DEPARTMENT OF TRANSPORTATION

Evaluation of Benefits of Drainable Base Systems used by MnDOT

Bernard Izevbekhai, Principal Investigator
Office of Materials and Roads Research
Minnesota Department of Transportation

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16. Abstract (Limit: 250 words)

The Minnesota Department of Transportation (MnDOT) recognizes the importance of subsurface drainage in pavements. Various studies have indicated that adequate drainage of pavement layers enhances performance of pavements in general. MnDOT thus uses various types of subsurface drainage in varying degrees of styles, frequency of use, and minor variation in construction practices in the various transportation districts of the state. The subsurface drainage technologies include Open Graded Aggregate Base (OGAB), Drainable Stable Base (DSB), Permeable Asphalt Stabilized Base (PASB), Geocomposite Joint Drain (GJD) and Class 5Q aggregate. This study examines the various drainable bases in the network and identifies their locations and limits. Using performance data from the pavement management system, the performance, measured via Ride Quality Index (RQI), of test sections with drainable base systems was compared to contiguous sections without the systems so that traffic and environmental factors as well as other variables were held constant. Reliability and logistic analysis were conducted to ascertain if there were performance advantages in the drainable systems. The difference between the systems was found to be advantageous in certain districts, and an operations research survey reflected advantages in the drainable systems where and when they were associated with proficiency in construction practice.

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Evaluation of the Benefits of Drainable Base Systems Used by MnDOT

Final Report

Prepared by:

Bernard Igbafen Izevbekhai P.E.; Ph.D.
Ceren Aydin Ph.D.
Raul Velasquez Ph.D.; P.E.
Minnesota Department of Transportation
Office of Materials and Road Research

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EXECUTIVE SUMMARY

Pavements can be destroyed or damaged severely due to water-related problems if there is not adequate subsurface drainage in the system. These water-related problems cause premature failure of the pavements, thus requiring costly maintenance and user costs. One way to mitigate this damage and implement the drainage is to use drainable base materials under the pavement, which allow water to drain from the base and subbase and let water that is present out of the pavement foundation. This report analyzes the performance of certain drainable bases, specifically Open Graded Aggregate Base (OGAB) and Permeable Asphalt Stabilized Base (PASB), on Minnesota Highways and Interstates using Weibull analysis to determine the characteristic life of the pavements. Comparisons with adjacent non-Open Graded Aggregate Base and non-Permeable Asphalt Stabilized Base sections were made to determine whether the drainable bases increase the service life of these roads or have little effect on the survival of the road. In addition, this report includes a summary of performance evaluation of the geocomposite joint drain (GJD), a cost-effective alternative to subsurface drainage. The GJD is an assemblage of a geonet sandwiched between two non-woven geofabric layers placed under transverse joints that provide drainage capabilities for concrete pavements.

The various levels of familiarity and proficiencies in drainable bases in the transportation districts of Minnesota were hypothesized to have no influence on the level of performance of each of the drainable base types in each district. Moreover, it is hypothesized that pavements maintain good ride quality over a longer period when they are built on drainable bases. In addition, there is a logistic regression between the relative performance of good drainable sections and the contiguous sections that point to an improvement in performance, when presence and absence of drainable bases are indicated as categorical variables one and zero, respectively.

Based on the above hypothetical lemma, the remaining service lives of various drainable base technologies and their respective contiguous non-drainable sections were analyzed. Statewide data of each drainable base type was also compared to its respective contiguous control section using Weibull analysis on all the sections.

Weibull analysis is used to study the success of each subsurface type/section and to study how the characteristics have changed over time. In this report, the performance of Open Graded Aggregate Base, Permeable Asphalt Stabilized Base, and Permeable Concrete Stabilized Base (PCSB) is studied. The study compares the Remaining Service Life (RSL), Preventative Maintenance Window or Threshold Time to Failure (TTTF) and failure pattern (Shape Parameter) of certain populations of drainable base sections to other non-drainable base populations. The RSL followed the institutional definition of how many years until pavement degrades to a Ride Quality Index (RQI) of 2.5. The TTTF indicated how many years following the initial construction are available to improve the life of the pavement through minor preventive maintenance activities when the pavement is still in a relatively pristine condition. The failure pattern shows how the population of a network or group of pavements fails, where a higher value means most roads in the given population are wearing out due to age (the goal of a good design). A lower value means most of the sections in that network in the given population are wearing out early, likely due to construction flaws or characteristics of the drainable base system, which cause the road to fail prematurely. This is referred to as "Burn-in" or "Infantile Failure" and it is undesirable in pavement design. Performing Weibull analysis allowed the identification of the parameters μ, t_0, and β with those respective performance measures the shape/threshold parameter is β, μ is the scale/characteristic life parameter, and t_0 is the location/shift parameter, which is the time to failure and when these parameters are specified. This is the model that best fits the given data and is technically the year at which the data trends seem to begin, hence the term "location parameter."

