paired differences (Δ). In addition, statistical tests of significance were performed to verify if the differences were greater than zero for skid and less than zero for the percent slip.

Table 4. 3 depicts the results of tests that verify if ΔSNs>0, Δpeak>0 and ΔPctSlip<0. The significance levels (percentages) in this table should be interpreted as the probability of being wrong when assuming that the answer to the question in the second column is yes. Results that answer "no" are depicted in red.

Table 4. 3 Significance Levels of Tests for Post-NGDG SN Increase and Percent Slip Decrease

Data	Question	R1	R2	L1	L2
Min SN	**ΔSN_{min}>0 ?**	<0.01%	<0.01%	<0.01%	<0.01%
Avg SN	**ΔSN_{avg}>0 ?**	<0.01%	1.87%	<0.01%	<0.01%
Max SN	**ΔSN_{max}>0 ?**	<0.01%	30.07%	<0.01%	<0.01%
Peak	**Δpeak>0 ?**	<0.01%	34.57%	<0.01%	<0.01%
Pct. Slip	**ΔPctSlip<0 ?**	0.02%	20.78%	N/A	N/A
	3mo-pre<0 ?	0.03%	2.27%	0.08%	0.13%
	6mo-pre<0 ?	0.86%	43.52%	30.64%	35.49%

*Post-NGDG data are the average of 3 and 6 month

Black font = yes; red font = no

The skid paired differences were highly significant in all lanes, except ΔSN_{max} and Δpeak in lane R2, which had better pre-NGDG skid resistance than all other lanes. The percent slip showed a statistically significant decrease for the data collected 3 months after NGDG but not for 6 months (except lane R1). The most important conclusion is that the average SN increase was statistically significant in all cases.

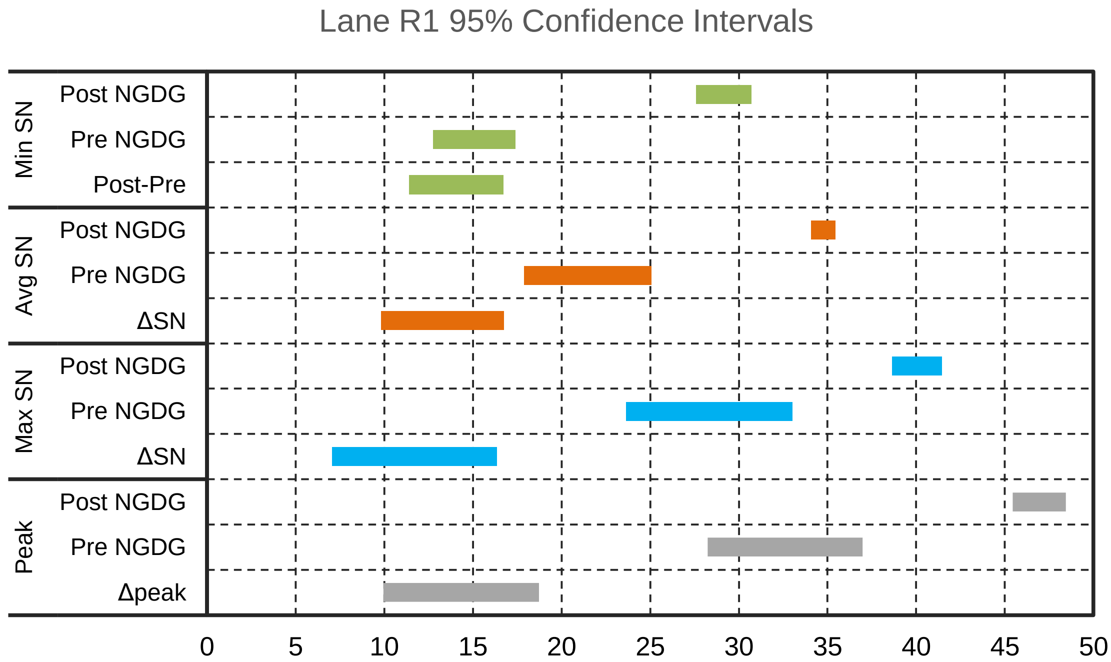
Table 4. 4 shows all 95% confidence interval results for the mean and the standard deviation of the 13 data points for each variable (minimum SN, average SN, etc.) and also for the mean of the 13 paired differences (ΔSN) between post-NGDG (3 and 6 months averaged) and pre-NGDG data.

Figure 4. 5 (northbound direction) and Figure 4. 6 (southbound direction) show the confidence intervals plotted to scale to facilitate visualization. The bars represent the width of the 95% confidence interval, starting at the lower limit and ending at the upper limit, in the horizontal axis scale.

Table 4. 4 95% Confidence Intervals for the Means of SN and ΔSN

Lane	Variable	Data	95% CI Lower	Mean	95% CI Upper	95% CI Lower	Std Dev	95% CI Upper
R1	Min SN	Post NGDG	27.6	29.2	30.7	1.9	2.6	4.3
	Min SN	Pre NGDG	12.8	15.1	17.4	2.8	3.8	6.3
	ΔSNmin	Post-Pre	11.4	14.1	16.7	2.6	3.3	4.6
R1	Avg SN	Post NGDG	34.1	34.8	35.4	0.8	1.1	1.9
	Avg SN	Pre NGDG	17.9	21.5	25.1	4.3	5.9	9.8
	ΔSNavg	Post-Pre	9.8	13.3	16.7	3.3	4.3	5.9
R1	Max SN	Post NGDG	38.6	40.0	41.4	1.7	2.3	3.8
	Max SN	Pre NGDG	23.6	28.3	33.0	5.6	7.8	12.8
	ΔSNmax	Post-Pre	7.1	11.7	16.4	4.5	5.7	8.0
R1	Peak	Post NGDG	45.4	46.9	48.4	1.8	2.5	4.1
	Peak	Pre NGDG	28.2	32.6	37.0	5.2	7.2	11.9
	Δpeak	Post-Pre	10.0	14.3	18.7	4.2	5.4	7.5

Lane	Variable	Data	95% CI Lower	Mean	95% CI Upper	95% CI Lower	Std Dev	95% CI Upper
R2	Min SN	Post NGDG	28.8	30.5	32.3	2.1	2.9	4.8
	Min SN	Pre NGDG	18.7	22.2	25.6	4.1	5.7	9.5
	ΔSNmin	Post-Pre	4.7	8.4	12.1	3.5	4.5	6.3
R2	Avg SN	Post NGDG	33.8	35.7	37.6	2.3	3.2	5.3
	Avg SN	Pre NGDG	27.5	31.0	34.5	4.1	5.8	9.5
	ΔSNavg	Post-Pre	0.9	4.7	8.5	3.6	4.7	6.5
R2	Max SN	Post NGDG	39.5	41.8	44.2	2.8	3.9	6.5
	Max SN	Pre NGDG	35.7	39.6	43.5	4.6	6.5	10.7
	ΔSNmax	Post-Pre	-2.1	2.2	6.6	4.2	5.4	7.4
R2	Peak	Post NGDG	46.7	49.3	51.9	3.1	4.3	7.1
	Peak	Pre NGDG	45.7	52.7	59.7	8.3	11.6	19.1
	Δpeak	Post-Pre	-10.4	-3.3	3.7	6.8	8.7	12.2
L1	Min SN	Post NGDG	32.7	33.7	34.7	1.2	1.6	2.7
	Min SN	Pre NGDG	11.1	12.8	14.5	2.0	2.8	4.6
	ΔSNmin	Post-Pre	19.1	21.0	22.8	1.8	2.3	3.2
L1	Avg SN	Post NGDG	36.9	37.8	38.7	1.1	1.5	2.4
	Avg SN	Pre NGDG	16.5	18.7	20.9	2.6	3.6	5.9
	ΔSNavg	Post-Pre	16.9	19.1	21.3	2.1	2.7	3.8
L1	Max SN	Post NGDG	41.1	42.4	43.6	1.5	2.1	3.4
	Max SN	Pre NGDG	22.0	27.3	32.7	6.3	8.8	14.6
	ΔSNmax	Post-Pre	9.9	15.1	20.3	5.0	6.4	8.9
L1	Peak	Post NGDG	49.5	51.0	52.4	1.7	2.4	4.0
	Peak	Pre NGDG	31.1	37.5	43.9	7.6	10.6	17.6
	Δpeak	Post-Pre	7.2	13.4	19.7	6.0	7.7	10.7
L2	Min SN	Post NGDG	25.1	28.3	31.5	3.8	5.3	8.8
	Min SN	Pre NGDG	12.6	14.4	16.1	2.1	2.9	4.8
	ΔSNmin	Post-Pre	10.5	13.9	17.4	3.3	4.3	6.0
L2	Avg SN	Post NGDG	33.3	34.4	35.4	1.2	1.7	2.9
	Avg SN	Pre NGDG	19.7	21.6	23.5	2.2	3.1	5.1
	ΔSNavg	Post-Pre	10.7	12.7	14.8	2.0	2.5	3.5
L2	Max SN	Post NGDG	37.5	38.6	39.7	1.3	1.8	3.0
	Max SN	Pre NGDG	28.5	32.2	36.0	4.5	6.2	10.3
	ΔSNmax	Post-Pre	2.7	6.4	10.1	3.6	4.6	6.4
L2	Peak	Post NGDG	45.8	46.9	48.1	1.3	1.9	3.1
	Peak	Pre NGDG	37.5	43.4	49.3	7.0	9.8	16.1
	Δpeak	Post-Pre	-2.2	3.5	9.2	5.5	7.0	9.8



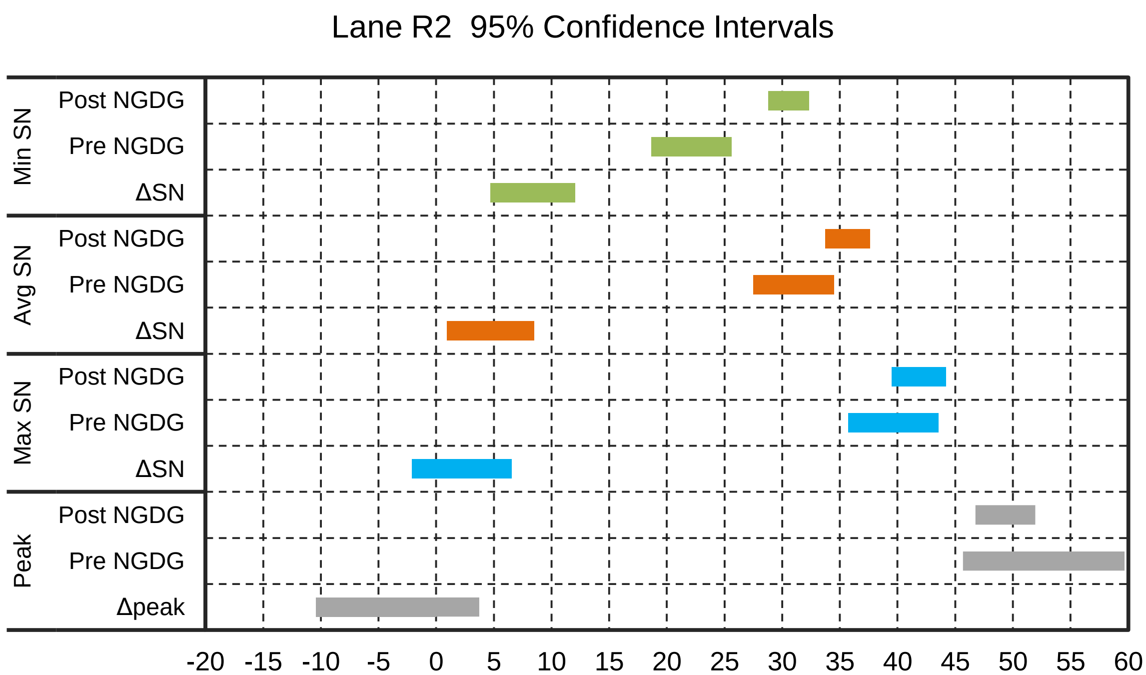
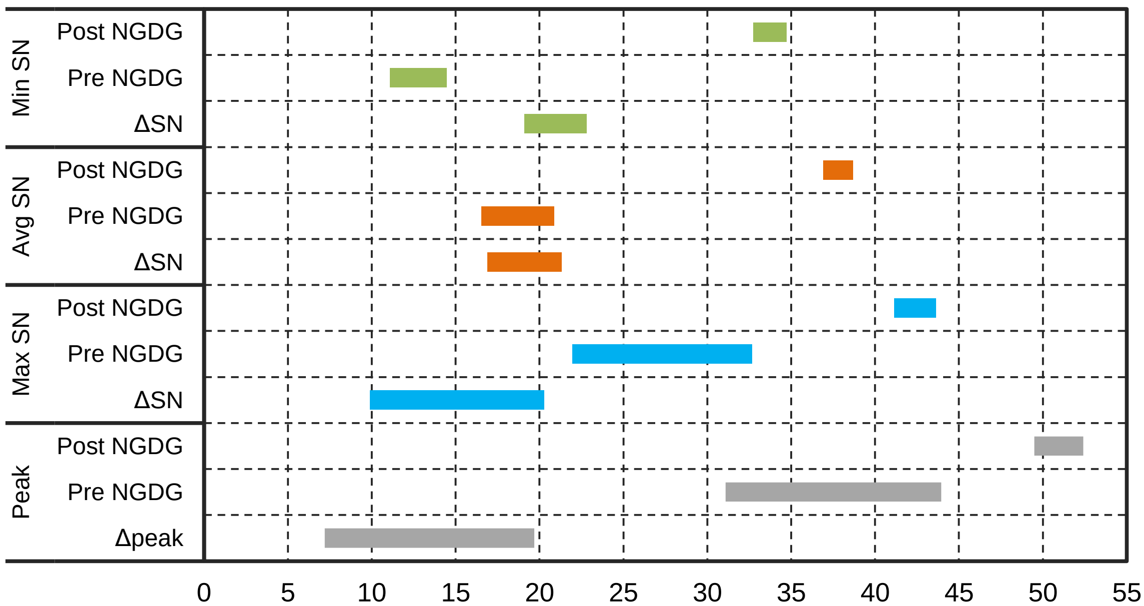


Figure 4. 5 Northbound Loop 610: 95% Confidence Intervals

Lane L1 95% Confidence Intervals



Lane L2 95% Confidence Intervals

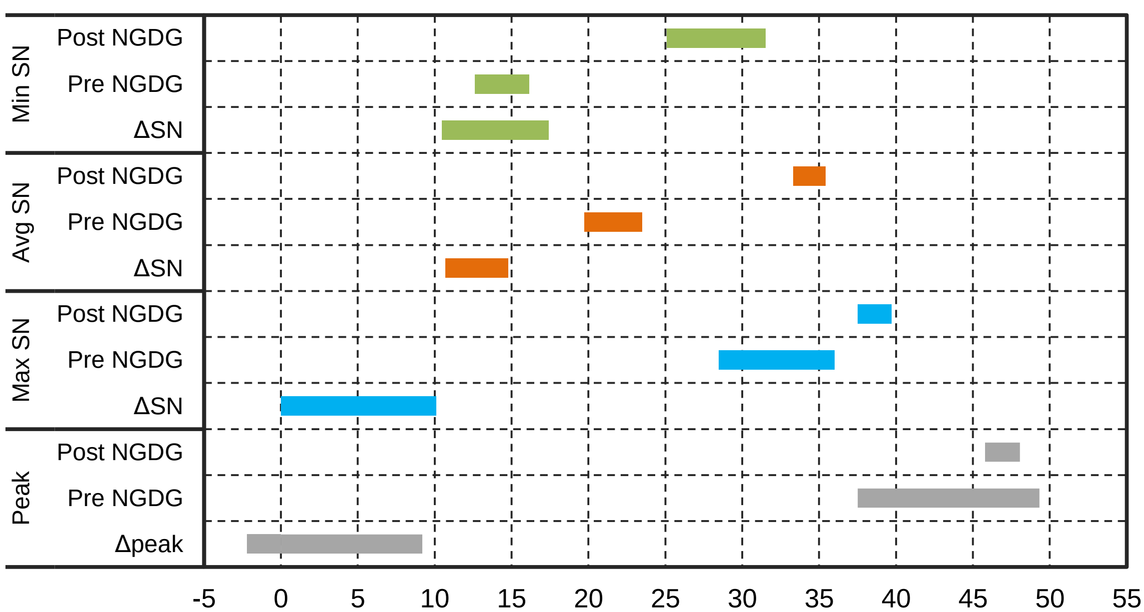


Figure 4. 6 Southbound Loop 610: 95% Confidence Intervals

Below is a summary to help interpret the confidence intervals in Figure 4. 5 and in Figure 4. 6. Good results would have the following characteristics:

- Post-NGDG confidence intervals should always be to the right of the pre-NGDG ones (larger post-NGDG values).
- No overlap between pre- and post-NGDG confidence intervals, i.e., a post-NGDG confidence interval lower limit greater than the upper limit of the pre-NGSG interval. This means all post-NGDG values are greater than all pre-NDGD.
- Confidence intervals for the post-pre differences (Δs) have positive means and preferably a positive lower limit as well.
- The further to the right the confidence interval for the post-pre differences (Δ), the better; this indicates greater improvements (large post-pre differences).
- The narrower the bar (confidence interval size), the better; this indicates less random variations, which means a uniform NGDG surface.

In the northbound direction, post-NGDG results improved in all cases for lane R1: Figure 4. 5 indicates that confidence intervals of all post-NGDG variables are located to the right of the pre-NGDG data, and the confidence intervals for all differences are positive. In other words, 95% of all possible differences are positive, i.e., showed an improvement. For lane R2, the minimum and average SN values improved; post-NGDG confidence intervals are to the right of pre-NGDG, and the confidence intervals for the differences are positive. For maximum SN, post-NGDG confidence intervals show only a slight shift to the right of pre-NGDG, and the ΔSN confidence interval is not entirely positive. The peak value shows little difference between pre- and post-NGDG. Lane R2 was the only lane with high pre-NGDG skid values.

In the southbound direction, there were very significant improvements in both lanes for all variables, as shown in Figure 4. 6. The post-NGDG confidence intervals are shifted entirely to the right of the pre-NGDG confidence intervals for all variables. The average ΔSN confidence interval is not only entirely positive, but also is approximately twice the pre-NGDG. Maximum SN and peak also show impressive improvement, with both post-NGDG confidence intervals showing a considerable shift to right of the corresponding pre-NGDG intervals. All confidence intervals for the post-pre differences are positive and have rather large upper limits. Lane L2 shows similar, albeit less considerable, improvements in all SN variables except the peak value. Even in the peak value case, results are not perfect but are still very good. The post-NGDG confidence interval is slightly shifted to right of pre-NGDG and the lower limit of the differences is, in absolute value, much smaller than its upper limit.

Figure 4. 7 compares the distribution of observed percent slip for all lanes and all three data collection dates, in boxplot format. Although not all differences between pre- and post-NGDG were statistically significant (see Table 4. 3), the post-NGDG means were less than the post-NGDG in all cases. The 3-month post-NGDG shows a significant improvement, while the

6-month data shows minor improvement. This is not consistent the overall SN improvement discussed above.

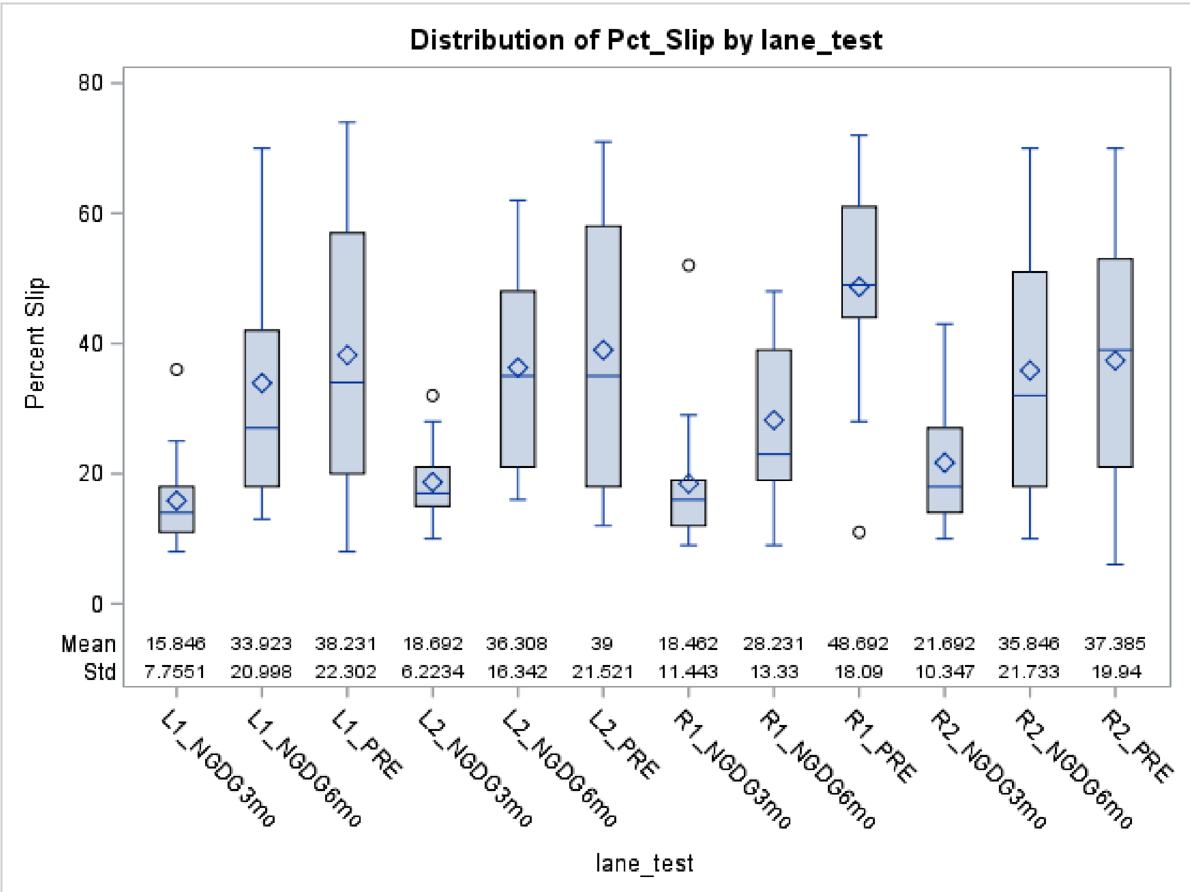


Figure 4. 7 Percent Slip Boxplots, All Lanes

Table 4. 5 summarizes the percent changes in SN, peak value and percent slip. Each row shows the percent change between the mean of 13 pre-NGDG data points and the mean of 26 post-NGDG data points, for each variable. All variables improved except peak SN for lane R2. The best improvement in average SN was observed in lane L1 (102.1%), and the smallest for lane R2 (15.2%). The overall improvement (all lanes) in average SN was 59.5%, and the overall percent slip (all lanes) dropped almost 35%.

Table 4. 5 Percent Change in SN, Peak Value and Percent Slip (post/pre)

Data	L1	L2	R1	R2	*Lane Average by Data*
Min SN	164.2%	96.8%	93.3%	37.8%	*98.0%*
Avg SN	102.1%	59.0%	61.8%	15.2%	*59.5%*
Max SN	55.2%	19.8%	41.3%	5.6%	*30.5%*
Peak	35.8%	8.1%	44.0%	-6.3%	*20.4%*
Pct. Slip	-34.9%	-29.5%	-52.0%	-23.0%	*-34.9%*
Data Average by Lane	*64.5%*	*30.8%*	*37.7%*	*5.9%*	*34.7%*

Summary of Conclusions

NGDG significantly improved the overall skid resistance of the NGDG pavement. Overall improvements for the aggregated data (all lanes) were:

- 95% confidence interval for the average pre-NGDG SN: 18.7 ± 2.2
- 95% confidence interval for the average post-NGDG SN: 33.7 ± 1.0
- Average SN improvement: 59.5%
- Smallest improvement: Lane R2, 15.2%
- Greatest improvement: Lane L1, 102.1%
- Overall percent slip improvement: 35%

Ride Quality

Available Data

Profile measurements were taken for the left and right wheel, totaling about 45,000 data points for each data set (exact number depends on the total length surveyed). The raw were analyzed using ProVAL Version 3.61, focusing on two ride quality indices: International Roughness Index (IRI) and the Ride Number (RN). ProVAL calculates indices after aggregating the measurements into segments whose length is a user input (Transtec Group, 2016).

Figure 4. 8 shows the comparative IRI plot for the left wheel in lane R1. The red line corresponds to IRI=95, a commonly used threshold for profile correction (TxDOT, 2004). Figure 4. 9 shows the comparative plot of RN for lane R1. The red line corresponds to RN=2.5, another commonly used threshold in pavement evaluation. Appendix 2 has comparative plots of all data in larger size to facilitate visualization.