The Weibull analysis on drainable base sections versus their contiguous non-drainable counterparts did not give a definitive answer as to whether the RSL of the drainable base sections were extended significantly because only about half of the districts had an RSL of the drainable base that was less than the non-drainable sections. This was confirmed with the t-test results.

The evaluation of GJD seemed to show that it is a feasible and cost-effective solution to subsurface drainage.

This result necessitated an operations research of district drainable base construction practices. Although the population sampling size was limited, there was sufficient evidence to associate good performance of certain drainable bases with the experience and proficiency in the engineering of these base types. This calls for a cross-training or drainable base forum where experience, lessons learned, and ideas can be shared to maximize the potential benefits of drainable bases.

Chapter 1: Introduction

1.1 Background

The Minnesota Department of Transportation (MnDOT) has 8 transportation districts. Each district performs its own pavement design prior to review from the Office of Materials and Road Research, a remote branch of the MnDOT Central Office, which has a pavement section that includes a pavement design unit. Within this context, various districts have developed expertise in certain technologies including the design and construction of subsurface drainage. After many years of service, this research investigates the performance of the various subsurface drainage types and locations in the network, particularly reviewing their remaining service life in comparison to contiguous sections with no drainable bases.

Excessive water and moisture in the pavement structure and the grade/subgrade can come from different sources, including capillary action from groundwater, vapor movements, and infiltration of run-off into joints and cracks in the pavement, and other damage to the pavement surface, or it may have seepage from high ground such as side ditches and pavement edges. Problems caused by excess moisture fall into three categories, as described by Eres Consultants (1) in the 2004 Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures Part 3, Chapter 1 and Liang, R (2) in their 2007 Evaluation of Drainable Bases under Asphalt Pavements. These include softening of pavement layers and subgrade as they become saturated and remain saturated for long periods of time, degradation of material quality from moisture interactions, and loss of bond between pavement layers from moisture saturation.

When water sits on the joints and beneath or between pavement layers, traffic loads trigger transient hydrodynamic forces that cause “scouring.” Tire-induced loading causes extreme scouring as shown in **Figure 1.1** when transient application and release of traffic loads result in delayed pavement response. This delay induces instantaneous and transient negative pressures causing cavitation. The hysteretic phenomena are thus the culprits of the scouring observed in concrete pavements without subsurface drains (**Figures 1.1 and 1.2**). On the contrary, subsurface drainage relieves any potential for water accumulation beneath the pavements and forestalls any potential for the destructive hydrodynamic forces. Therefore, it is not unexpected that many authors (3-5) have shown that drainable bases result in less degradation of concrete joints than non-drainable bases.

Other obvious pavement deterioration from poor drainage bases includes trapezoidal wedge material loss at the bottom of the concrete joint and secondary spalling at the top of the joint. In asphalt pavements, poorly drained pavements experience stripping of the asphalt at the bottom of the pavement layer, ultimate loss of support, and accelerated degradation. Stripping has also been observed in composite systems, certain unbonded overlays where the asphaltic interlayer is non-draining or poorly draining, and in whitetoppings where the asphalt layer being the substrate may not withstand the constant presence of water (6). Stripping is the systematic rubblization of an asphalt pavement when it is continuously loaded in a wet to saturated condition. The formation of loose fragments due to stripping causes severe loss of layer moduli and overall pavement load capacity.

Figure 1.1 PCC on non-drainable MnDOT Class 5 gradation base (left) vs Permeable Asphalt Stabilized Base (a drainable base) (right).

Figure 1.2 Differences in scouring and other pavement distresses in pavements using Class 5 gradation base vs PASB (drainable) gradation base.

For this reason, most agencies ensure that edge drains, drainable bases, or a combination of both are in their pavement designs. Many agencies, including MnDOT, perform daylighting of the pavement subsurface where terrain and roadway geometrics are suitable. Daylighting and edge drains are only consequential if the subsurface layer provides sufficient lateral transmissivity or hydraulic conductivity to convey the subsurface water to the edge of the pavement. Izevbekhai and Van Deusen (7) also showed that stability of a drainable base is indispensable to overall pavement performance. Their research showed that an early failure of a test cell at MnROAD in 2012 was traceable to the instability of the drainable base material that resulted in initially gradual but ultimately catastrophic failure of the pavement. In addressing this and other base issues, other authors, including Wilde et al. (8), have postulated algorithms for quantification of the stability of aggregates. According to Wilde et al. (8), a gravimetric gravel to sand ratio is expressed as an index from in which it can be shown that