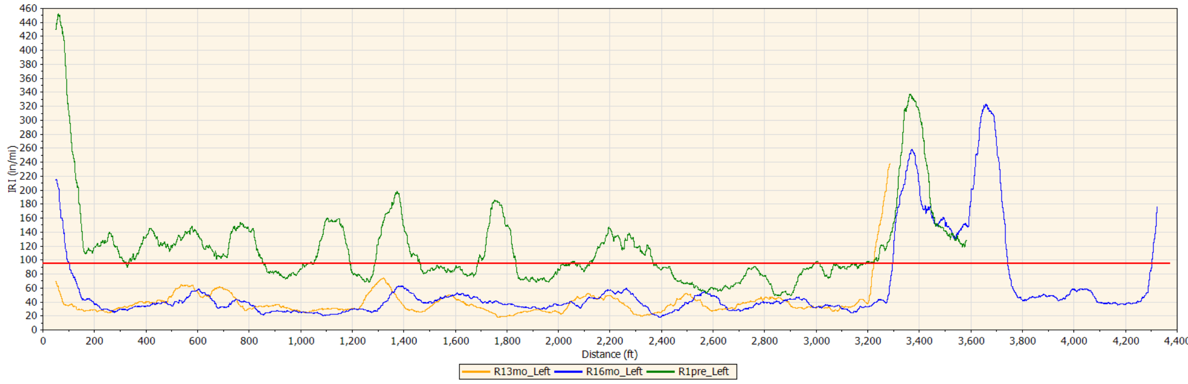


Figure 4. 8 Comparative IRI Plot for Lane R1, Left Wheel

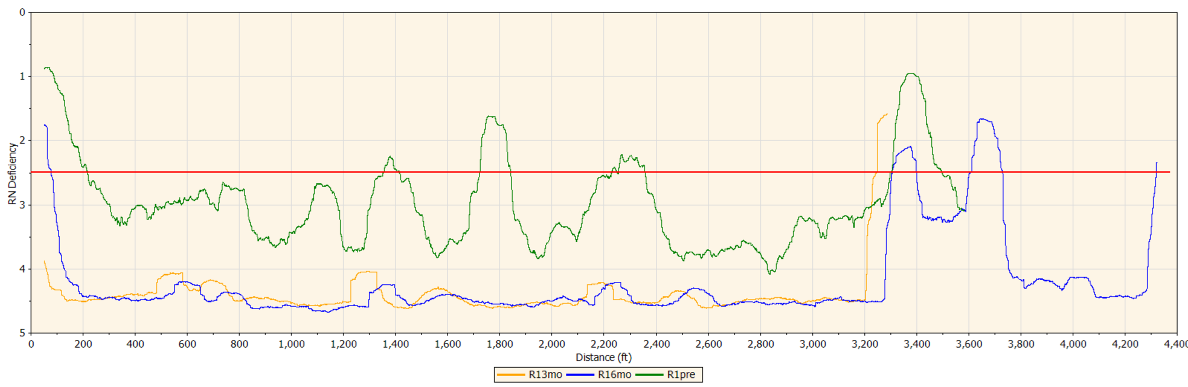


Figure 4. 9 Comparative RN Plot for Lane R1

Table 4. 1Figure 4. 8 and Figure 4. 9 show that NGDG considerably improved both indices, and that both 3- and 6-month post-NGDG data are very similar. Comparable data behavior can be observed for all lanes and wheel paths (see Appendix 2). Table 4. 6 shows the total length surveyed on each lane in each data run. The total test section length was approximately 3590 ft (Ella to TC Jester and vice-versa, excluding the bridges at both ends). It was not possible to determine where each profile run physically started and ended; their surveyed lengths indicate that all post-NGDG data except lane R1 must have included either the Ella Blvd overpass, or the bridge over TC Jester and the White Oak Bayou, or parts of both. NGDG was not done over structures, so any data points on bridges are invalid. Unfortunately, it was not possible to identify which data points were outside the test section with any certainty.

Table 4. 6 Length of Sections Surveyed for Profile Measurements (ft)

	R1	R2	L1	L2
Pre-NGDG	3,585	3,547	3,537	3,533
Post-NGDG 3mo	3,288	4,264	4,362	4,282
Post-NGDG 6 mo	4,322	4,344	4,442	4,330

Comparative Analysis

Overall IRI and RN results are summarized in Table 4. 7 as well as in Figure 4. 10 (IRI) and Figure 4. 11 (RN), for the entire length surveyed. The NGDG treatment caused a significant improvement on both ride quality indices, IRI and RN. The overall improvements between the pre-NGDG and average post-NGDG ranged from 91% to 202% for IRI and from 35% to 64% for RN. These results are consistent with previous experience with NGDG, where significant ride quality improvements were also observed.

Table 4. 7 Pre- and Post-NGDG Ride Quality Comparison-Entire Surveyed Section

Lane	RQ Index	Pre-NGDG	Post-NGDG			Percent Improvement		
			3 mo	6 mo	Avg(3,6)	3mo/pre	6mo/pre	Avg/pre
R1	Left IRI	121.4	43.8	62.4	53.1	177%	95%	129%
	Right IRI	134.7	54.9	64.4	59.7	145%	109%	126%
	Average IRI	128.0	49.4	63.4	56.4	159%	102%	127%
	RN	2.6	3.8	3.6	3.7	48%	40%	44%
R2	Left IRI	177.6	58.6	59.0	58.8	203%	201%	202%
	Right IRI	179.1	70.5	64.4	67.5	154%	178%	165%
	Average IRI	178.4	64.6	61.7	63.1	176%	189%	183%
	RN	2.3	3.7	3.8	3.7	61%	64%	62%
L1	Left IRI	146.8	70.2	75.1	72.6	109%	95%	102%
	Right IRI	179.8	91.7	64.7	78.2	96%	178%	130%
	Average IRI	163.3	80.9	69.9	75.4	102%	134%	117%
	RN	2.4	3.4	3.5	3.5	42%	46%	44%
L2	Left IRI	133.7	85.1	54.7	69.9	57%	144%	91%
	Right IRI	162.4	72.4	67.8	70.1	124%	140%	132%
	Average IRI	148.1	78.7	61.2	70.0	88%	142%	112%
	RN	2.6	3.5	3.7	3.6	35%	42%	38%
IRI	Max	179.8	91.7	75.1	78.2	203%	201%	202%
	Min	121.4	43.8	54.7	53.1	57%	95%	91%
	Average	154.4	68.4	64.1	66.2	131%	142%	134%
RN	Max	2.6	3.8	3.8	3.7	61%	64%	62%
	Min	2.3	3.4	3.5	3.5	35%	40%	38%
	Average	2.5	3.6	3.6	3.6	46%	48%	47%

IRI measured in in/mi

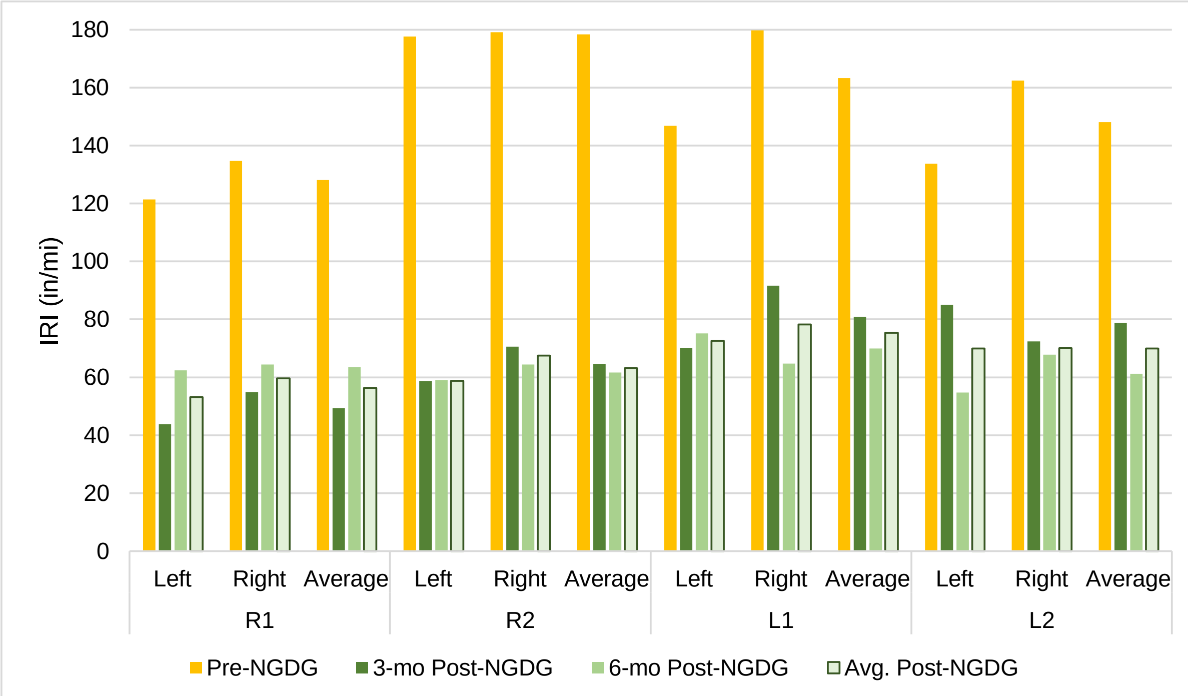


Figure 4. 10   Comparison of Overall IRI

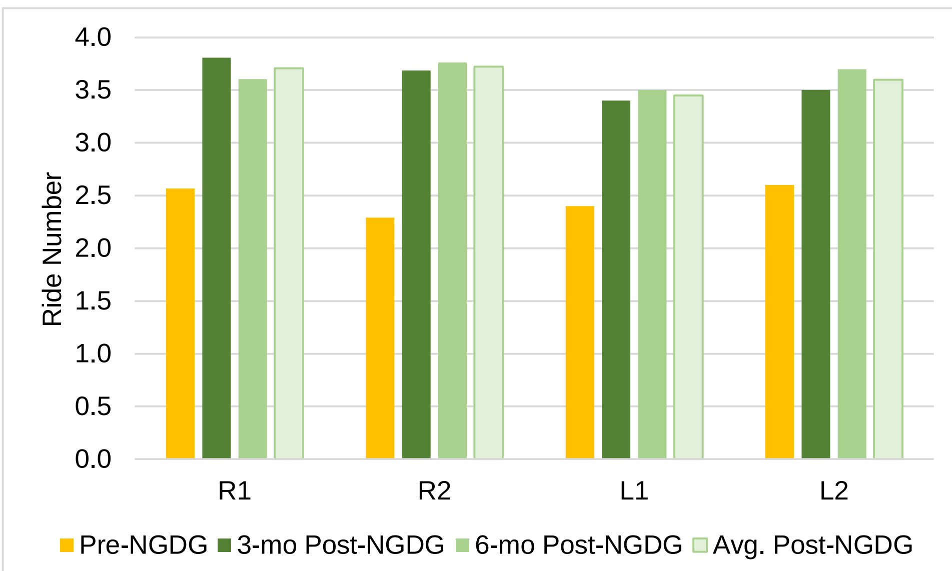


Figure 4. 11   Comparsion of Overall RN

Table 4. 8 summarizes of the lengths with IRI > 95, calculated with ProVAL. Pre-NGDG IRIs were estimated for 100-ft-long segment lengths, while post-NDGD were estimated for 20-ft segments to increase accuracy. The analysis was performed for the total surveyed length, so it includes the high IRI values seen in Figure 4. 8 (as well as in the other comparative plots in Appendix 2). Data points on structures could not be removed from the analysis, nor could the data points be paired among lanes and data runs due to lack of information about the start and ending points of the post-NGDG profile surveys. Nevertheless, the improvement

was considerable. In terms of overall average of all lanes and both wheels, pre-NGDG pavement presented 2,853ft with IRI > 95, or 80% of the average survey length, while the treated pavement averaged 480ft with IRI > 95, or 11% of the average survey length. As previously discussed, any survey longer than 3500-3600ft would include bridges and overpasses, which did not receive NGDG; in addition, their joints may negatively affect profile measurements.

Table 4. 8 Segments with IRI > 95

Lane	Wheel	Test	Survey Length (ft)	Total Length with IRI>95	Percent Length with IRI>95	Average IRI>95
R1	*Left*	Pre-NGDG	3,585	1,977.2	55%	139.9
		Post-NGDG 3mo	3,288	99.7	3%	160.8
		Post-NGDG 6mo	4,322	499.3	12%	296.2
	Right	Pre-NGDG	3,585	2,468.4	69%	156.2
		Post-NGDG 3mo	3,288	207.0	6%	171.9
		Post-NGDG 6mo	4,322	496.5	11%	216.0
R2	*Left*	Pre-NGDG	3,547	3,058.3	86%	194.6
		Post-NGDG 3mo	4,264	368.4	9%	178.1
		Post-NGDG 6 mo	4,344	450.8	10%	184.3
	Right	Pre-NGDG	3,547	3,166.7	89%	210.3
		Post-NGDG 3mo	4,264	570.9	13%	156.9
		Post-NGDG 6 mo	4,344	511.9	12%	159.2
L1	*Left*	Pre-NGDG	3,537	2,917.5	82%	140.9
		Post-NGDG 3mo	4,362	523.3	12%	157.9
		Post-NGDG 6 mo	4,442	489.8	11%	214.6
	Right	Pre-NGDG	3,537	3,366.5	95%	196.3
		Post-NGDG 3mo	4,362	831.4	19%	149.9
		Post-NGDG 6 mo	4,442	484.0	11%	170.8
L2	*Left*	Pre-NGDG	3,533	2,679.8	76%	152.0
		Post-NGDG 3mo	4,282	654.0	15%	143.0
		Post-NGDG 6 mo	4,330	456.4	11%	324.5
	Right	Pre-NGDG	3,533	3,188.9	90%	137.1
		Post-NGDG 3mo	4,282	575.0	13%	148.5
		Post-NGDG 6 mo	4,330	466.8	7%	293.6

Summary of Conclusions

Overall improvements in ride quality, for all lanes combined, ranged from:

- 91% to 202% for IRI
- 35% to 64% for RN

Segments with IRI > 95:

- Pre-NGDG: 2,853ft or 80% of the surveyed length
- Post-NGDG: 480ft or 11% of the average surveyed length.

As discussed above, the post-NGDG data necessarily included non-treated segments, since the surveyed length was longer than the NGDG length. It is likely that most if not all post-NGDG segments with IRI>95 were not ground.

Macrotexture

Available Data

Macrotexture data consists of mean profile depth (MPD) measured in millimeters (mm), reported at every 2ft and available for pre-NGDG and the two post-NGDG data collection efforts. Table 4. 9 summarizes the basic statistics for the data collected on Loop 610. The 6-month post-NGDG data included a replication (2 data runs per lane, unpaired).

Table 4. 9 Summary of Mean Profile Depth (MPD in mm) Data Collected on Loop 610

Lane	Data	Survey Length (ft)	Data Points	Minimum	Mean	Std. Dev.	Maximum
R1	Pre-NGDG	4210	2331	0.27	1.22	0.44	7.50
	3 mo.	5993	3317	0.35	1.55	0.66	4.85
	6 mo, run1	5575	3087	0.15	1.40	0.70	4.30
	6 mo, run 2	5689	3150	0.05	1.45	0.73	4.70
R2	Pre-NGDG	4210	2331	0.33	1.30	0.47	4.86
	3 mo.	6107	3380	0.20	1.80	0.96	6.70
	6 mo, run 1	5803	3213	0.05	1.39	0.67	5.35
	6 mo, run 2	5689	3150	0.05	1.35	0.69	5.45
L1	Pre-NGDG	4210	2331	0.19	1.59	0.83	8.12
	3 mo.	6334	3506	0.30	1.96	1.09	6.55
	6 mo, run 1	5689	3150	0.10	1.51	1.00	9.00
	6 mo, run 2	5462	3024	0.10	1.42	0.75	5.30
L2	Pre-NGDG	4210	2331	0.27	1.57	0.62	7.62
	3 mo.	5537	3065	0.30	1.72	1.07	9.70
	6 mo, run 1	5575	3087	0.05	1.34	0.87	5.30
	6 mo, run 2	5575	3087	0.05	1.26	0.76	4.80

Note: MPD definition is discussed in Chapter 2. See ASTM E1845-09 for details.

The pre-NGDG texture data runs started at the first sound segment landmark of each lane. For example, the data run on Loop 610 lane R1 started at the beginning of sound data segment A (see Chapter 3 for locations of sound data segments). Starting and/or ending points were not reported for either set of post-NGDG data, and the distance surveyed was longer than the pre-NGDG test section (see Table 4. 9). This precludes a paired data analysis. As previously discussed, any survey section longer than 3500-3600ft would include bridges and overpasses, which did not receive NGDG treatment; in addition, bridge joints may negatively affect profile and texture measurements.

Preliminary Analysis

Pre-NGDG data collected in November 2014, on both US290 and Loop 610 test sections, were analyzed in detail in order to develop a methodology to later perform paired comparisons and to correlate to sound measurements if applicable. Data points that fell inside each sound segment were identified based on the cumulative distance from the starting point of each data run. There were between 314 and 316 texture data points on each of the 24 sound data segments (see Chapter 3 for sound segment locations). These results were documented in a technical memorandum, but could not be used as initially planned due to lack of information about starting and ending points of the post-NGDG data.

The first step in the preliminary analysis determined how to treat the only set of data with two replications, which were available for the 6-month post-NGDG. Figure 4. 12 shows the scatter plot of both data runs for lane R1. The red line is the line of equality. All other lanes presented similar scatters.

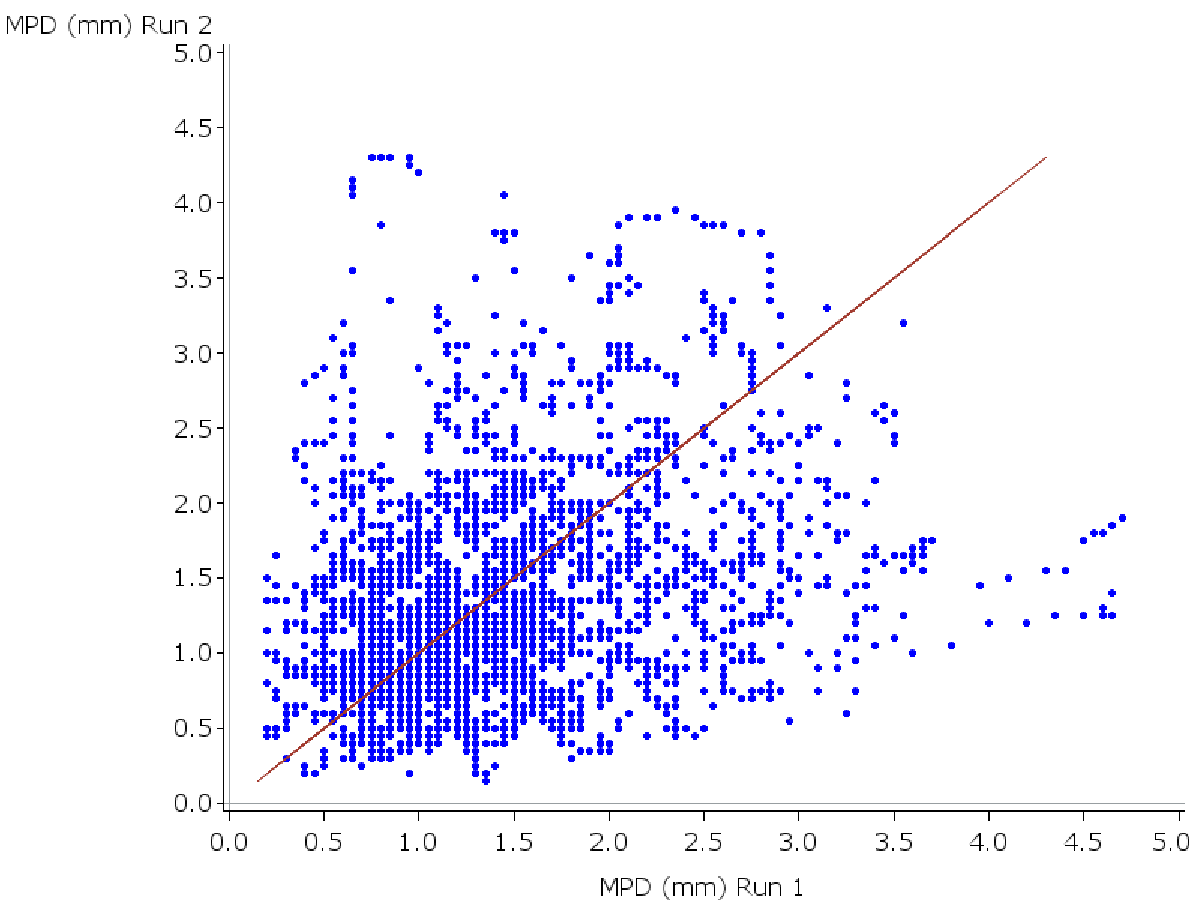


Figure 4. 12 Correlation between Two Data Runs Available for Post-NGDG-6mo –Lane R1

The consistency among both data runs was tested and the results are summarized in Table 4. 10. The tests check if both data sets come from the same population (i.e., are statistically the same). The P-values can be interpreted as the probability of being wrong when assuming that they do. Both data runs are consistent for all lanes, i.e., there is no difference between the runs. The standard errors of both samples are also reported; their similar order of magnitude reinforces the conclusion. Figure 4. 13 illustrates the comparison being performed. Distributions for other lanes were similar. Conclusion: aggregate both data runs for the comparative analysis.

Table 4. 10 Post-NGDG-6-month Data—Consistency between Replications

	R1	R2	L1	L2
Standard Error Run 1	0.0127	0.0118	0.0178	0.0156
Standard Error Run 2	0.0130	0.0123	0.0137	0.0136
P-value	1.03%	1.32%	0.01%	0.03%
Interpretation	Consistent	Consistent	Consistent	Consistent

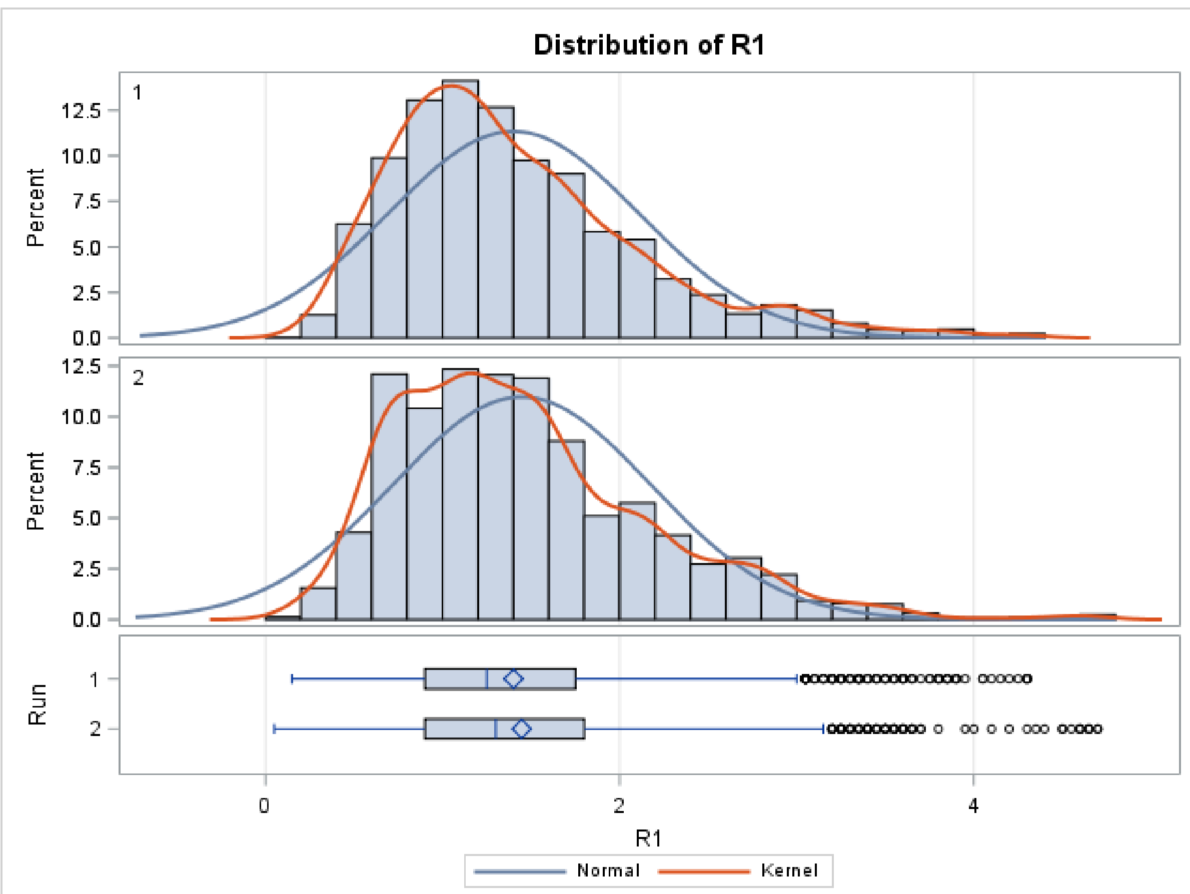


Figure 4. 13 Comparison of the Distributions of Post-NGDG-6 month Replications

Comparative Analysis

Table 4. 11 shows the 95% confidence intervals for the mean MPD, for pre-NGDG and for post-NGDG 3 and 6-month separately as well as aggregated. Table 4. 12 summarizes the percent changes in mean MPD, comparing post-NGDG 3 and 6mo to pre-NGDG and also comparing the aggregated post-NGDG means to pre-NGDG.

Table 4. 11 95% Confidence Intervals for Mean MPD

		Lower Limit	Mean	Upper Limit
L1	*Post-NGDG all*	1.623	1.643	1.663
	Post-NGDG-6mo	1.441	1.463	1.485
	Post-NGDG-3mo	1.923	1.959	1.995
	Pre-NGDG	1.555	1.588	1.622
L2	*Post-NGDG*	1.420	1.439	1.458
	Post-NGDG-6mo	1.279	1.300	1.320
	Post-NGDG-3mo	1.680	1.718	1.756
	Pre-NGDG	1.544	1.569	1.594
R1	*Post-NGDG*	1.455	1.469	1.483
	Post-NGDG-6mo	1.406	1.424	1.441
	Post-NGDG-3mo	1.531	1.554	1.576
	Pre-NGDG	1.200	1.218	1.236
R2	*Post-NGDG*	1.501	1.501	1.534
	Post-NGDG-6mo	1.348	1.365	1.382
	Post-NGDG-3mo	1.771	1.804	1.837
	Pre-NGDG	1.284	1.303	1.322

Table 4. 12 Percent Changes in Mean MPD

Comparison	L1	L2	R1	R2	Average All Lanes
Post NGDG 3mo/Pre NGDG	23.4%	9.5%	27.6%	38.4%	24.7%
Post NGDG 6mo/Pre NGDG	-7.9%	-17.2%	16.9%	4.7%	-0.9%
Post NGDG (all) /Pre NGDG	3.4%	-8.3%	20.6%	16.4%	8.0%

The results were inconsistent, indicating a considerable MPD improvement for the 3-month post-NGDG but not for 6-month post-NGDG. This is neither consistent with the improvements observed in skid resistance and in ride quality, nor with the expected behavior of concrete pavements. Aggregating all post-NGDG data to attempt to "average out" those inconsistencies resulted in small MPD increases for all lanes except L1, where an 8.3% decrease of mean MPD was observed with respect to the pre-NGDG (see last row of Table 4. 12). The statistical significance of the differences between aggregated post-NGDG and pre-

NGDG were tested using the same tests described in the Skid Resistance section. The results indicated significant differences for all lanes except L1.

Summary of Conclusions

NGDG caused a small but significant increase in MPD for lanes R1, R2 and L2. It caused no change in lane L1. The post-NGDG-3-mo data set is more consistent with the other surface quality measurements. The MPD improvements between pre- and post-NGDG-3mo are consistent with the improvements observed for the other parameters that evaluate the surface roughness. If one considers only these measurements, the post-NGDG mean MPD improved 24.7% with respect to the pre-NGDG.

Conclusions and Discussion

Skid Resistance

- Data for parallel lanes are statistically heterogeneous; therefore, the comparative analysis was performed for each traffic lane individually rather than by traffic direction.
- For the southbound direction (L1 and L2), the 3 and 6-month post-NGDG data were statistically the same in all cases except the percent slip (both L1 and L2). Conclusion: no change in skid resistance after 3 months.
- For the northbound direction (R1 and R2), only the peak value and the percent slip were statistically the same for 3 and 6-month data. Visual inspection of the data (see Appendix 1) indicates that skid resistance consistently improved after 3 months of traffic. This is counterintuitive and seems due to unknown external factors influencing post-NGDG data collection. Conclusions: minimize external influence by averaging 3 and 6-month post-NGDG northbound data for the analysis.
- Calculations of the percent change between the mean of 13 pre-NGDG data points and the 26 aggregated post-NGDG data points indicated improvements for all variables except peak friction in lane R2. The best improvement in average SN was observed in lane L1 (102.1%), and smallest in lane R2 (15.2%). The overall improvement in average SN was 59.5%, and the overall percent slip dropped almost 35%.
- Post-NGDG results showed considerable improvement for all lanes, but especially for lanes L1 and L2, where the maximum pre-NGDG SN was less than the minimum post-NGDG SN for both 3 and 6-month post-NGDG data sets.
- The skid resistance results are consistent with NGDG experience reported in the literature, where significant improvements were also observed.

Ride Quality

- Post-NGDG surveyed lengths indicate that they must have included segments over bridge structures (which did not receive NGDG). There was no information on the physical location of start and ending points of each data run. Therefore, only a few data points at one end or another of the surveyed section could be removed from the analysis; the analysis includes nearly all data points. Even so, the improvements were impressive.

- The improvements between pre-NGDG and post-NGDG ranged from 91% to 202% for IRI and from 35% to 64% for RN.
- Pre-NGDG pavement presented 2,853 ft with IRI > 95, or 80% of the average survey length, while the post-NGDG surface averaged 480 ft with IRI > 95, or 11% of the average survey length. As noted in the first bullet, post-NGDG surveys were longer than the test section (3590 ft), which means they include parts of one or both overpasses that limit the test section. Overpasses did not receive NGDG; in addition, their joints may negatively affect profile measurements.
- The ride quality results are consistent with NGDG experience reported in the literature.

Macrotexture

- Post-NGDG macrotexture surveyed lengths were also longer than the test section. The observations in the first bullet under ride quality are also applicable for macrotexture.
- The 3-month post-NGDG MPD increases ranged between 9.5% and (lane L2) and 38.4% (lane R2), averaging 24.7%. Based on the literature and on the improvements in the other measurements related to surface roughness, significant MPD improvement was expected.
- The 6-month post-NGDG MPD decreased with respect to pre-NGDG for the southbound traffic direction. The average change was -0.9%. This is neither consistent with the literature or with the 3-month data, nor with the improvements in the other measurements related to surface roughness.
- Aggregating post-NGDG resulted in small, statistically significant MPD increases for lanes L1, R1 and R2, and a non-significant decrease for lane L2.
- Only the post-NGDG-3-month data was consistent with NGDG experience reported in the literature. Nevertheless, even the results including the 6-month post-NGDG data set indicate positive effects of the NGDG treatment.

Summary conclusion: the NGDG surface significantly improved the skid resistance properties as well as the ride quality of the test section analyzed. Macrotexture showed a less significant improvement; however, the macrotexture analysis was inconclusive due to inconsistencies with the other indices related to roughness as well as inconsistencies between the 3- and 6-month post-NGDG data. Some of these inconsistencies may have been due to the fact that the surveyed length included structures not treated with NGDG.

Chapter 5
Comparative Analysis of Sound Data

This chapter discusses the analysis of the sound data collected during this project, first explaining the methodology then presenting the results, conclusions and recommendations. It covers Tasks 3 and 4 for the sound data; these tasks are respectively titled "Perform Pre and Post-Diamond Grinding Measurements," and "Perform Mid-Range Post-Diamond Grinding Measurements."

As previously stated, Loop 610 data were collected on three occasions: before construction (pre-NGDG), and 3 and 6 months after completion, respectively termed in this chapter as “post-NGDC-3mo” and “post-NGDG-6mo.” Pre-NGDG data is also available for US290. Since no post-NGDG data exists for US290 due to changes in construction schedules, a comparative analysis was possible only for Loop 610.

Available Data

As documented in Chapter 3, “Monitoring Plan,” TxDOT collected data on the two rightmost lanes of Loop 610 test section in each traffic direction. On-board sound intensity (OBSI) test section length is standardized as 440ft (AASHTO 2015). Lane designations R1, R2, L1 and L2 follow TxDOT's Pavement Management Information System (TxDOT 2014) nomenclature. TxDOT collected data for 3 segments for each lane in each traffic direction, designated as depicted in Figure 5. 1.

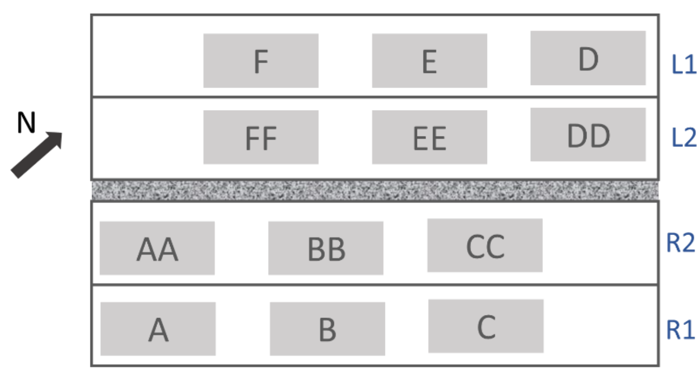


Figure 5. 1 Lane and Test Segment Designations for Sound Data

Note: Loop 610 is wider; this sketch shows only the surveyed lanes.

TxDOT performed 3 data runs per sound data segment, covering 3 segments per lane and 2 lanes per highway and traffic direction. TxDOT provided spreadsheets containing on-board sound intensity (OBSI) and sound pressure levels (SPL) data for both probes (leading and trailing), for each segment and data run. Table 5. 1 is a sample of one of the sound data files.

Table 5. 1 Sample of Sound Data File, One Data Run

1/3 Octave Band

	Leading Edge			Trailing Edge			Average	
Freq, Hz	OBSI_L	SPL	PI Index	OBSI	SPL	PI Index	OBSI	SPL
250	80.5	83.7	3.1	81.4	82.7	1.3	81.0	83.2
315	82.0	84.0	2.0	83.3	86.2	2.9	82.7	85.2
400	85.7	87.2	1.5	84.1	87.5	3.4	85.0	87.3
500	88.1	90.9	2.8	89.1	91.9	2.8	88.6	91.4
630	94.6	96.5	1.8	92.1	93.6	1.5	93.5	95.3
800	100.9	102.4	1.5	99.7	101.5	1.9	100.3	102.0
1000	104.9	107.1	2.2	107.0	109.2	2.2	106.1	108.2
1250	101.8	103.1	1.2	104.1	105.4	1.3	103.1	104.4
1600	99.1	101.3	2.2	101.0	102.7	1.8	100.1	102.1
2000	98.5	100.6	2.1	97.8	99.8	2.0	98.1	100.2
2500	95.2	97.0	1.8	95.9	97.4	1.5	95.6	97.2
3150	88.8	91.1	2.2	91.7	93.0	1.3	90.5	92.1
4000	83.7	86.5	2.8	86.4	88.3	1.9	85.3	87.5
5000	80.8	84.5	3.8	82.7	86.1	3.4	81.9	85.4
Overall	109.1	111.0		110.5	112.4		109.9	111.7

Narrow Band

	Leading Edge				Trailing Edge				Average	
Freq, Hz	OBSI	SPL	PI Index	Coherence	OBSI	SPL	PI Index	Coherence	OBSI	SPL
254	81.4	83.6	2.1	1.0	81.3	81.7	0.4	0.9	81.4	82.7
261	77.4	82.4	5.0	1.0	77.7	82.6	4.8	0.9	77.6	82.5
269	76.8	82.3	5.4	1.0	80.9	84.6	3.7	1.0	79.3	83.6
277	74.9	82.6	7.8	1.0	82.1	86.2	4.1	1.0	79.8	84.8
285	78.8	84.0	5.2	1.0	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

800 data points

[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	60.0	63.4	3.3	0.9	59.1	62.5
6335	57.4	61.0	3.6	0.9	59.6	63.0	3.4	0.8	58.7	62.1
6342	57.7	61.2	3.6	0.9	59.7	63.1	3.4	0.9	58.8	62.3
6350	58.3	61.8	3.5	0.9	60.1	63.4	3.3	0.9	59.3	62.7
6358	57.2	60.9	3.6	0.8	60.8	64.1	3.3	0.9	59.4	62.8
6365	56.9	60.6	3.7	0.9	60.2	63.6	3.4	0.9	58.9	62.3
6373	56.9	60.6	3.7	0.8	59.3	62.8	3.5	0.8	58.3	61.8
6381	55.8	59.5	3.7	0.8	59.7	63.0	3.4	0.9	58.1	61.6
6389	56.0	59.7	3.8	0.8	59.1	62.5	3.5	0.8	57.8	61.3
6396	56.6	60.3	3.6	0.8	59.3	62.7	3.4	0.8	58.1	61.6

As illustrated in Table 5. 1, the sound data Excel workbooks contain the following data for each segment (designated by letters according to Figure 5. 1):

- Narrow band spectra, 254 Hz to 6396 Hz, increasing by 7.7 Hz. There are 800 data points per probe per run, totaling 4800 data points per segment.
- 1/3 octave spectra, from 250 to 5000 Hz, increasing by an average factor of 1.26. There are 14 data points per probe per run. AASHTO 76 (2015) recommends the frequency range between 400 Hz and 5000 Hz; therefore, 12 data points per segment per data run were used in the analysis. The 2 probes were averaged.
- In addition, the sound data Excel workbooks came with following:
    - OBSI and SPL averages of both probes.
    - Pressure-intensity (PI) index, a data quality check index that equals the difference between SPL and OBSI.
    - Coherence for the narrow band measurements. Coherence is the correlation coefficient between both probes' measurements (Sandberg et al., 2002).

- Overall averages of A-weighted OBSI levels for the 1/3 octave band frequencies.
- Sound intensity averages by test run and microphone, calculated according to AASHTO 76 (2015) requirements (energy-averaged).
- Standard charts and data reports for the 1/3 octave band data.

Due to lane closures during some data collection days, not all segments have all data. Table 5. 2 summarizes the available data. The initially planned complete nested factorial design turned out to be incomplete; nevertheless there was enough data for a meaningful analysis.

Table 5. 2 Available Sound Data

Lane	Segment	Pre-NGDG	Post-NGDG 3 months	Post-NDGD 6 months
R1	*A*	✓	✓	✓
	B	✓	✓	✓
	C	✓	✓	✓
L1	*D*	✓	✓	Closed
	E	✓	✓	Closed
	F	✓	✓	Closed
R2	*AA*	✓	Closed	✓
	BB	✓	Closed	✓
	CC	✓	Closed	✓
L2	*DD*	✓	✓	✓
	EE	✓	✓	Closed
	FF	✓	✓	Closed

Preliminary Analysis

In the OBSI procedure, the A-weighted scale is used to mimic the human hearing spectrum. As discussed in Chapter 2, "Literature Review," the 1/3 octave band, A-filter, is the closest way to mimic how humans process sound. Therefore, this project's analysis focuses on AASHTO 76 (2015) recommendation: "reporting shall be done in 1/3 octave band frequency spectra with center frequencies between 400 and 5000 Hz."

As discussed in Chapter 2, decibels are ratios of the sound pressures or intensities to reference values, expressed in a logarithmic scale. Therefore, decibels should not be added, subtracted or averaged directly; they must be reconverted into the corresponding pressure or intensity, then averaged (or other calculation), then converted back into decibels. AASHTO (2015) calls this average "energy average." Energy-averages by probe and by data run were already provided in TxDOT's spreadsheets. All other calculations and averages used in this chapter were also "energy-averaged" (or "energy-calculated") as recommended by AASHTO (2015). Figure 5. 2 through Figure 5. 5 show plots of the 1/3 octave frequency spectra averaged by probe and by data run.

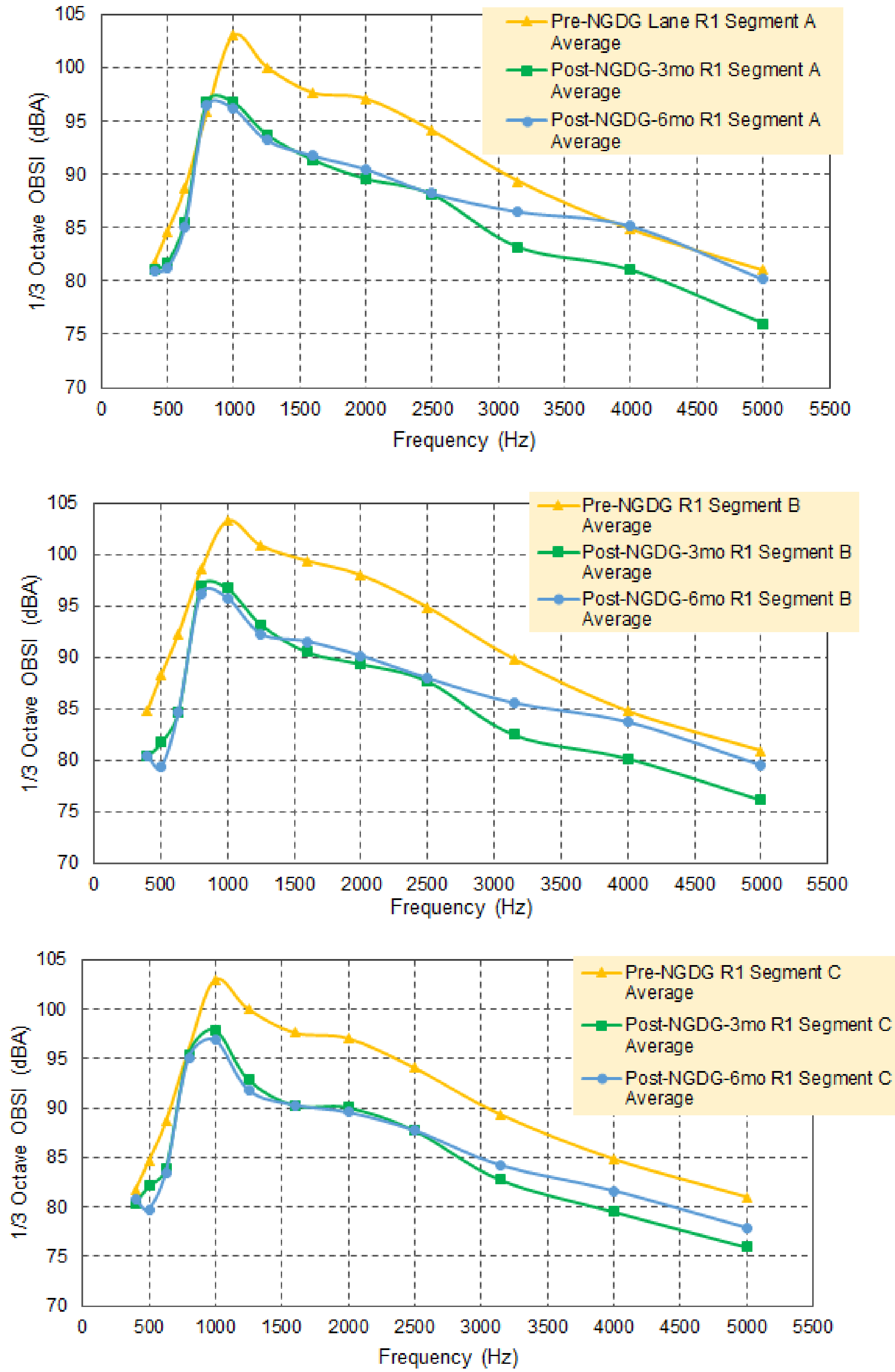


Figure 5. 2 Comparative Plots of 1/3 Octave Band Intensity Levels, Lane R1

Note: Averages of both probes and the three data runs.

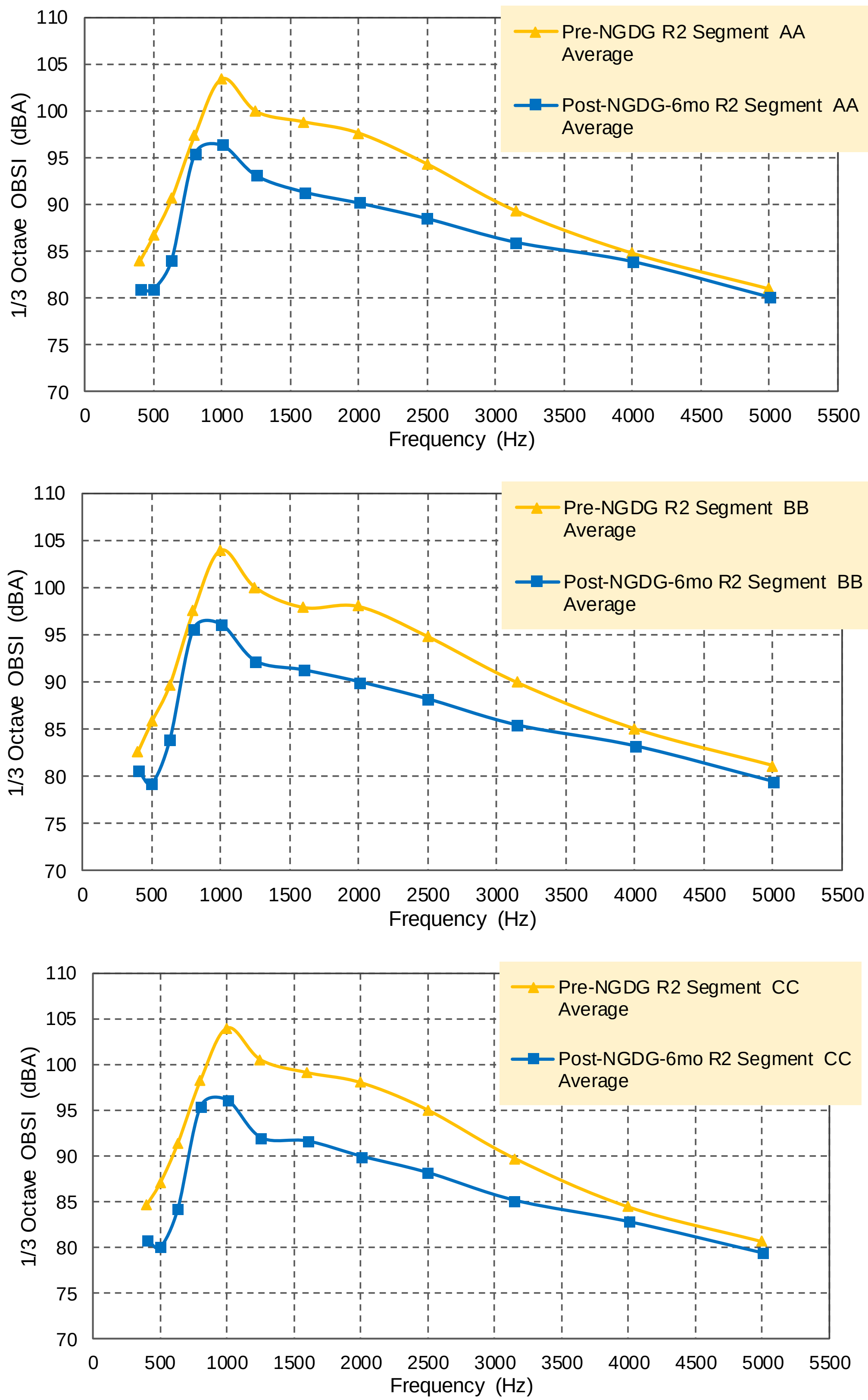
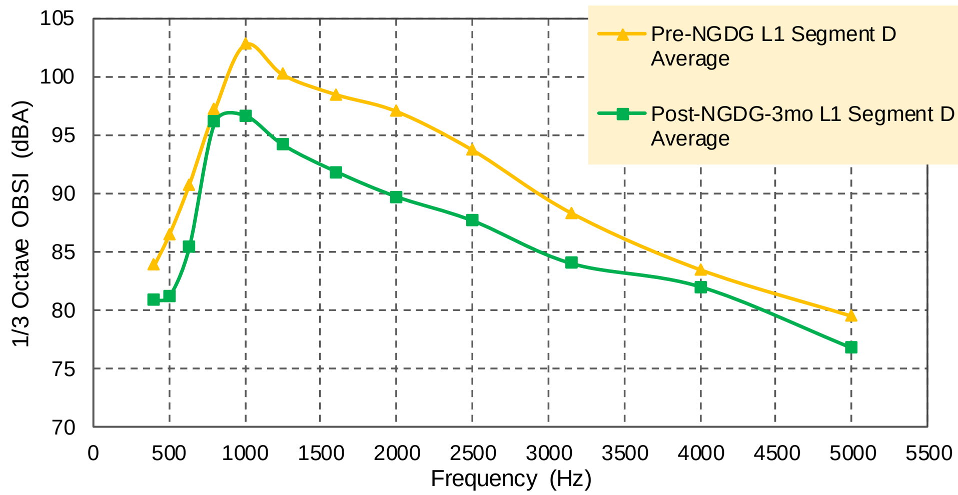
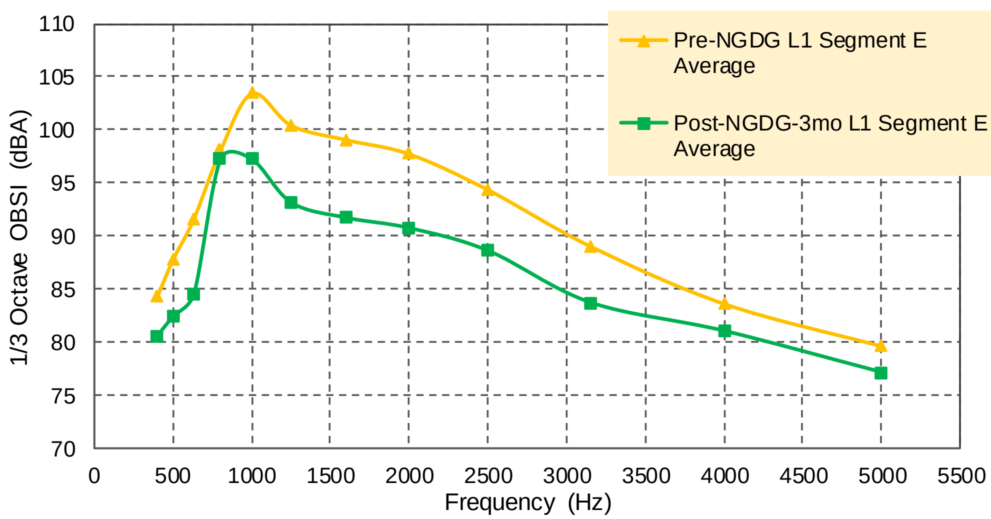


Figure 5. 3 Comparative Plots of 1/3 Octave Band Intensity Levels, Lane R2

Note: Averages of both probes and the three data runs.





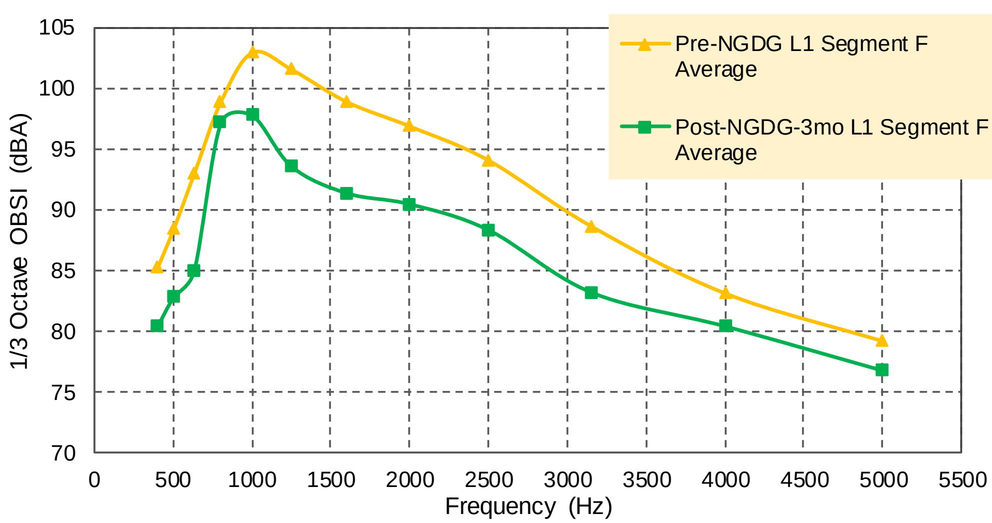


Figure 5. 4 Comparative Plots of 1/3 Octave Band Intensity Levels, Lane L1

Note: Averages of both probes and the three data runs.

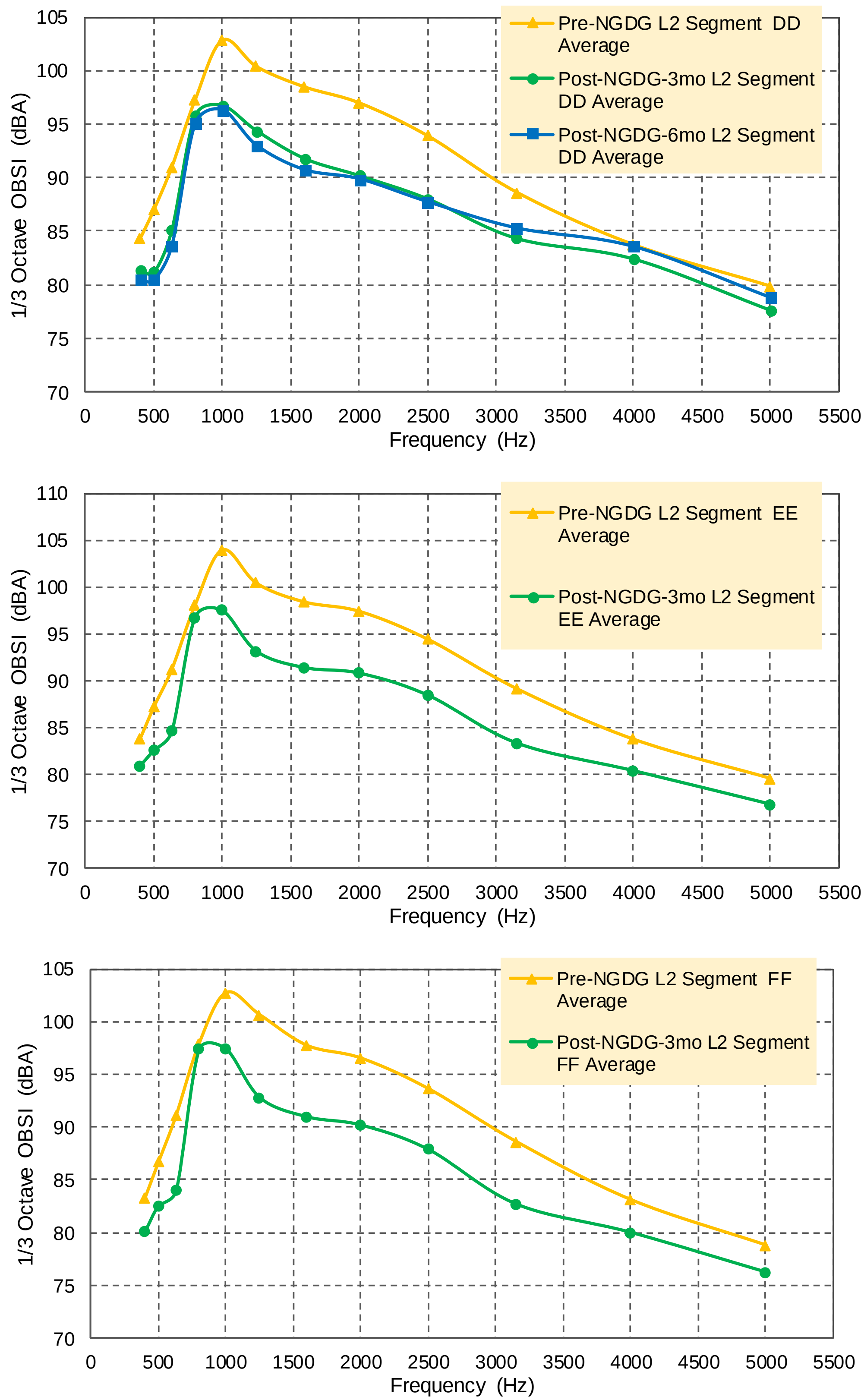


Figure 5. 5 Comparative Plots of 1/3 Octave Band Intensity Levels, Lane L2

Note: Averages of both probes and the three data runs.

Visual inspection of Figure 5. 2 through Figure 5. 5 indicates a significant improvement. The data also appears to indicate little difference between both post-NDGD data collection dates. All these visual indications were verified with statistical tests as discussed in the next section.

AASHTO 76 (2015) specifies the data quality checks listed below for OBSI data:

- Pressure-intensity (PI). PI is the difference between average OBSI and SPL. A large difference indicates that the measurement is affected by noises other than pavement-tire interaction. AASHTO 76 (2015) requires that the pressure-intensity (PI) stay within certain ranges for each frequency in the 1/3 octave band (between 400 and 5000 Hz) for the average of both probes. Data outside recommended PI ranges must be disregarded. All data were within AASHTO specifications. Just as a curiosity, the PI index was checked for each probe individually, and the only data point outside the recommended AASHTO range was pre-NGDG, lane L1, segment E, leading probe, 1000 Hz. The PI value was 2.72 dB.
- Coherence between both microphones. Coherence is the correlation between both microphones. AASHTO 76 (2015) states that coherence values less than 0.8 are not acceptable. The spreadsheets provided by TxDOT had coherence calculations for the 1/24 octave bands but not for the 1/3 octave. Therefore, the coherences were calculated for the 1/3 octave band for all segments and all data runs. All were over 0.94, as shown in Table 5. 1, therefore exceeding AASHTO's threshold of 0.8.
- The range of sound intensities from multiple test runs must be no greater than 2.0 dB. All data points satisfied this requirement.
- Temperature normalization. According to TxDOT, the data provided was already normalized.

Table 5. 3 Coherence for 1/3 Octave Band Data Runs

		Pre-NGDG			**Post-NGDG-3mo**			**Post-NGDG-6mo**		
Lane	**Segment**	*Run 1*	*Run 2*	*Run 3*	*Run 1*	*Run 2*	*Run 3*	*Run 1*	*Run 2*	*Run 3*
R1	*A*	0.98	0.98	0.99	0.98	0.98	0.98	0.96	0.97	0.96
	B	0.98	0.98	0.98	0.98	0.99	0.98	0.95	0.94	0.95
	C	0.97	0.97	0.97	0.99	0.99	0.99	0.94	0.95	0.95
R2	*AA*	0.98	0.98	0.98	Closed			0.95	0.96	0.95
	BB	0.98	0.98	0.98	Closed			0.95	0.95	0.94
	CC	0.98	0.98	0.98	Closed			0.96	0.96	0.95
L1	*D*	0.98	0.98	0.98	0.98	0.98	0.99	Closed		
	E	0.98	0.98	0.98	0.98	0.98	0.98	Closed		
	F	0.98	0.98	0.98	0.98	0.98	0.98	Closed		
L2	*DD*	0.98	0.98	0.98	0.99	0.99	0.98	0.96	0.96	0.96
	EE	0.98	0.98	0.97	0.98	0.98	0.98	Closed		
	FF	0.98	0.98	0.97	0.98	0.98	0.98	Closed		

The three data runs were within AASHTO's validity criteria in all cases; nevertheless, statistical tests of homogeneity were performed to confirm that there is no difference among data runs (within each segment). The non-parametric tests of homogeneity discussed in Chapter 4 were also used here. Table 5. 4 shows the results. The numbers in this table are the tests' P-values, interpreted as the probability of being right when assuming that the 3 runs are the same.

Table 5. 4 Homogeneity Tests Among Data Runs

Lane	Segment	Pre-NGDG	Post-NGDG 3 months	Post-NDGD 6 months
R1	*A*	0.9410	0.954	0.997
	B	0.9111	0.883	0.953
	C	0.9221	1.000	0.9908
L1	*D*	0.9077	0.955	Closed
	E	0.9726	0.955	Closed
	F	0.9617	0.995	Closed
R2	*AA*	0.9910	Closed	0.863
	BB	0.9646	Closed	0.986
	CC	0.9581	Closed	0.950
L2	*DD*	0.9858	0.992	0.989
	EE	0.9509	0.980	Closed
	FF	0.9646	0.971	Closed

The next homogeneity test checked if there was a statistically significant difference among the three segments in each lane, for each data collection effort. Table 5. 5 shows the P-values. The interpretation again is the probability of being right when assuming that the 3 segments are statistically the same. The results indicated that they are. Therefore, data aggregated by lane is statistically the same.

Table 5. 5 Homogeneity Tests Among Segments Within Lanes

Lane	Pre-NGDG	Post-NGDG 3 months	Post-NDGD 6 months
R1	0.5193	0.883	0.452
L1	0.7982	0.985	Closed
R2	0.8351	Closed	0.812
L2	0.7982	0.880	Segment DD only

Once it was determined that segments within lanes were homogeneous, an additional set of homogeneity tests verified if the 3- and 6-month post-NGDG measurements on the same lane could be aggregated. Both sets of post-NGDG data were available only for lane R1 and for segment DD of lane L2 (see Table 5. 2). The P-values are listed below and the interpretation is the same as for previously presented tests.

R1 .... 0.9317

L2 .... 0.9834

The final homogeneity test verified if all 3- and 6-month post-NGDG data could be meaningfully aggregated for the entire test section. The P-value was 0.9521. Therefore, comparative analyses by lane are meaningful, and so is the comparison between pre- and post-NGDG data for the entire Loop 610 test section.

Comparative Analysis

Methodology

The comparative analysis was developed by lane and segment, by lane, and for the entire test section, for the overall sound intensity. Overall intensity is the sum of all intensities in each frequency in the 1/3 octave band, namely: 400—500—630—800—1000—1250—1600—2000—2500—3150—4000—5000Hz. As previously discussed, the sum must be performed by first converting the decibel values for each frequency back into the actual sound intensity, then adding the intensities and reconverting into decibels.

Sound intensities were first averaged for both probes and the three data runs for each of the frequencies in the 1/3 octave band. Then, the overall intensities were calculated by lane and segment. Next, overall intensities were averaged by lane, and for the entire test section. The decrease in noise level, which was very significant in all cases, was reported and evaluated using three methods:

1. Direct dBA subtraction. Such calculation is not mathematically correct but can be evaluated according to Table 5. 6. Note: this method is discussed in Chapter 2.
2. Sound intensity reduction expressed in dBA and as a percent change with respect to the pre-NGDG sound intensity.
3. Equivalent traffic reduction, an interesting evaluation method found in a Minnesota DOT report on in tire-pavement noise reduction (Izevbekhai 2007). Chapter 2 discusses this method.

Table 5. 6 Interpretation of Decibel Changes

Sound Amplitude - Loudness

Change in Sound Level (Δ dB)	Change in Loudness
1 to 3 dB	Just perceptible change
5 dB	Noticeable change
10 dB	Twice or (1/2) as loud
20 dB	Four times or (1/4) as loud

True only for the same sound!

Source: Wirth 2009

As explained in Chapter 2, decibels are the decimal logarithm of the sound intensity ratio with respect to a reference sound intensity selected so that, in an acoustic-free field, one obtains the same decibels when measuring sound pressure and sound intensity (Sandberg et al, 2002). The actual difference in sound intensity is calculated according to equation 5.1. The reduction factor with respect to pre-NGDG is calculated according to equation 5.2 and the percent improvement, according to equation 5.3.

$$\Delta I = 10log\left[\left(10^{\frac{Pre}{10}}\right) - \left(10^{\frac{Post}{10}}\right)\right] \tag{5.1}$$

$$RF = \frac{I_{pre}}{I_{post}} = \frac{10^{\frac{Pre}{10}}}{10^{\frac{Post}{10}}} \tag{5.2}$$

$$\%\ improvement = 100\left(1 - \frac{I_{post}}{I_{pre}}\right) \tag{5.3}$$

Where:

ΔI = actual difference between pre- and post-NGDG sound intensities (dBA)

Pre = Pre-NGDG sound intensity level (dBA)

Post = Post-NGDG sound intensity level (dBA), used for 3-month post-NGDG, 6-month post-NGDG and average post-NGDG

RF = noise reduction factor

I_{pre} = actual pre-NGDG sound intensity

I_{post} = actual post-NGDG sound intensity.

As discussed in Chapter 2, Minnesota DOT reported the noise reduction factors expressed in equations 5.2 and 5.3 as the equivalent traffic reduction (Izevbekhai 2007). The reasoning is explained in detail in Chapter 2, but in short it can be put as follows: assuming all vehicles make the same noise (sound intensity), the ratio I_{pre} / I_{post} is equivalent to the traffic reduction necessary for the pre-NDGD pavement to achieve the same noise level as the post-NGDG pavement.

Comparisons by Lane and Segment

Table 5. 7 shows the overall sound intensity levels observed at each segment and lane, for each post-NGDG data set and for the post-NGDG average. It also shows the difference in decibels by direct subtraction. Figure 5. 6 shows the same sound intensities data in graphic format, and Figure 5. 7 shows the averages by lane for easier visualization of the noise level improvements.

Table 5. 7 Overall Intensity Level by Lane and Segment

Lane and Segment	Pre-NGDG OBSI (dBA)	Post-NGDG-3mo OBSI (dBA)	ΔdBA (pre-3 mo)	Post-NGDG-6mo OBSI (dBA)	ΔdBA (pre-6 mo)	Average Post-NGDG	ΔdBA (pre-avg.post)
R1 average	**107.6**	**101.8**	**5.8**	**101.5**	**6.1**	**101.7**	**5.9**
A	107.0	102.0	5.0	101.9	5.1	102.0	5.0
B	107.9	101.8	6.1	101.4	6.6	101.6	6.3
C	107.9	101.7	6.2	101.2	6.7	101.4	6.4
R2 average	**107.8**			**101.3**	**6.4**	**101.3**	**6.4**
AA	107.5			101.5	5.9	101.5	5.9
BB	107.7			101.2	6.5	101.2	6.5
CC	108.1			101.3	6.8	101.3	6.8
L1 average	**107.6**	**102.2**	**5.4**			**102.2**	**5.4**
D	107.2	101.9	5.3			101.9	5.3
E	107.8	102.3	5.5			102.3	5.5
F	107.9	102.4	5.5			102.4	5.5
L2 average	**107.4**	**102.1**	**5.3**	**101.3**	**6.2**	**102.0**	**5.5**
DD	107.3	101.9	5.4	101.3	6.1	101.6	5.7
EE	107.9	102.2	5.7			102.2	5.7
FF	107.1	102.2	4.9			102.2	4.9

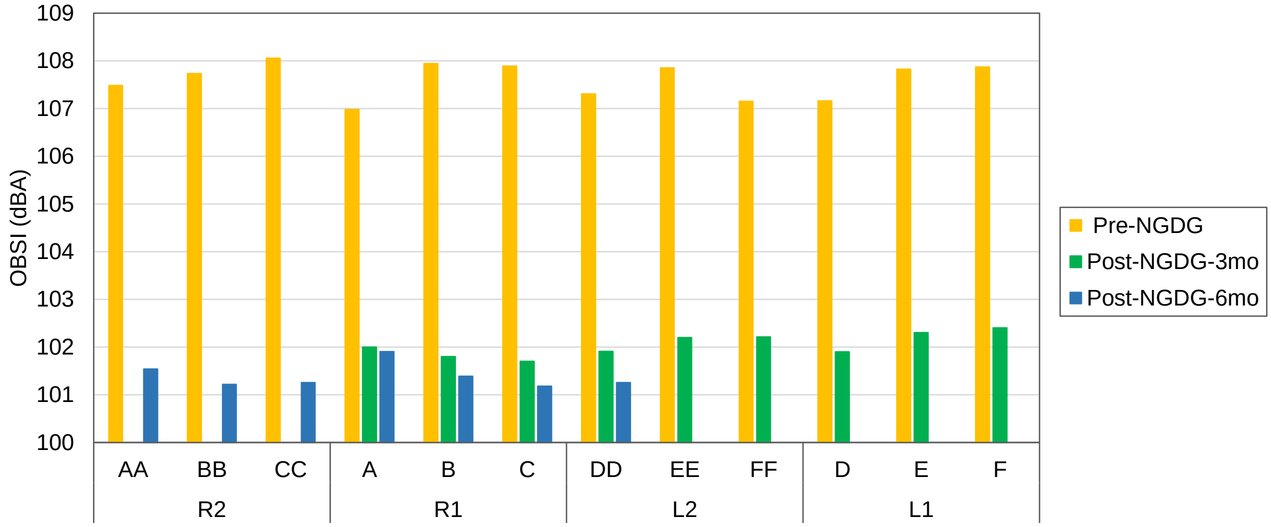


Figure 5. 6 Overall Sound Intensity Level by Lane and Segment

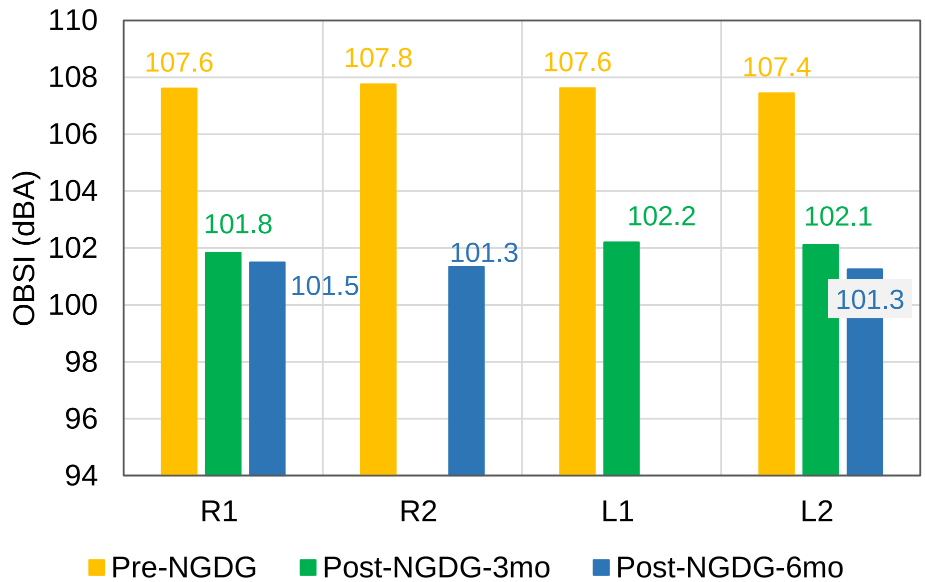


Figure 5. 7 Average Sound Intensity by Lane

The decibel drop (direct subtraction) ranged from 4.9 in lane L2, segment FF, to 6.8 in lane R2, segment CC. According to Table 5. 6, the threshold for a noticeable change is 5.0; therefore, all but one segment is classified as having had a "noticeable change." Moreover, the only segment with a decibel drop less than 5.0 was a drop of 4.9, which is very close to the "noticeable change" threshold.

Table 5. 8 shows the improvements in sound intensity levels with respect to the pre-NGDG measurements. The columns titled "Pre-3mo," "Pre-6mo" and "Pre-Avg.Post" are the actual differences in sound intensity calculated according to equation 5.1 and expressed in dBA. The columns titled "% improvement" show the actual noise improvement ratio with respect to the pre-NGDG expressed in percentage, calculated according to equation 5.3.

Table 5. 8 Sound Intensity Improvements in dBA and in Percent

Lane and Segment	Pre-3mo (dBA)	Post-NGDG-3mo % improvement	Pre-6mo (dBA)	Post-NGDG-6mo % Improvement	Pre-Avg.Post (dBA)	Acg. Post-NGDG % Improvement
R1 average	**106.3**	**73.5%**	**106.4**	**75.5%**	**106.3**	**74.5%**
A	105.3	68.2%	105.4	68.9%	105.3	68.6%
B	106.7	75.7%	106.9	77.9%	106.8	76.8%
C	106.7	75.9%	106.8	78.7%	106.8	77.3%
R2 average			**106.6**	**77.2%**	**106.6**	**77.2%**
AA			106.2	74.5%	106.2	74.5%
BB			106.6	77.7%	106.6	77.7%
CC			107.0	79.1%	107.0	79.1%
L1 average	**106.1**	**71.3%**			**106.1**	**71.3%**
D	105.6	70.2%			105.6	70.2%
E	106.4	72.0%			106.4	72.0%
F	106.4	71.6%			106.4	71.6%
L2 average	**105.9**	**70.7%**	**106.2**	**75.9%**	**106.0**	**73.3%**
DD	105.8	71.1%	106.1	75.2%	106.0	73.2%
EE	106.5	72.8%			106.5	72.8%
FF	105.5	67.9%			105.5	67.9%

Table 5. 9 shows the reduction factor calculated according to equation 5.2 and interpreted as traffic reduction necessary for the pre-NGDG to achieve the post-NGDG noise levels. The last three columns show how many years of traffic growth at a 5% annual rate it would take for the post-NGDG pavement to cause the same noise as the pre-NGDG pavement causes with today's traffic.

The comparisons summarized in Table 5. 8 and Table 5. 9 confirm significant noise level improvements for all segments as well as for the lane averages. Sound intensity decreased from 105.3 dBA (lane R1, segment A, post-NGDG-3mo) to 107 dBA (lane R2, segment CC, post-NGDG-6mo). These differences mean percent decreases in sound intensity ranging from 67.9% to 79.1%. As indicated in Table 5. 9, the sound intensity level reductions depicted in Table 5. 8 as dBA and as percent changes correspond to traffic reduction factors ranging from 3.12 (lane L2, segment FF, post-NGDG-3mo) to 4.79 (R2, CC, 6mo). At a 5% annual traffic growth rate, this means it would take from 23 to 32 years for the post-NGDG pavement to cause the same noise as the pre-NGDG pavement with today's traffic.

Table 5. 9 Traffic Reduction Factors and Years to Reach Pre-NGDG Noise Level

Lane and Segment	Traffic Reduction Factor (RF)			Years to Reach Pre-NGDG Noise		
	Pre/3mo	*Pre/6mo*	*Pre/Avg.Post*	*Pre/3mo*	*Pre/6mo*	*Pre/Avg.Post*
R1 average	**3.78**	**4.09**	**3.92**	**27.2**	**28.8**	**28.0**
A	3.15	3.22	3.18	23.5	24.0	23.7
B	4.11	4.52	4.31	29.0	30.9	29.9
C	4.16	4.69	4.41	29.2	31.7	30.4
R2 average		**4.38**	**4.38**		30.3	30.3
AA		3.93	3.93		28.0	28.0
BB		4.48	4.48		30.8	30.8
CC		4.79	4.79		32.1	32.1
L1 average	**3.48**		**3.48**	25.6		25.6
D	3.35		3.35	24.8		24.8
E	3.57		3.57	26.1		26.1
F	3.52		3.52	25.8		25.8
L2 average	**3.41**	**4.15**	**3.75**	25.2	29.2	27.1
DD	3.47	4.03	3.73	25.5	28.6	27.0
EE	3.68		3.68	26.7		26.7
FF	3.12		3.12	23.3		23.3

Assumption: 5% annual traffic growth rate

Overall Comparison—Entire Test Section

Figure 5. 8 shows a comparison between the pre-NGDG noise levels and the post-NGDG noise levels energy-averaged for the entire test section. It is opportune to remind the reader that the preliminary analysis indicates that such data aggregation is statistically meaningful.

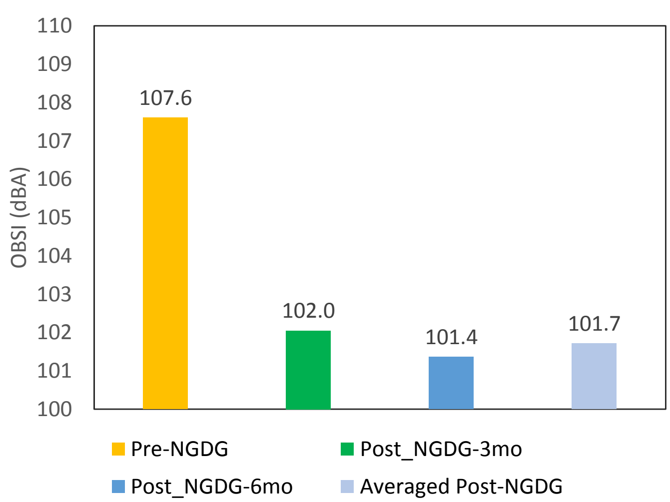


Figure 5. 8 Overall Average Sound Intensity Levels

The direct decibel subtraction is always greater than 5.0, the threshold for "noticeable change" indicated in Table 5. 6.The overall post-NGDG noise level was 101.7 dBA. According to a study by the Iowa State University's National Concrete Pavement Technology Center (2006), this test section stayed in noise Zone 2, the "quality zone." The study also defined a quieter Zone 1, calling it "innovation zone." The study added the following caveat to Zone 1: "with the exception of some experimental pervious concrete pavements, there were no concrete solutions in Zone 1. It appears that conventional (dense) concrete may not have the ability to be built consistently in Zone 1. Research and innovation will therefore be required to develop solutions that consistently provide OBSI levels within the zone."(NCPTC, Iowa State University 2006). In short, the Loop 610 test section is as quiet as possible with today's concrete pavements.

The actual reduction in sound intensity calculated by subtracting the overall pre- and post-NGDG energy averages was 106.3 dBA, and the average traffic reduction factor is 4.08. Assuming a 5% annual traffic growth rate, it would take 28.8 years for the NGDG surface to cause the same noise as the pre-NGDG with today's traffic.

Discussion, Conclusions and Recommendations

Discussion

TxDOT informed that all data already came normalized according to AASTHO's standards, so the entire analysis documented in this chapter was done without data adjustments listed in AASHTO 76 (2015). TxDOT reported the air temperatures during the three data collection dates. It is interesting to note that the unadjusted post-NGDG-6mo data (July, 95°F) are consistently less than the post-NGDG-3mo (March, 73°F). This difference disappears when the overall average data are normalized for temperature according to the instructions in item 6.15.2, page TP-76-7 of AASHTO 76 (2015). Figure 5. 9 shows the same results as Figure 5. 8 after applying AASHTO's temperature normalization.

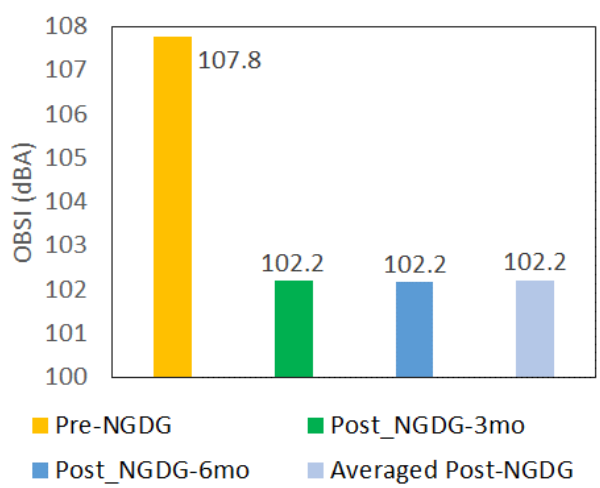


Figure 5. 9 Overall Average Sound Intensities, Temperature-Normalized

This discussion is academically interesting but does not affect this study's conclusions and recommendations, since there is little change between normalized and un-normalized sound intensity improvements.

Summary of Conclusions

- An Iowa University study defined noise zones for pavements, with Zone 1, "innovation zone," being the quietest, followed by Zone 2, "quality zone". The NGDG fell in Zone 2, which according to this study is as quiet as possible for today's concrete pavements. Zone 1 came with the caveat that "It appears that conventional (dense) concrete may not have the ability to be built consistently in Zone 1. Research and innovation will therefore be required to develop solutions that consistently provide OBSI levels within the zone." (Iowa State University's National Concrete Pavement Technology Center 2006).
- The difference in decibel (by direct subtraction) is always greater than 5.0, the threshold for "noticeable change" indicated in Table 5. 6 (Wirth 2008). This is true for every lane and for the overall test section.
- The actual sound intensity decreases ranged from 105.3 dBA (lane R1, segment A, post-NGDG-3mo) to a maximum of 107 dBA (lane R2, segment CC, post-NGDG-6mo). The overall average sound intensity in the test section decreased by 106.3 dBA. Considering that a sports event crowd averages 105 dB and a rock band, 110 dB (noisehelp.com, 2016, Sandberg et al, 2002), it is clear that removing 106.3 dBA from the environment with NGDG is a very significant improvement in noise level.
- The observed sound intensity reductions result in percent decreases in noise ranging from 67.9% to 79.1%.
- The ratios of before / after sound intensities ranged from 3.12 (lane L2, segment FF, post-NGDG-3mo) to 4.79 (R2, CC, 6mo). The overall test section average was 4.08. Assuming that all vehicles cause the same noise, these ratios can be interpreted as traffic reduction factors necessary for the pre-NGDG surface to be as quiet as the NDGD. In other words, the pre-NGDG test section would emit as much noise as the post-NGDG section when carrying only ¼ of the traffic.
- Assuming a 5% annual traffic growth rate, the abovementioned ratios can be expressed as years of traffic growth. The overall test section average reduction factor of 4.08 means that the NGDG would cause the same noise as the pre-NGDG does with today's traffic after 28.8 years of steady traffic growth.

Recommendations

The initial scope of this study included two test sections with complete sound data factorials: Loop 610 and US290. Due to changes in construction schedules, US290 did not receive NGDG within this study's time frame. Moreover, lane closures on Loop 610 precluded a complete factorial for the sound data. Surveying US290 after NGDG completion is highly recommended as a cost effective way to expand the scope of this study while taking advantage of the pre-NGDG data collected in November 2014.

The literature mentions other types of longitudinal tining that can also be rather quiet (while mentioning that NGDG is quieter). It is recommended to design a comprehensive factorial to compare NGDG to other PCCP surfaces that may be of interest in terms of noise control, especially if NGDG cost differential with respect to other treatments is a consideration.

This study schedule did not allow proper durability evaluation. The two post-NGDG data sets are about 4 months apart, not enough time for concrete pavement surface treatments to deteriorate. It is therefore recommended to collect post-NGDG data after 1 and 2 years of traffic. Annual data collection for 5 consecutive years would provide enough data for a time-series durability analysis.

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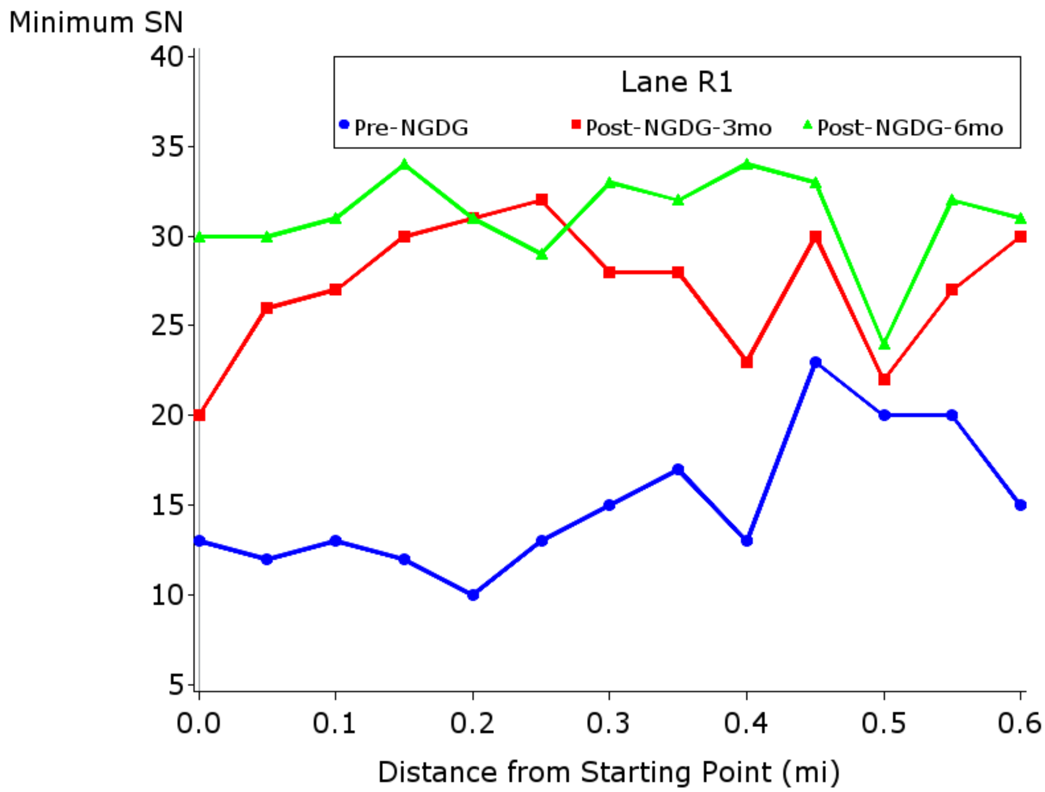
Appendix 1

Comparative Plots of

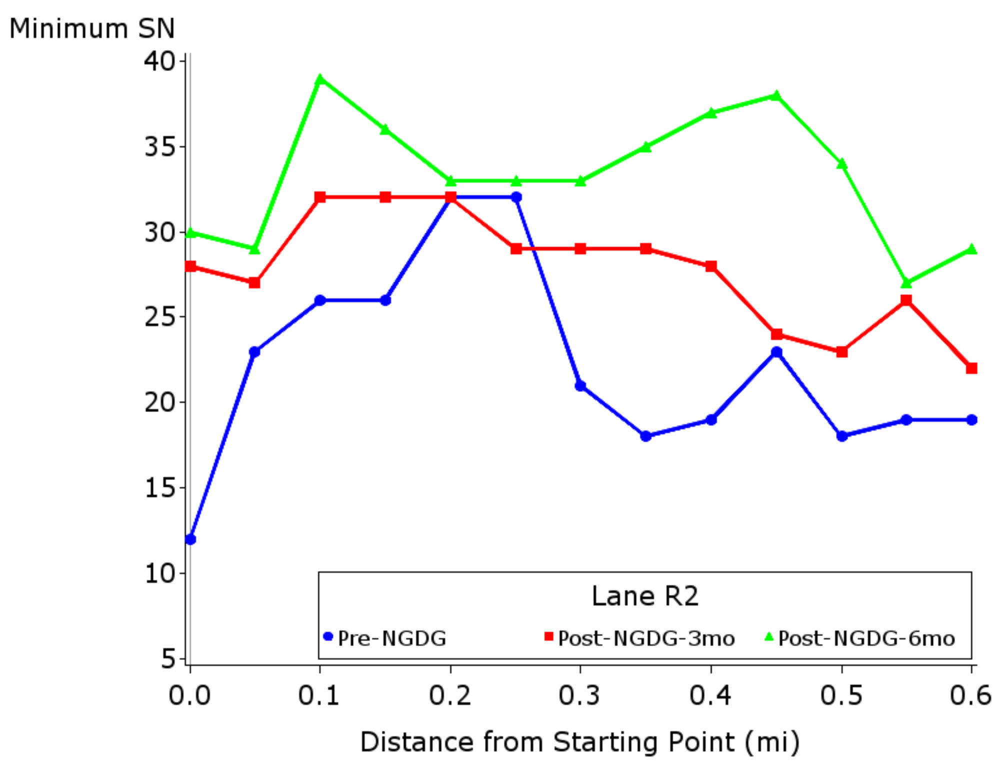
Skid Number and Percent Slip Data

Summary of Plots

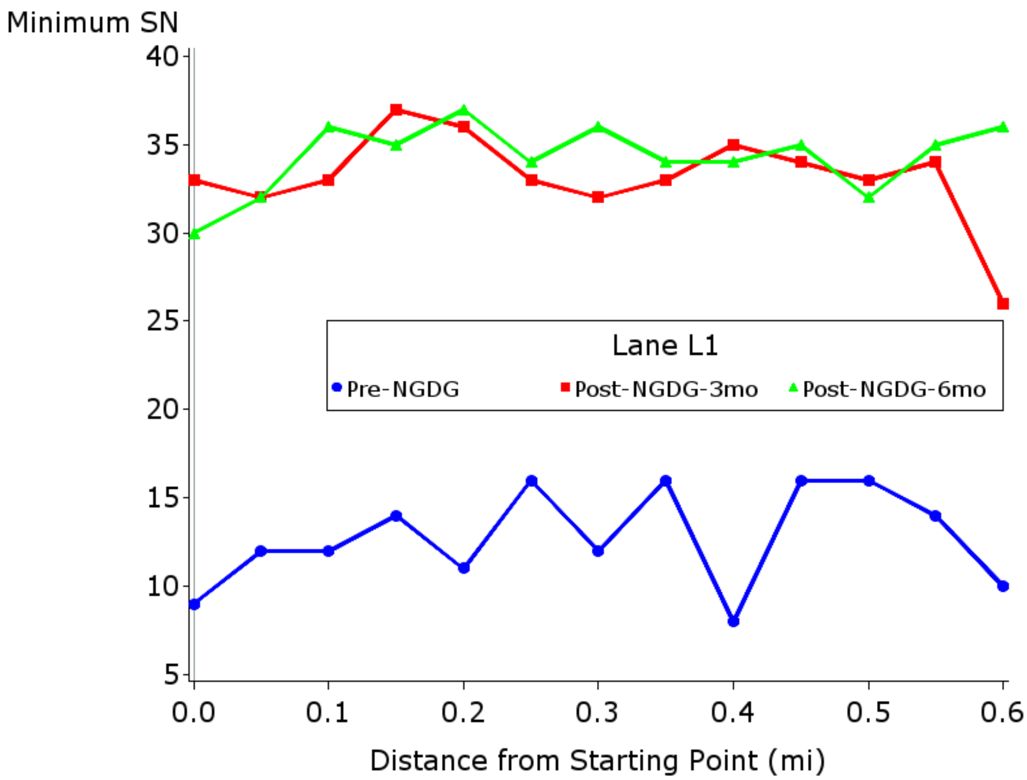
Data	**Lane**			
	R1	*R2*	*L1*	*L2*
Minimum SN	✓	✓	✓	✓
Average SN	✓	✓	✓	✓
Maximum SN	✓	✓	✓	✓
Peak	✓	✓	✓	✓
Percent Slip	✓	✓	✓	✓

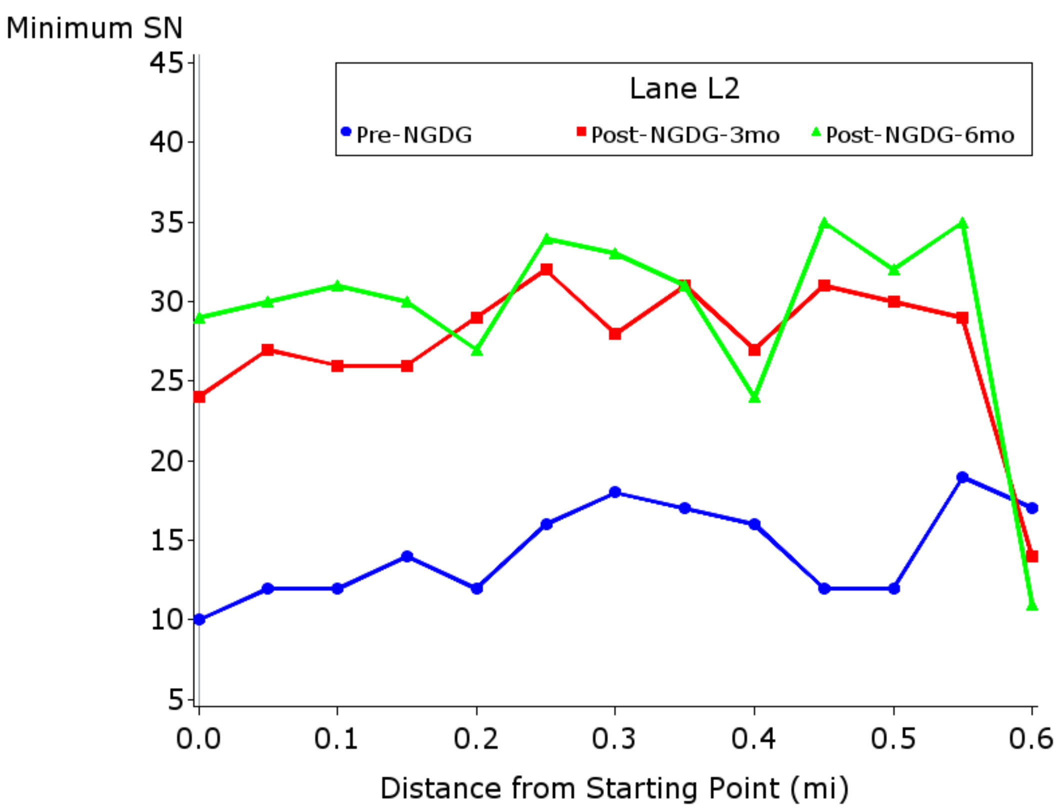


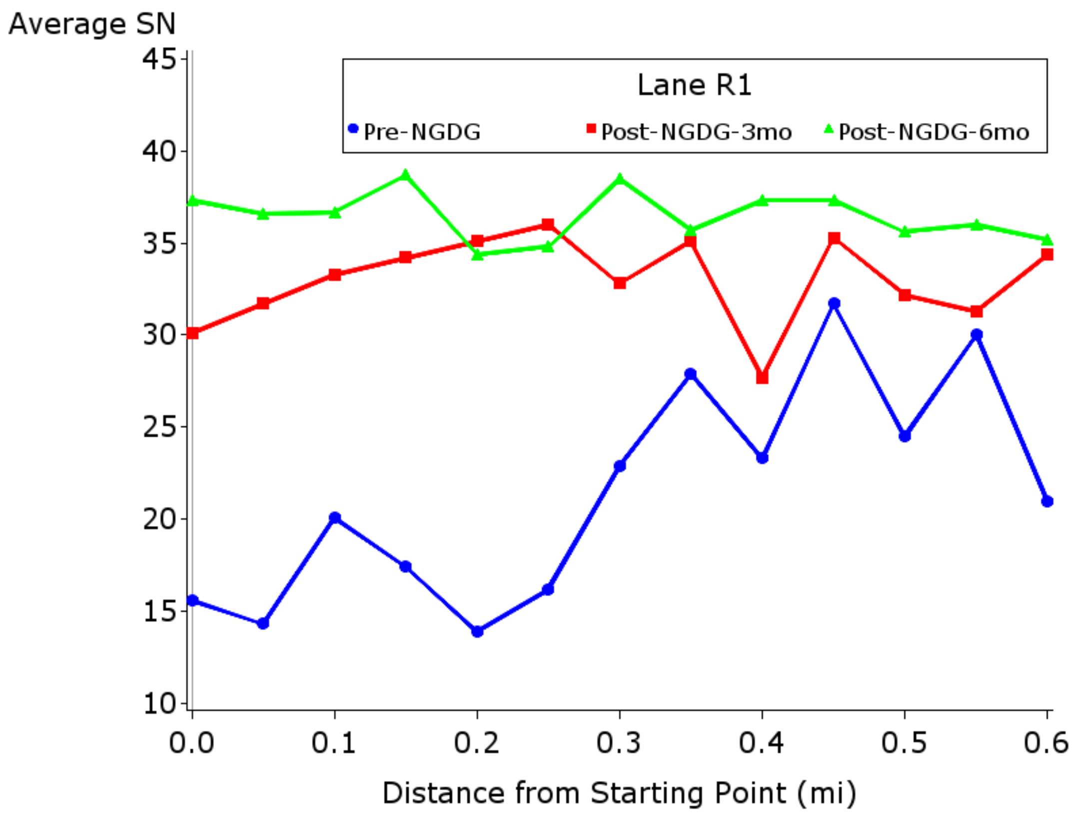
Minimum SN
Lane R1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
40
35
30
25
20
15
10
5
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)



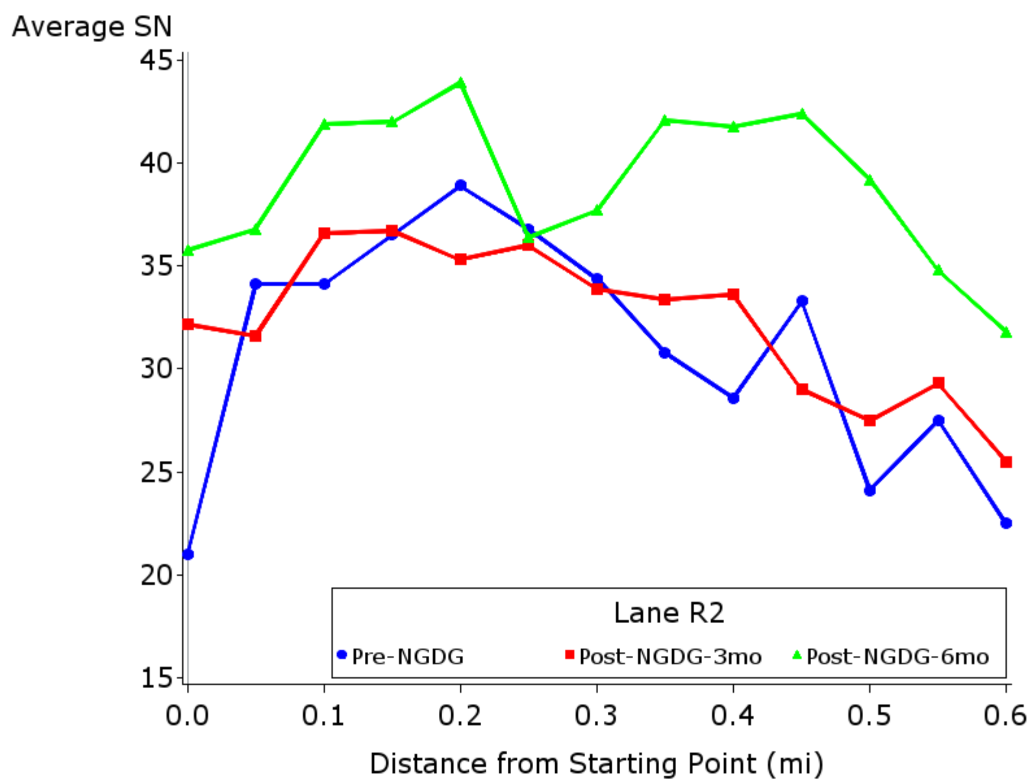
Minimum SN
Lane R2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
40
35
30
25
20
15
10
5
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)


Minimum SN
40
35
30
25
20
15
10
5
Lane L1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

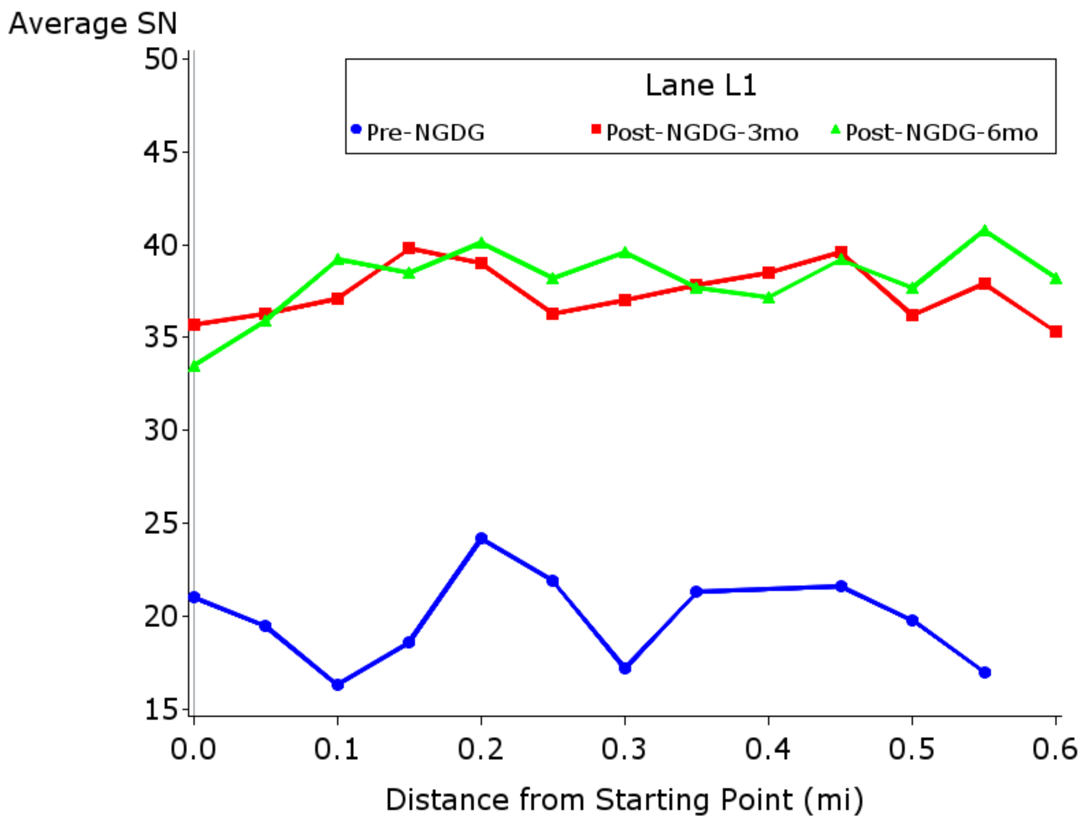

Minimum SN
45
40
35
30
25
20
15
10
5
Lane L2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

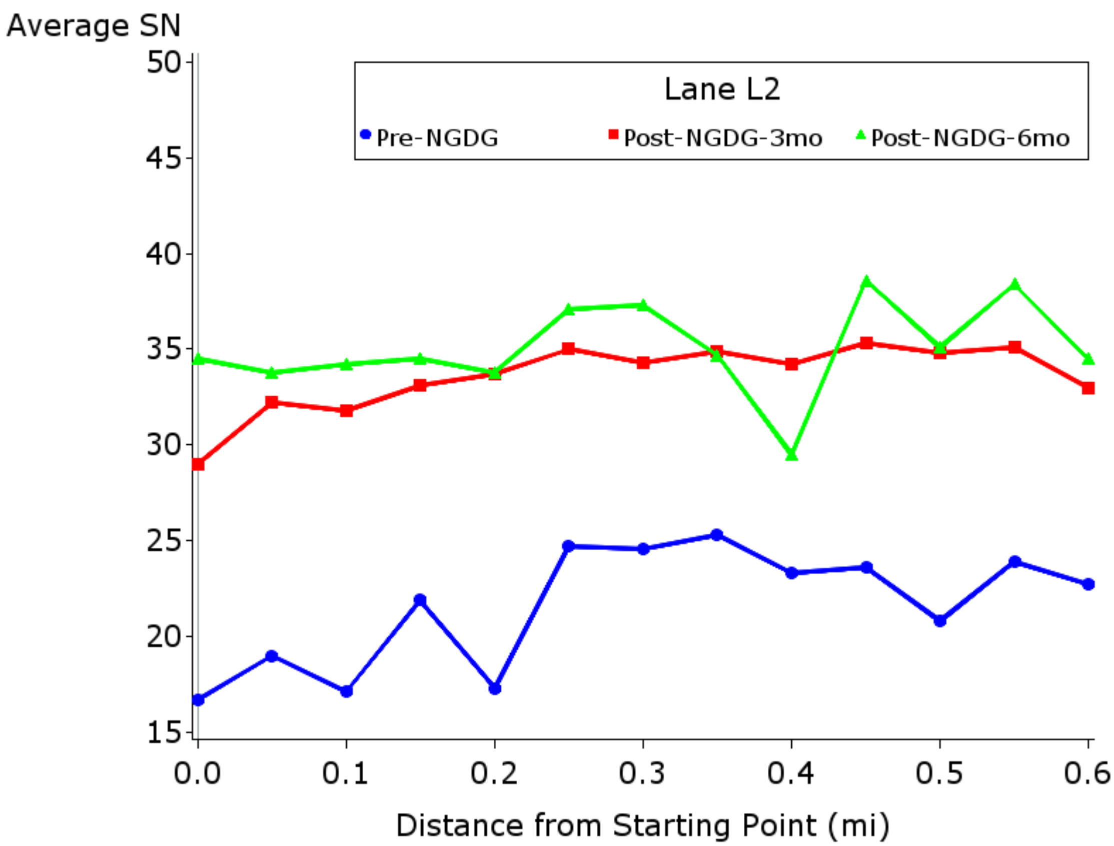


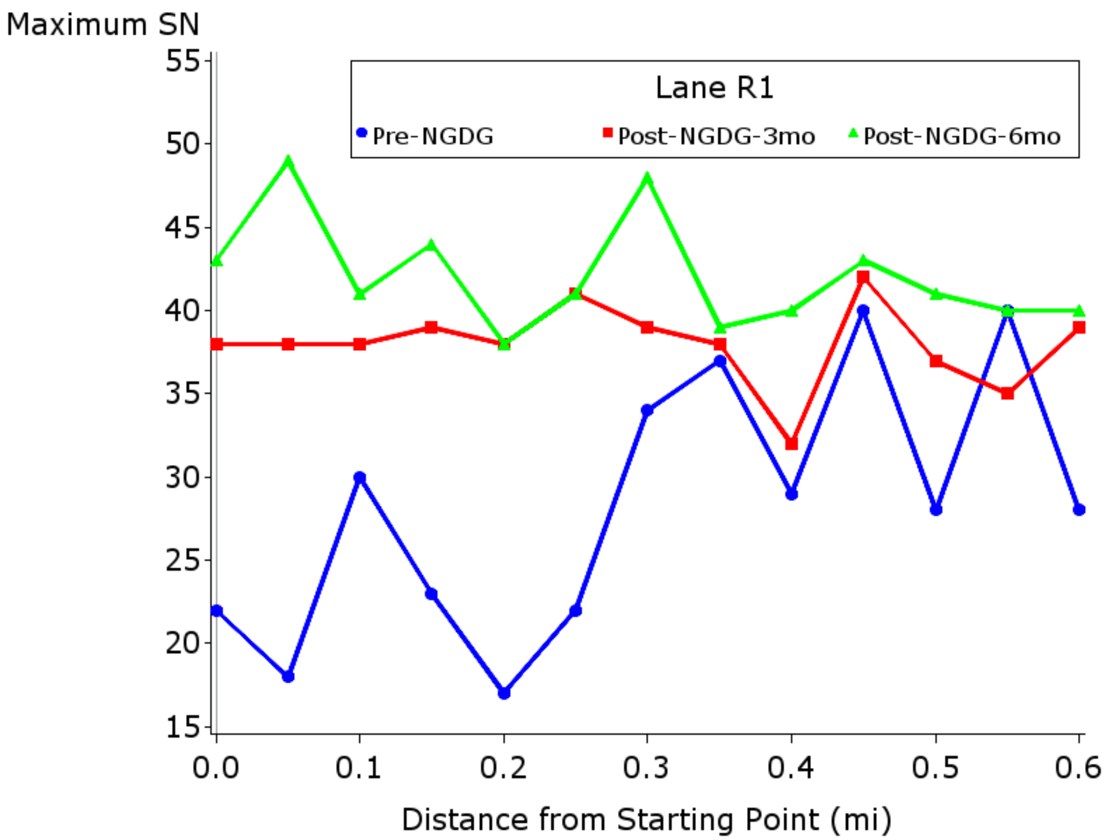
Average SN
Lane R1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
45
40
35
30
25
20
15
10
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

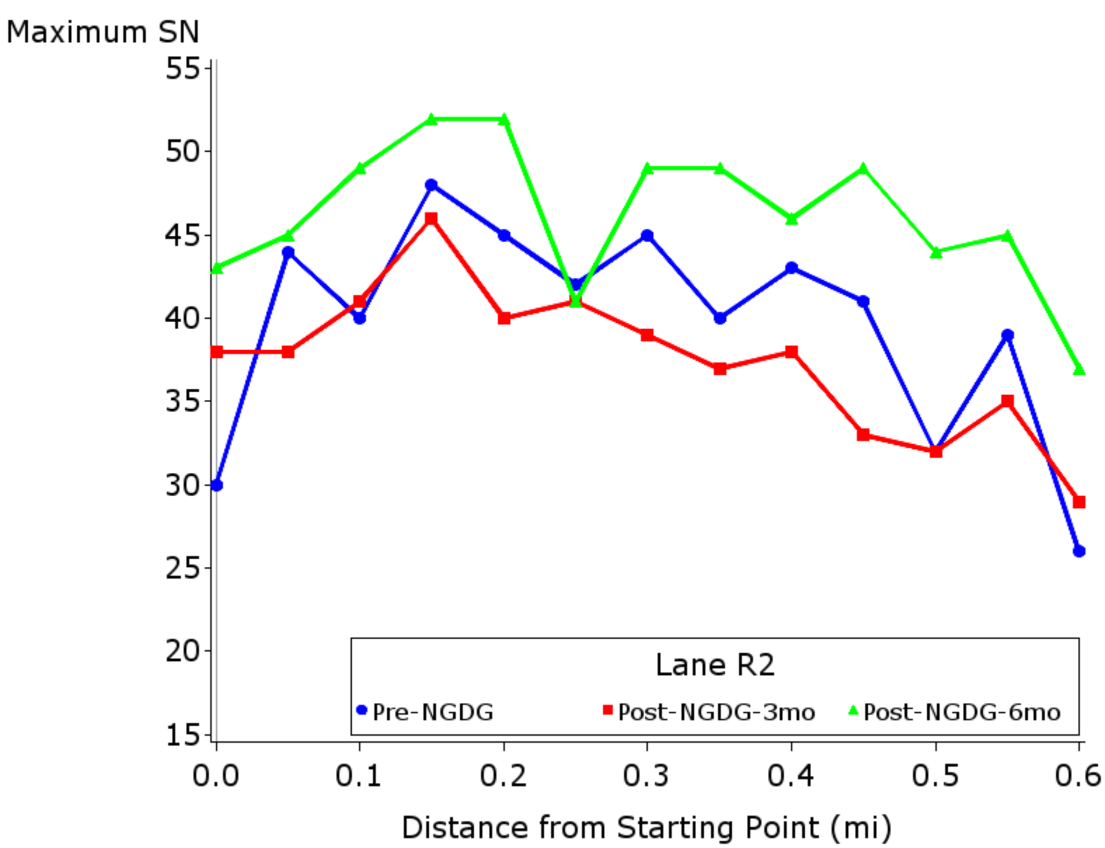


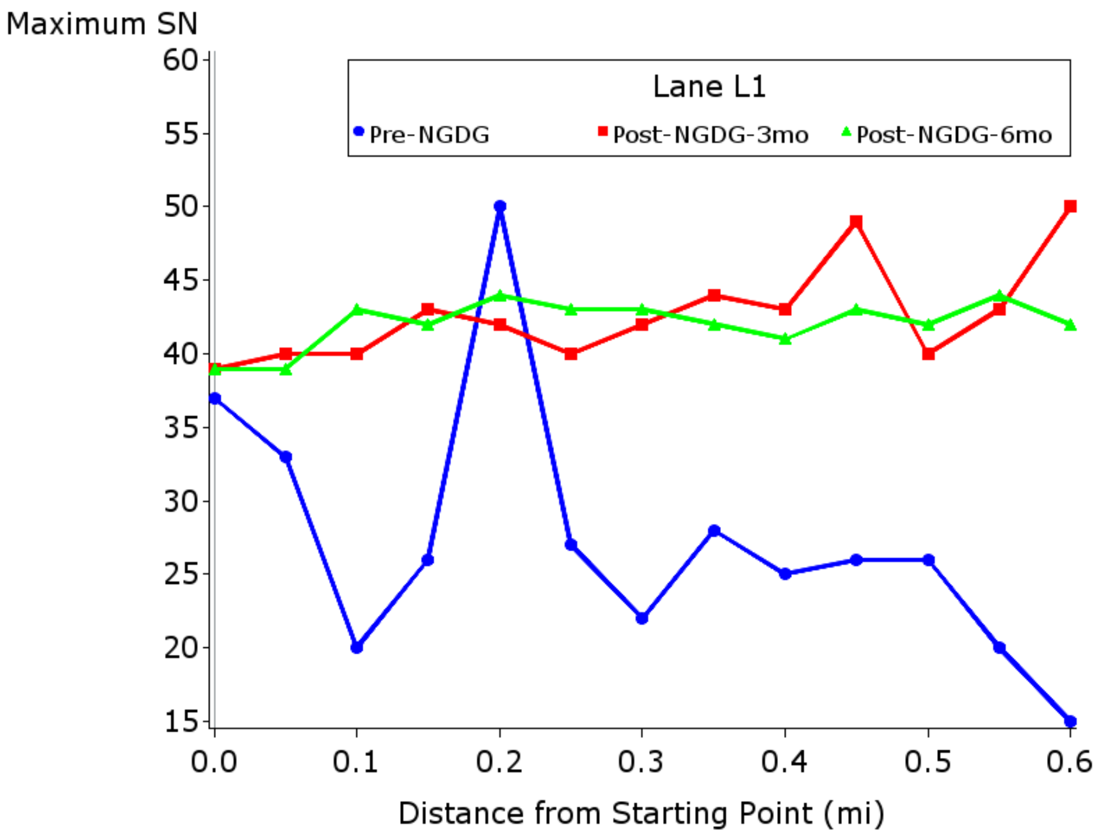
Average SN
Lane R2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
45
40
35
30
25
20
15
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

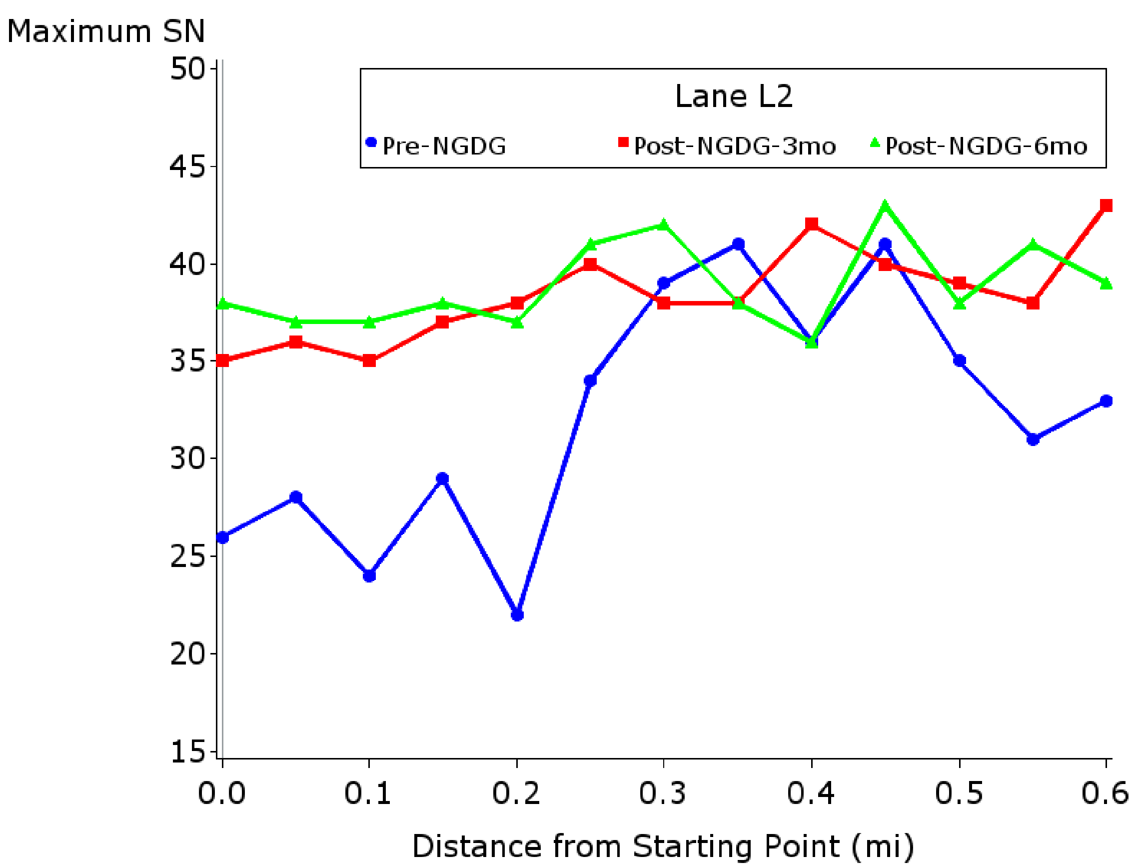

Average SN
Lane L1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
50
45
40
35
30
25
20
15
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

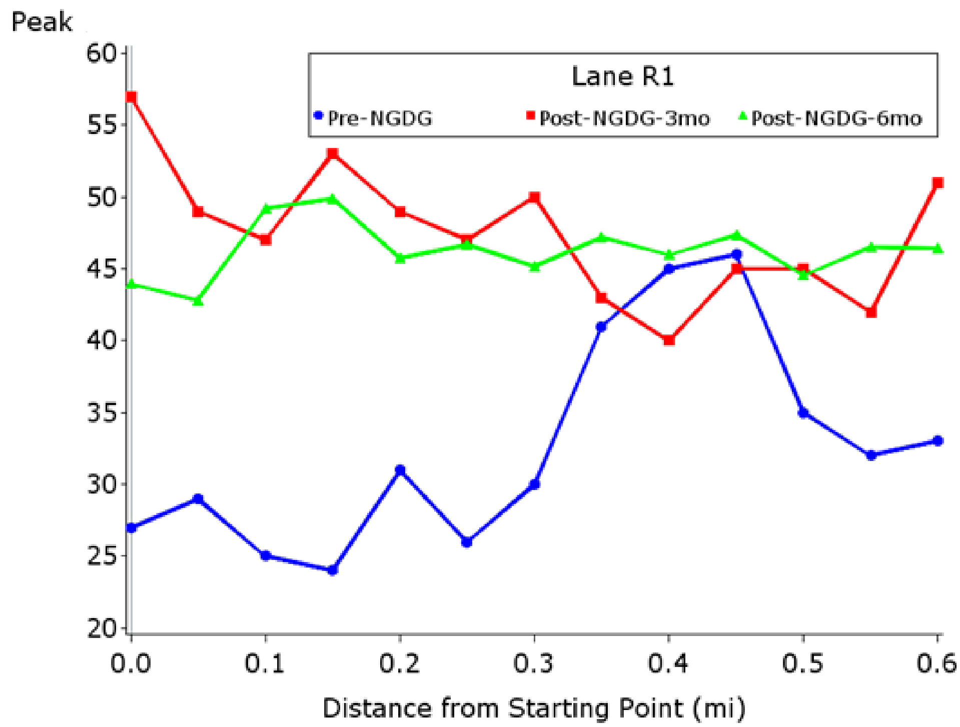

Average SN
Lane L2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
50
45
40
35
30
25
20
15
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

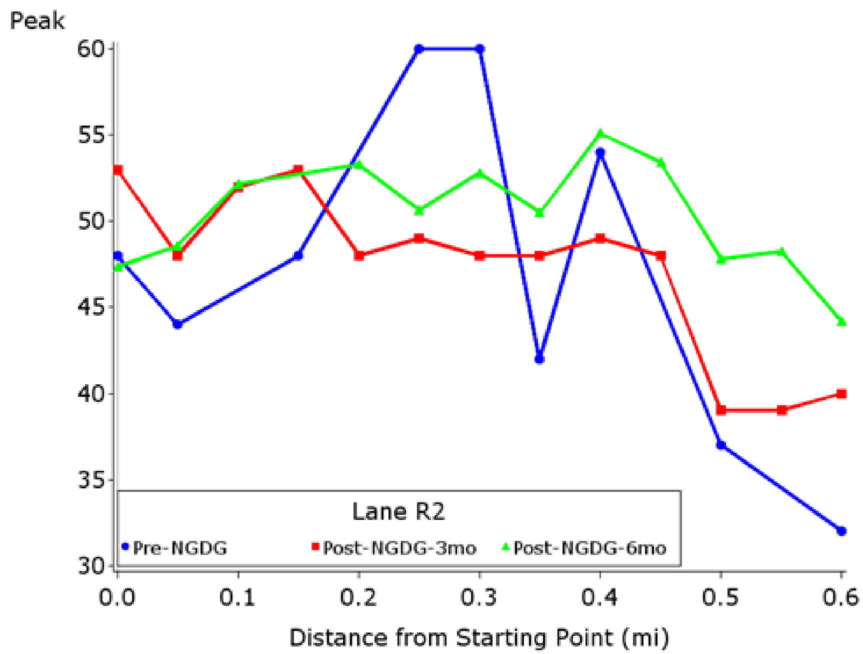

Maximum SN
55
50
45
40
35
30
25
20
15
Lane R1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

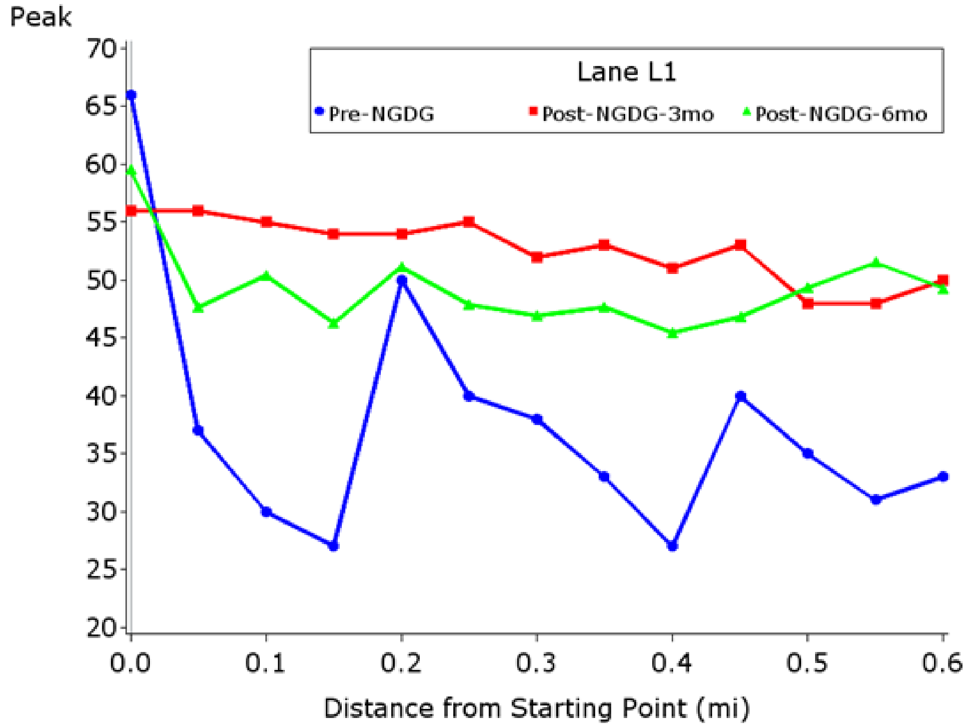

Maximum SN
55
50
45
40
35
30
25
20
15
Lane R2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

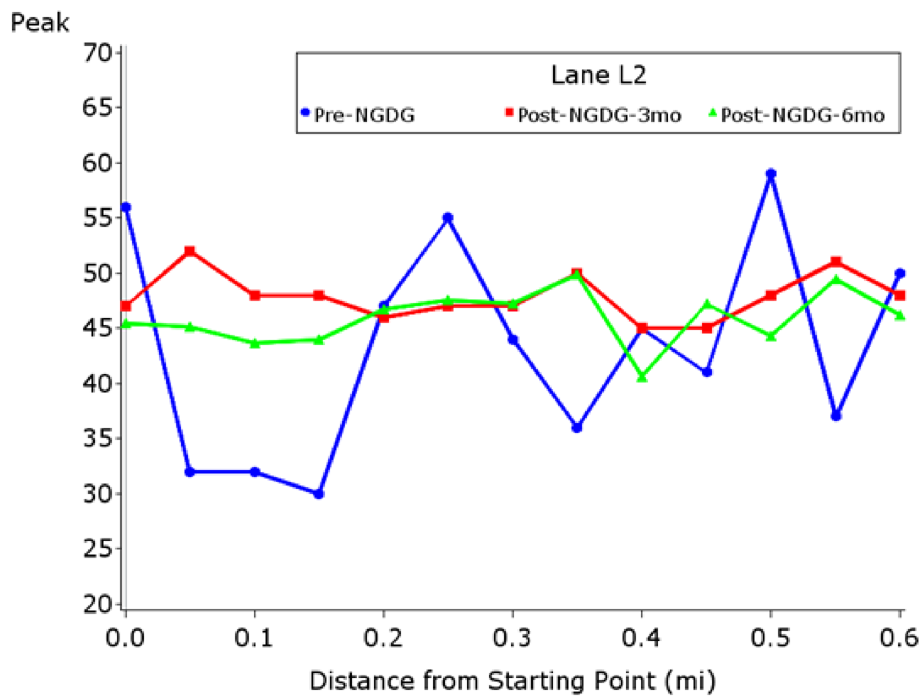

Maximum SN
Lane L1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
60
55
50
45
40
35
30
25
20
15
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

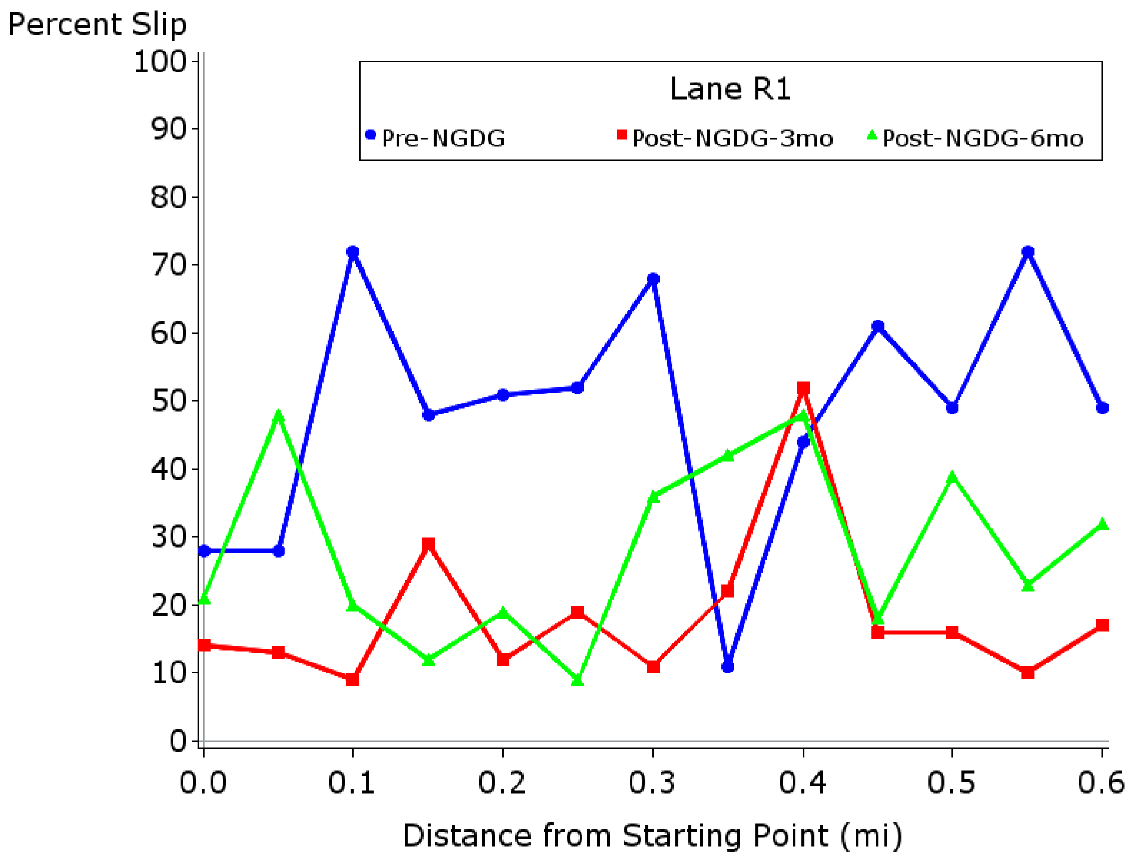

Maximum SN
Lane L2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
50
45
40
35
30
25
20
15
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

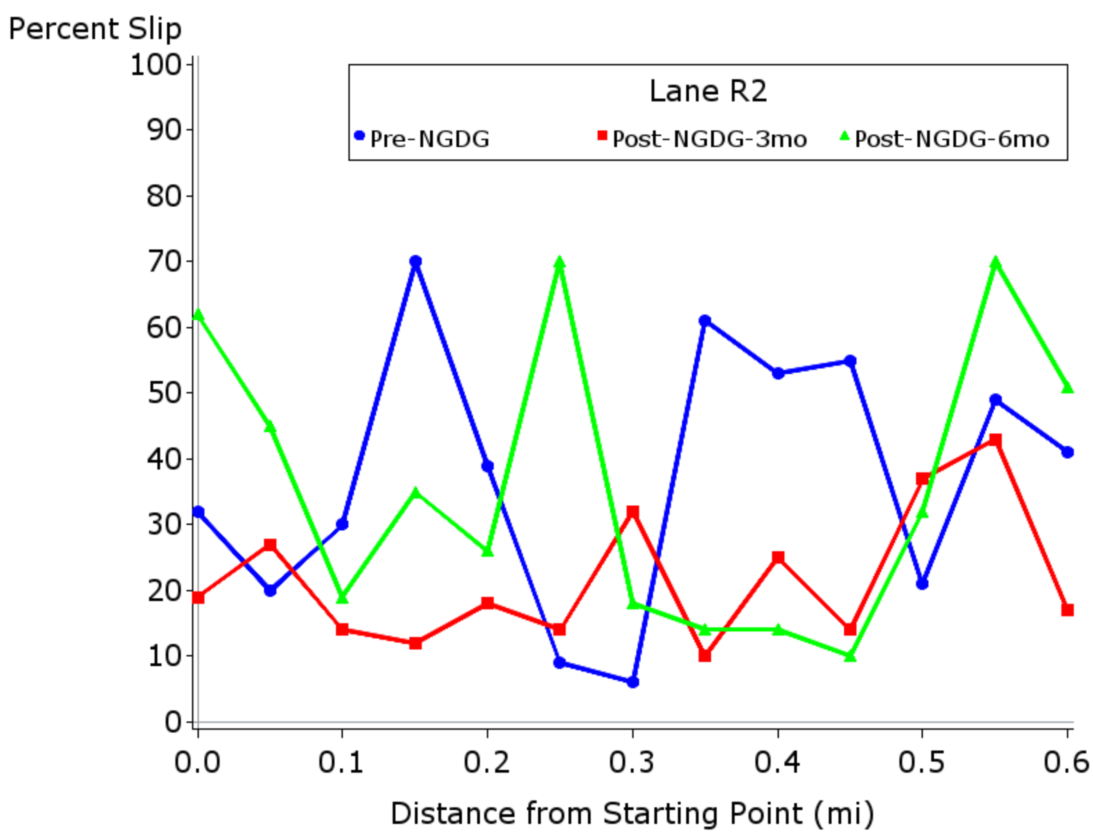

Peak
Lane R1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
60
55
50
45
40
35
30
25
20
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

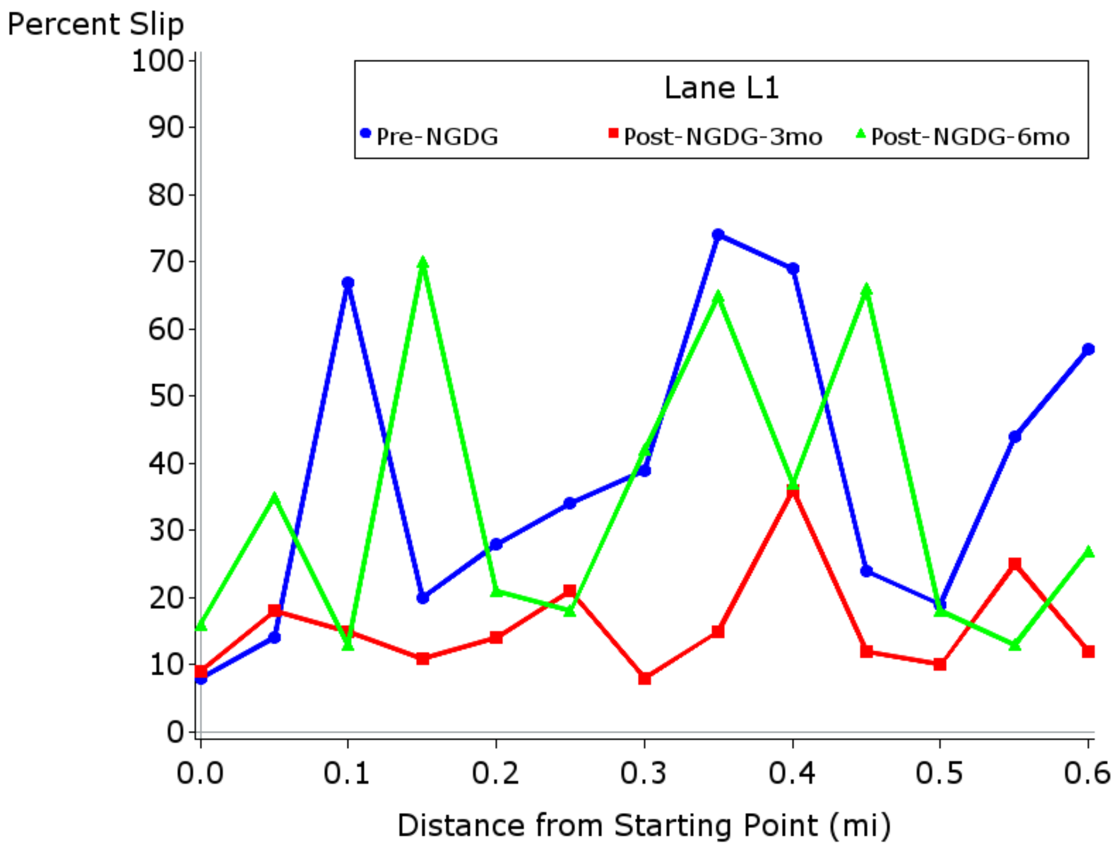

Peak
Lane R2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
60
55
50
45
40
35
30
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

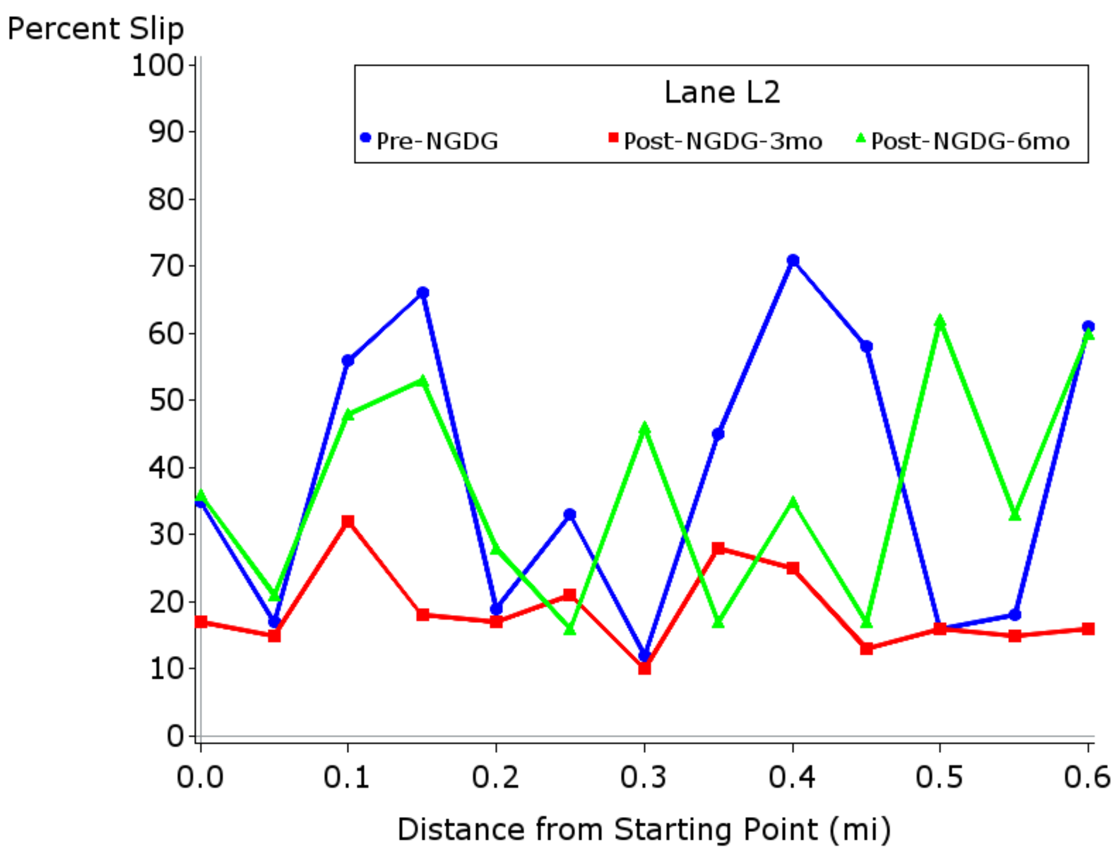

Peak
70
65
60
55
50
45
40
35
30
25
20
Lane L1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)


Peak
70
65
60
55
50
45
40
35
30
25
20
Lane L2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)


Percent Slip
100
90
80
70
60
50
40
30
20
10
0
Lane R1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)


Percent Slip
100
90
80
70
60
50
40
30
20
10
0
Lane R2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)


Percent Slip
100
90
80
70
60
50
40
30
20
10
0
Lane L1
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)


Percent Slip
100
90
80
70
60
50
40
30
20
10
0
Lane L2
Pre-NGDG
Post-NGDG-3mo
Post-NGDG-6mo
0.0
0.1
0.2
0.3
0.4
0.5
0.6
Distance from Starting Point (mi)

Appendix 2

Comparative Plots of

International Roughness Index (IRI)

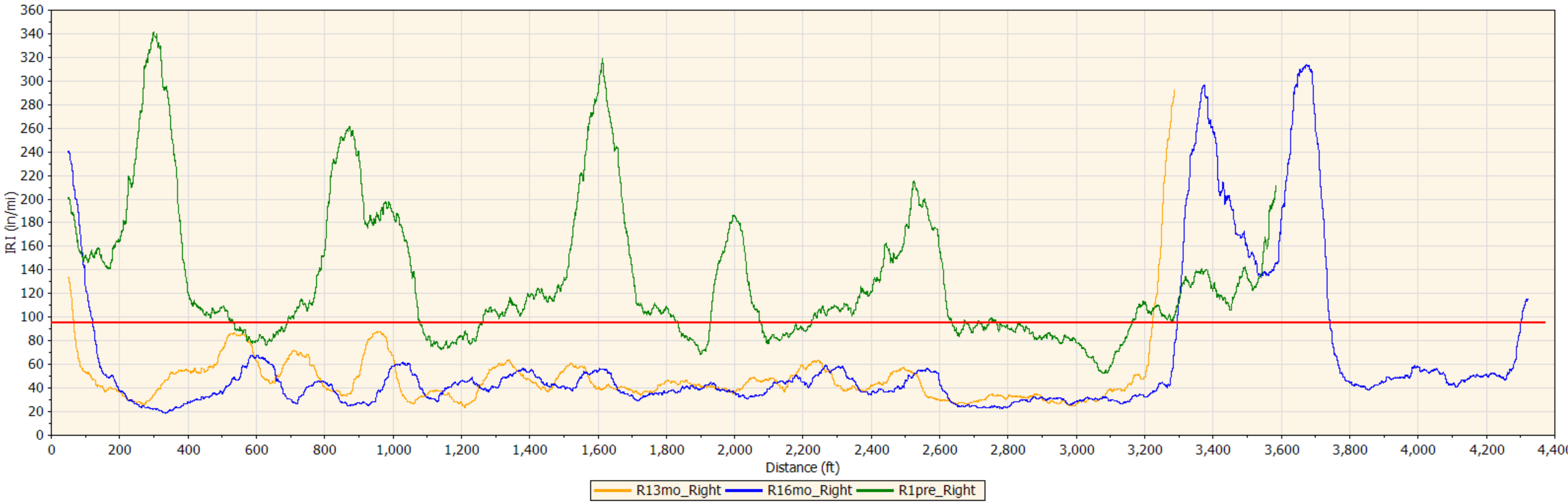
and

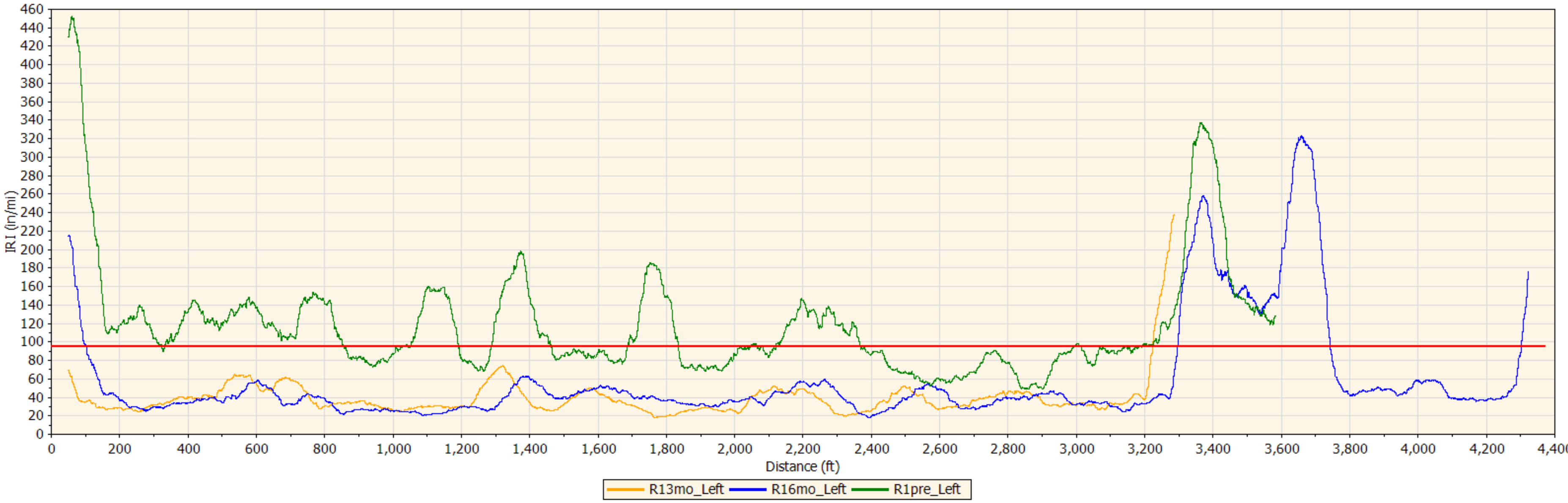
Ride Number (RN)

Summary of Plots

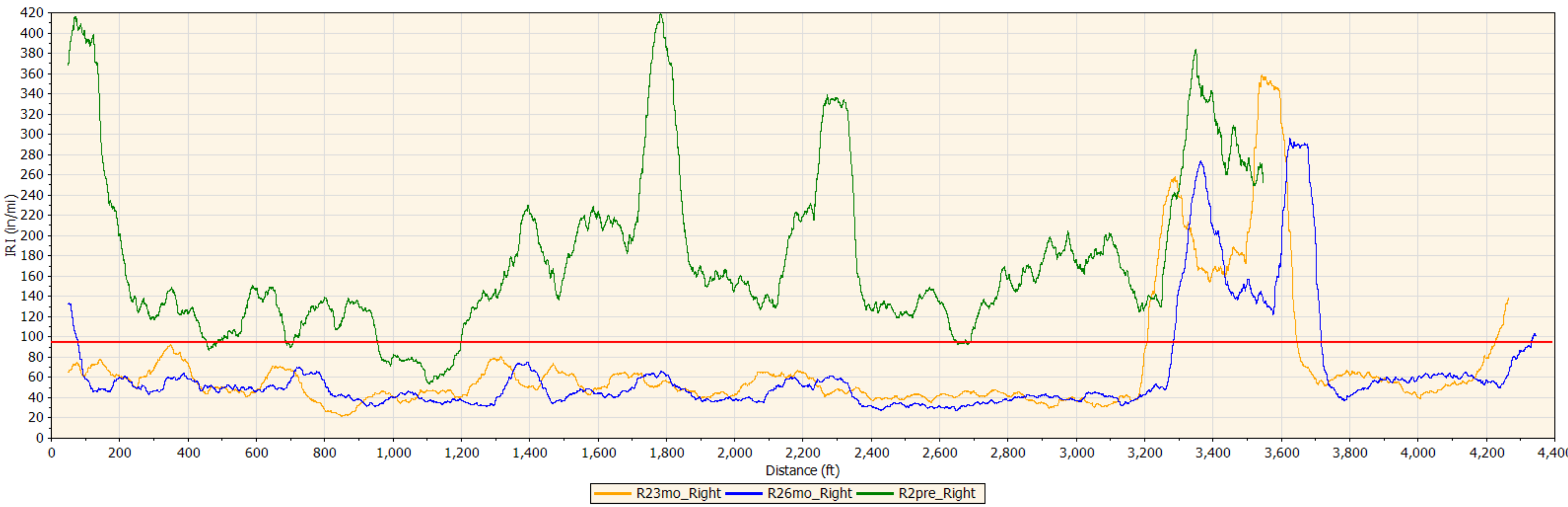
Data	**Lane**			
	R1	R2	L1	L2
Left and Right Wheel Path IRI	✓	✓	✓	✓
RN	✓	✓	✓	✓

International Roughness Index (IRI)

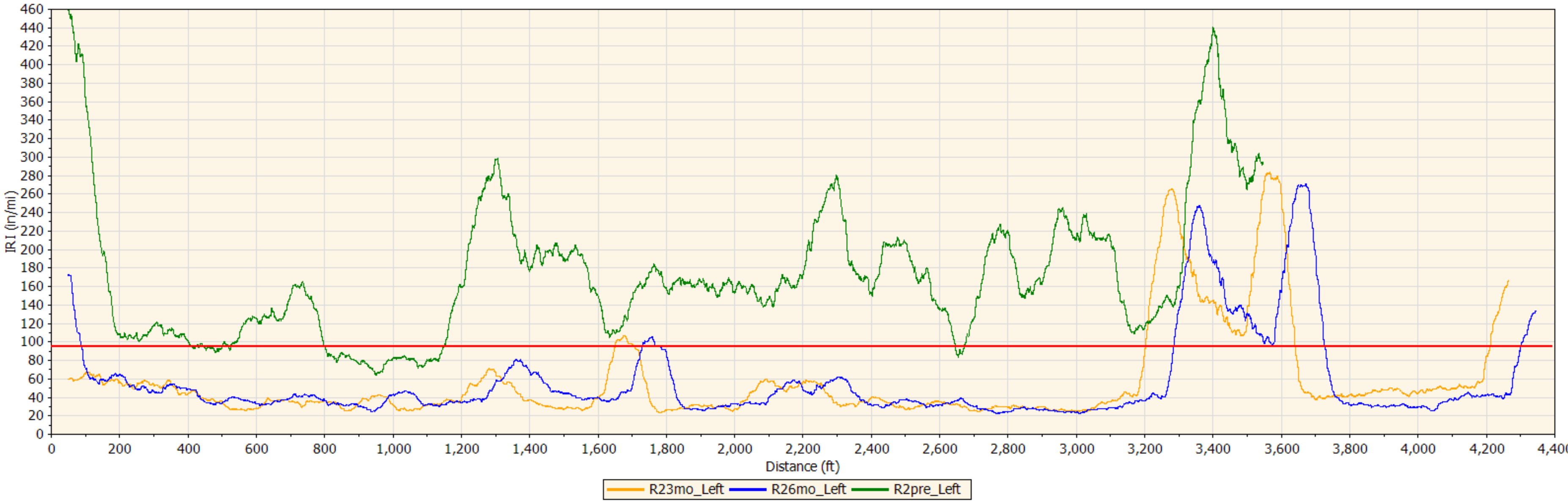


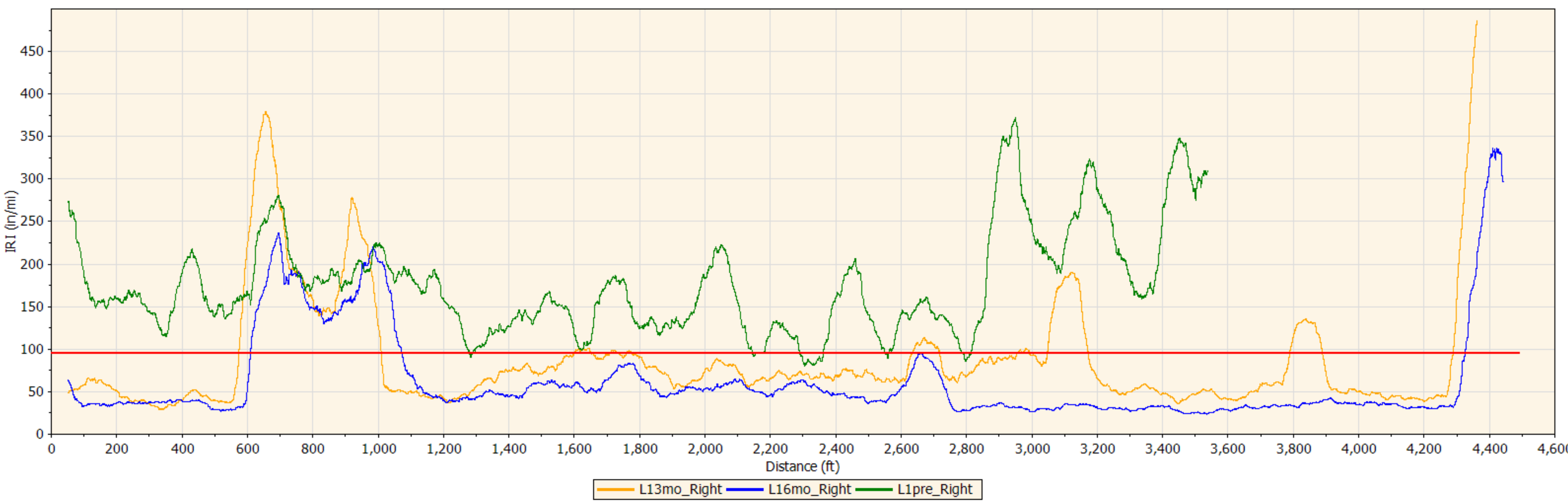


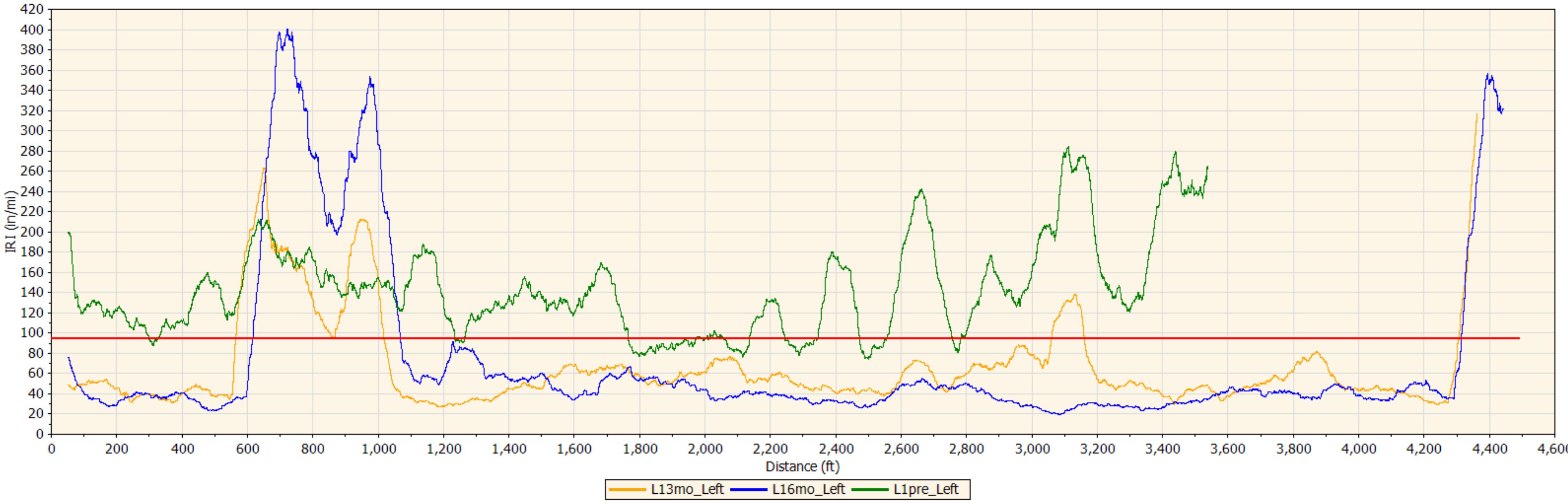
460
440
420
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
IRI (in/mi)
0
200
400
600
800
1,000
1,200
1,400
1,600
1,800
2,000
2,200
2,400
2,600
2,800
3,000
3,200
3,400
3,600
3,800
4,000
4,200
4,400
Distance (ft)
R13mo_Left
R16mo_Left
R1pre_Left

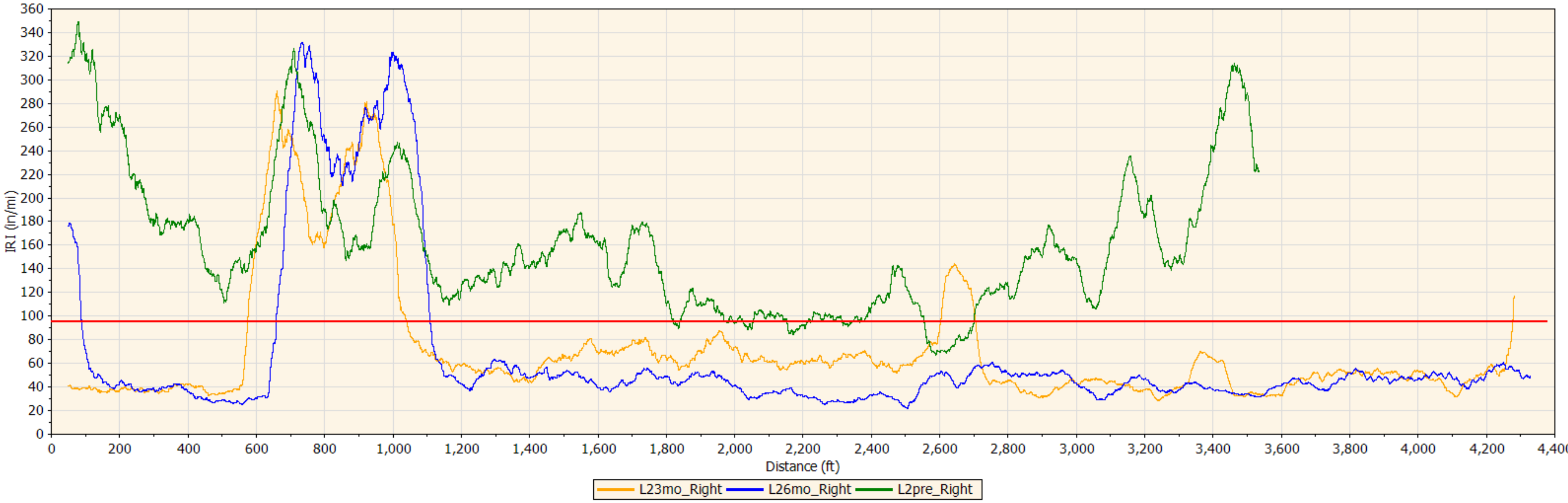


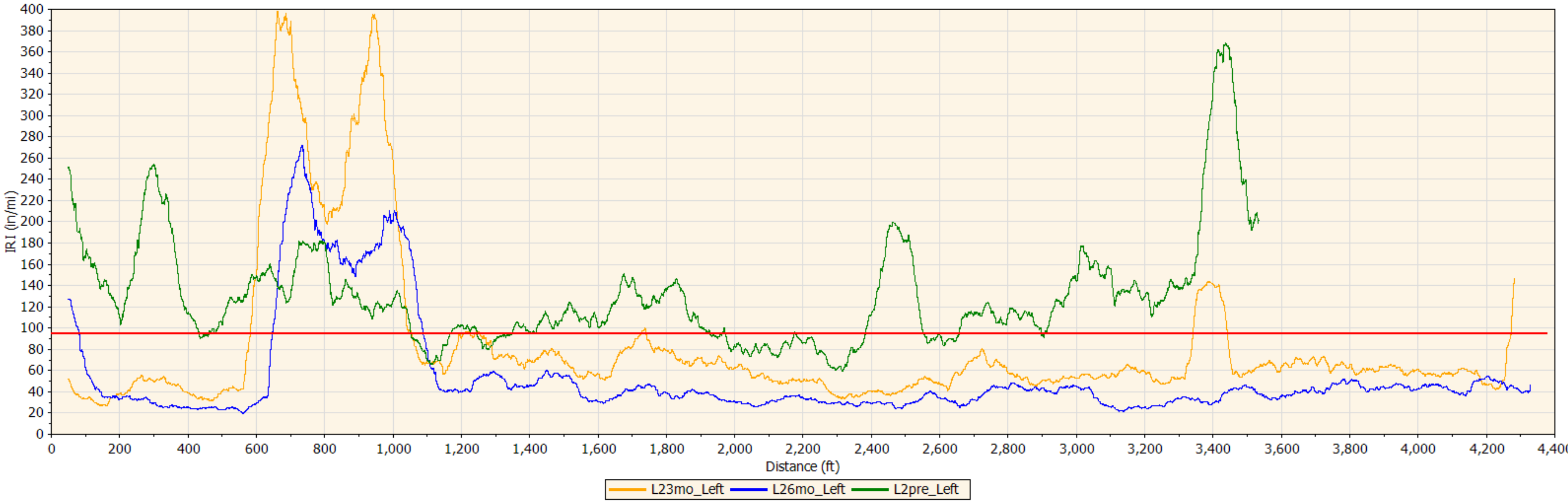
420
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
IRI (in/mi)
0
200
400
600
800
1,000
1,200
1,400
1,600
1,800
2,000
2,200
2,400
2,600
2,800
3,000
3,200
3,400
3,600
3,800
4,000
4,200
4,400
Distance (ft)
R23mo_Right
R26mo_Right
R2pre_Right


IRI (in/mi)
460
440
420
400
380
360
340
320
300
280
260
240
220
200
180
160
140
120
100
80
60
40
20
0
0
200
400
600
800
1,000
1,200
1,400
1,600
1,800
2,000
2,200
2,400
2,600
2,800
3,000
3,200
3,400
3,600
3,800
4,000
4,200
4,400
Distance (ft)
R23mo_Left
R26mo_Left
R2pre_Left


IRI (in/mi)
450
400
350
300
250
200
150
100
50
0
0
200
400
600
800
1,000
1,200
1,400
1,600
1,800
2,000
2,200
2,400
2,600
2,800
3,000
3,200
3,400
3,600
3,800
4,000
4,200
4,400
4,600
Distance (ft)
L13mo_Right
L16mo_Right
L1pre_Right

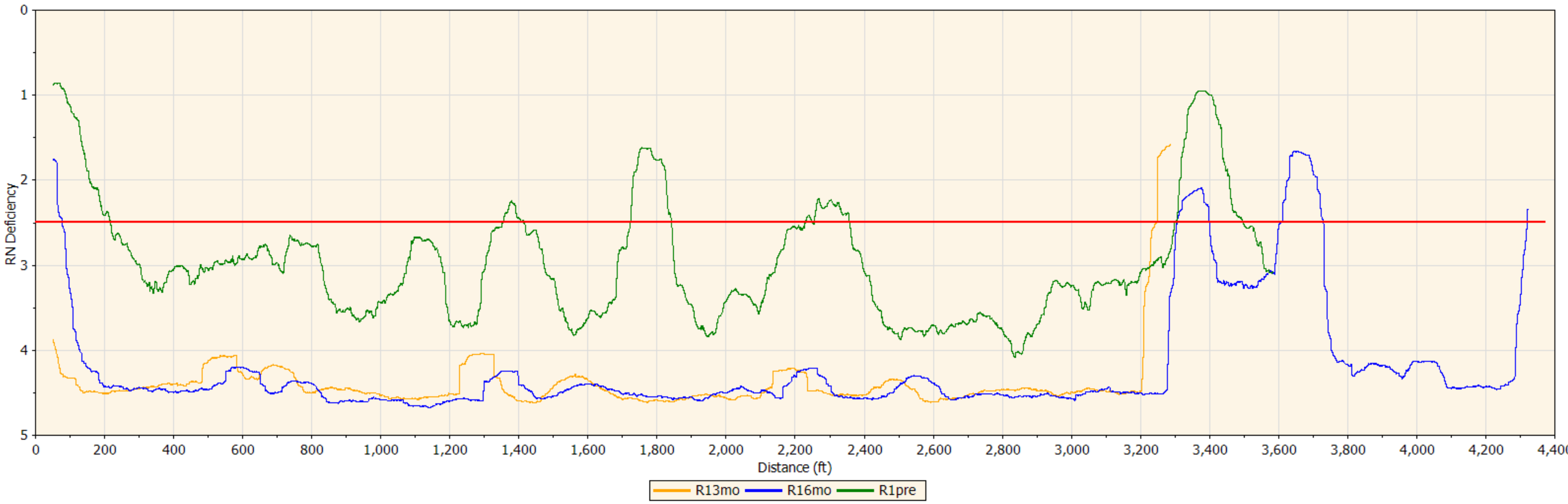

IRI (in/mi)
Distance (ft)
L13mo_Left
L16mo_Left
L1pre_Left

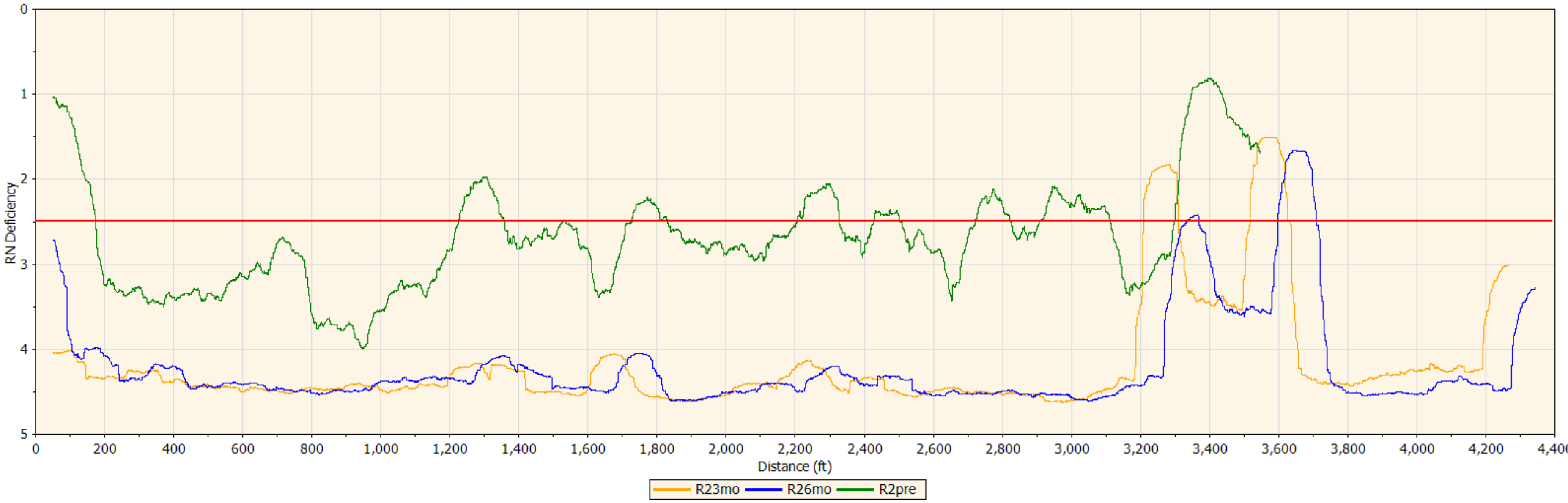

IRI (in/mi)
Distance (ft)
L23mo_Right
L26mo_Right
L2pre_Right

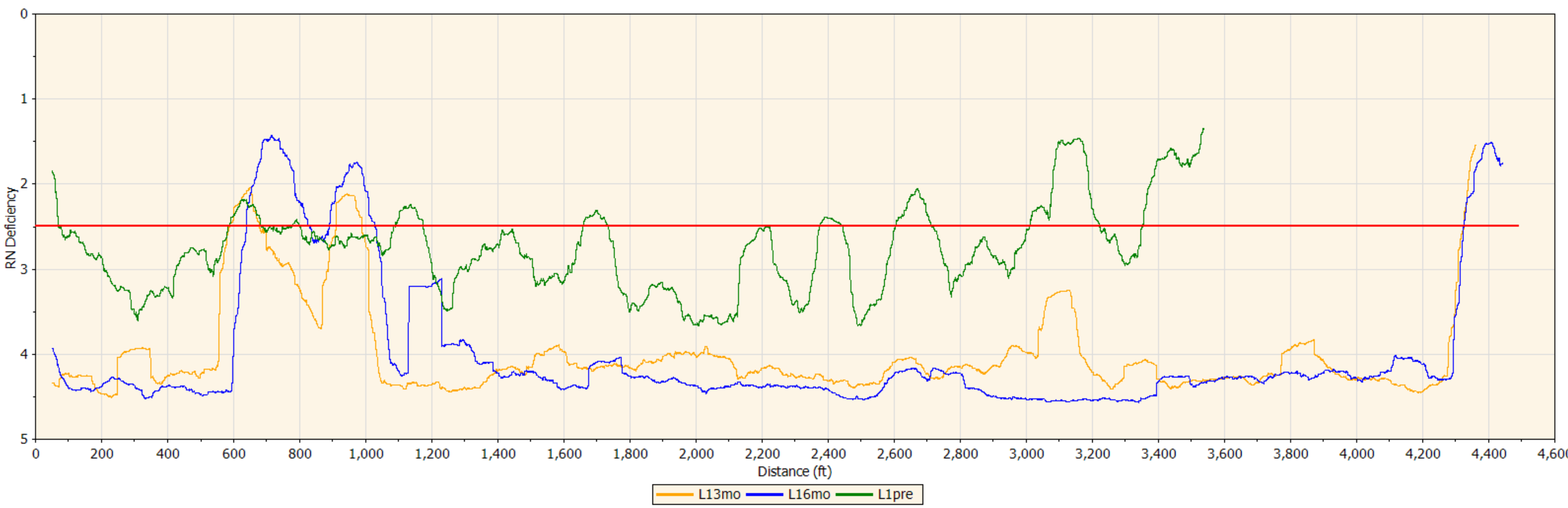


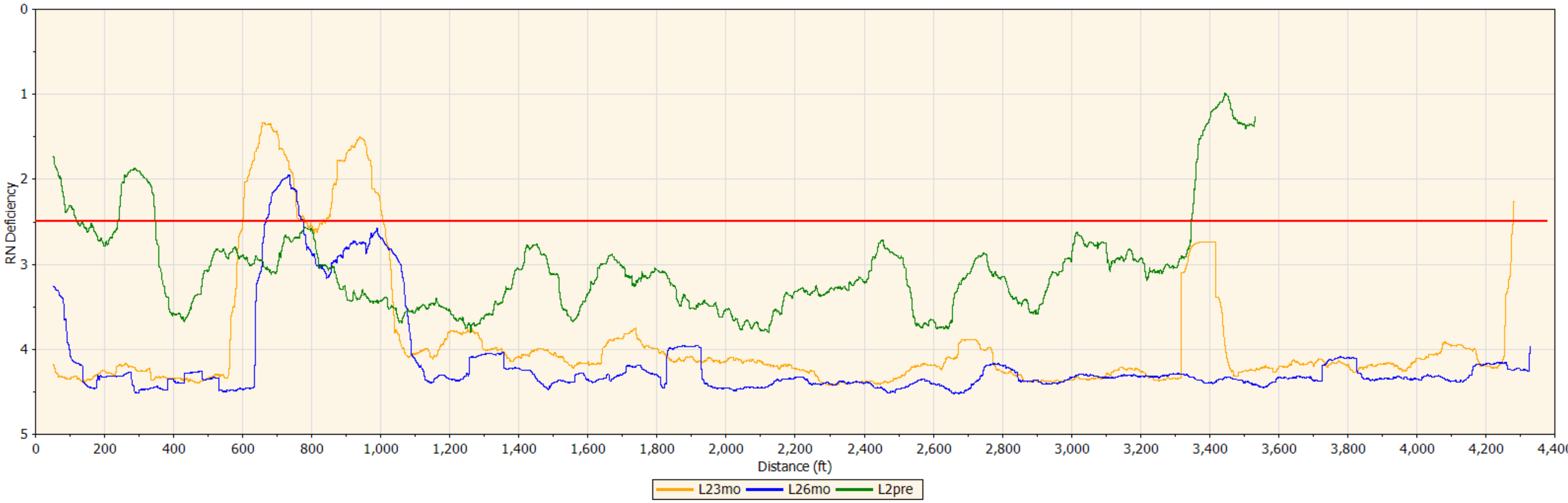
IRI (in/mi)
Distance (ft)
L23mo_Left
L26mo_Left
L2pre_Left

Ride Number (RN)




RN Deficiency
Distance (ft)
R23mo
R26mo
R2pre


RN Deficiency
Distance (ft)
L13mo
L16mo
L1pre


RN Deficiency
Distance (ft)
L23mo
L26mo
L2pre
0
1
2
3
4
5
0
200
400
600
800
1,000
1,200
1,400
1,600
1,800
2,000
2,200
2,400
2,600
2,800
3,000
3,200
3,400
3,600
3,800
4,000
4,200
4,400

Appendix 3

Comparative Histograms of

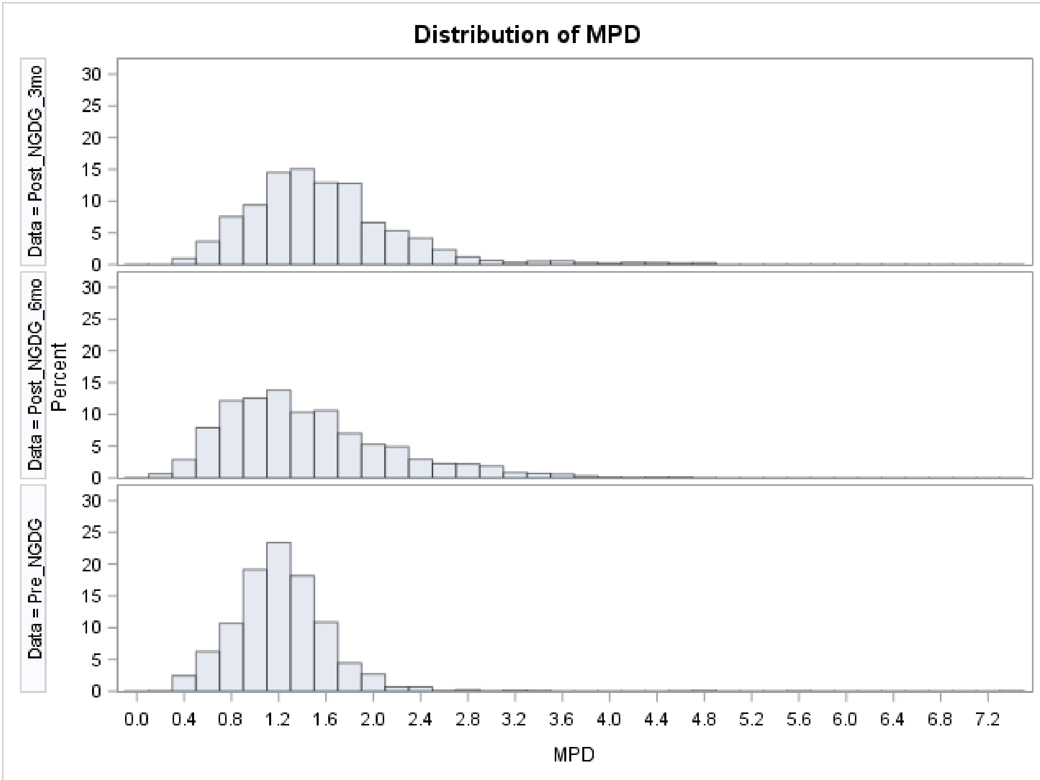
Mean Profile Depth (MPD)

Summary of Plots

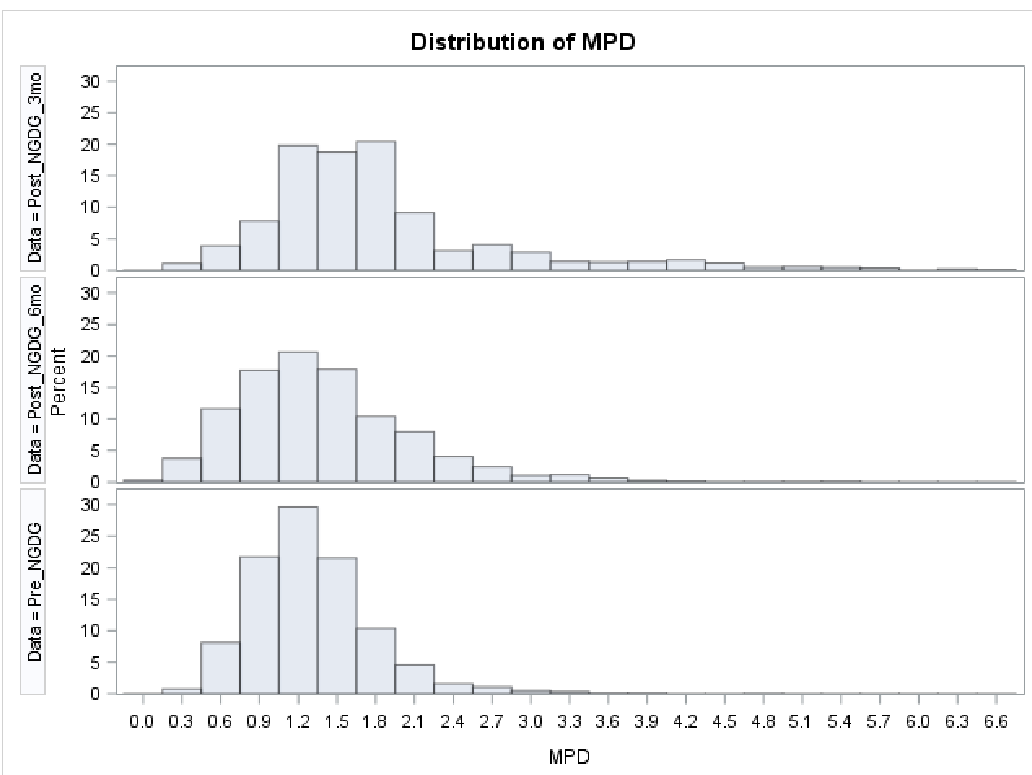
Data	**Lane**			
	R1	R2	L1	L2
Pre-NGDG	✓	✓	✓	✓
Post-NGDG-3 month	✓	✓	✓	✓
Post-NGDG-6 month	✓	✓	✓	✓

Note: in this case, histograms were developed in lieu of scatterplots due to the large sample sizes (smaller data set had 2,331 data points and the largest had 6,363 data points (the post-NGDG-6 months had two replications).

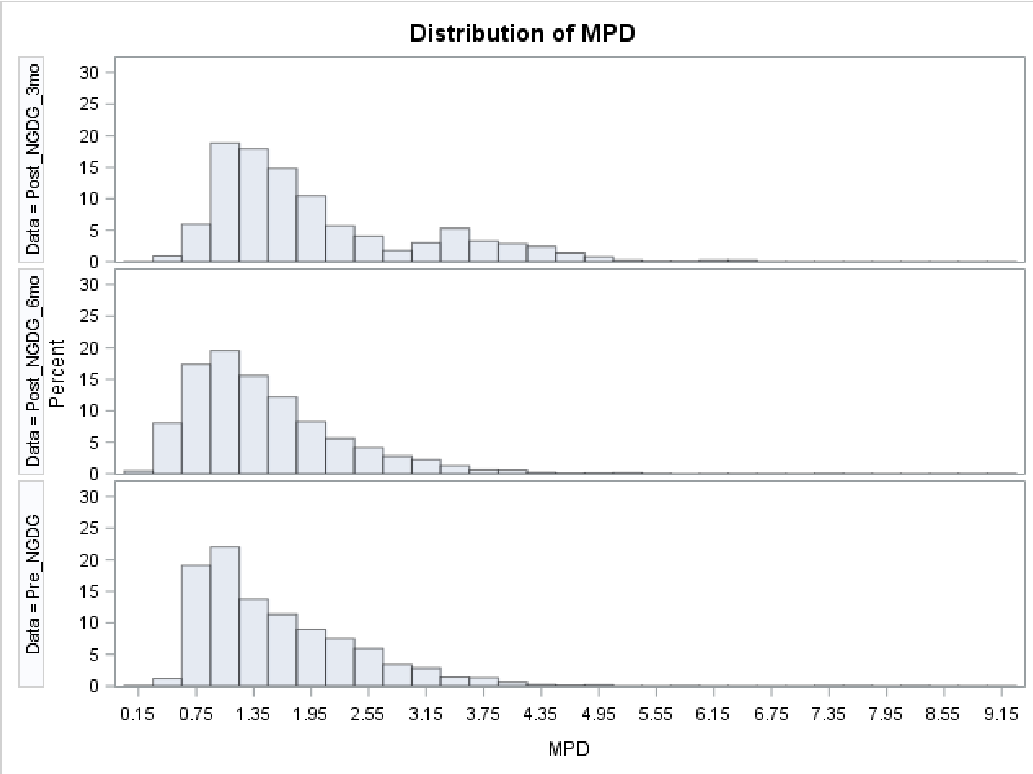
Lane R1



Lane R2



Lane L1



Lane L2