$$\frac{G}{S} = \frac{p_{25} - p_{4.75}}{P_{4.75} - p_{0.075}} \tag{1}$$

$$= \frac{1 - \left(\frac{4.75}{D_{max}}\right)^n}{\left(\frac{4.75}{D_{max}}\right)^n - \left(\frac{0.075}{D_{max}}\right)^n} \tag{2}$$

$$= \frac{{D_{max}}^n - 4.75^n}{4.75^n - 0.075^n} \tag{3}$$

where $\frac{G}{S}$ is gravel to sand ratio, n is shape factor of the grading curve, p_i is the percentage passing sieve size i and D_{max} is the maximum aggregate size.

Equations 1 to 3 provide a proxy for stability although with drainable bases, other ratios that enhance drainability must be considered. Achieving optimal stability and drainability through design and construction is thus a goal for maximum performance. Although chemical stabilization ensures adequate stability of aggregates, most virgin untreated and well-designed aggregate bases have been proven to be stable and drainable when properly designed and constructed.

This study included an operations research component that was used to ascertain how the various MnDOT transportation districts choose and use drainable bases. The choice of drainable bases was hypothesized to be heuristic and not based on the experience of the materials engineer in designing and constructing that drainable base type. A survey was designed to evaluate that hypothesis.

1.2 Types of Drainable Bases

A permeable base is an open-graded drainage layer with a typical laboratory permeability value of 1,000 feet/day or greater (8). Its primary function is to dissipate water infiltrating the pavement surface by moving it toward the edge of the pavement within a reasonable timeframe. The geometry of the pavement determines the drainage path and the hydraulic gradient. The permeable base layer when properly designed allows a hydraulic channel for water to flow freely. These permeable bases can be treated to ensure higher strength and stiffness. When treated with asphalt, they are referred to as Permeable Asphalt Stabilized Base (PASB). Some drainable bases are chemically treated with lime or fly ash for enhancement of granular equivalency and stiffness. Occasionally, a separator layer is placed underneath the permeable base so that fines from the subgrade or other layers below do not contaminate the permeable base, which could impact the deterioration of the pavement over time. The permeability of this layer needs to be balanced with stability, which is necessary for long-term pavement performance and construction purposes.

Open Graded Aggregate Base (OGAB) is a base that is limited to Portland cement concrete (PCC) pavements because it requires certain stability from the pavement. This is because the OGAB, while having great drainability, can be unstable. Based on MnDOT planning requirements, it must be placed on a dense-graded aggregate base, thereby preventing fine materials from entering the OGAB. OGAB is typically accompanied by edge-drains. Drainable Stable Base (DSB) is similar to OGAB but has more stability and less drainability. It can be used with the addition of a drainage path where dense-graded aggregate base is used. This drainage path can be either edge-drains or can be achieved by daylighting the layer to the in-slope (9). PASB is a highly permeable drainage layer, which provides for a high rate of flow. Asphalt stabilization adds cost to the pavement construction but enhances stability. The drainable bases described here are dependent on the aggregate gradation, given in **Table 1.1.** Another plausible alternative for subsurface drainage presented by Izevbekhai (3) is the Geocomposite Joint Drain (GJD). The GJD is a geonet system sandwiched between two non-woven geofabrics. This system is shown to exhibit high lateral transmissivity while its thinness is advantageous in lieu of granular bases when bridge clearance is an issue.

MnDOT uses OGAB, PASB, DSB, Class 5Q, and GJD as subsurface drainage alternatives (**Table 1.1**). The *MnDOT Grading and Base manual* (10) specifies 85% and 60% for minimum 2 face crushing of OGAB and DSB, respectively, and a Los Angeles Rattler of 40% is the maximum for both. It specifies a d60/d10 requirement of more or equal to 4 for OGAB and a maximum Acid Insoluble Residue of 10% for OGAB and DSB. Further, a 5% threshold is established for maximum spall.

Table 1.1 Gradation tables for MnDOT OGAB, PASB, Class 5Q, and Class 5 Aggregates.

Sieve Size	Base Type - Percentage Passing Sieve Size				
	OGAB	PASB	Class 5Q	DSB	Class 5
1½ in (37.5 mm)	100	100	100	100	-
1 in (25.4 mm)	95-100	95-100	-	-	100
¾ in (19 mm)	65-95	85-95	65-95	75-100	90-100
3/8 in (9.5 mm)	30-65	30-60	45-85	45-75	50-90
No. 4 (4.75 mm)	10-35	10-30	35-70	30-60	35-80
No. 10 (2.00 mm)	3-20	0-10	15-45	10-35	20-65
No. 40 (400 µm)	0-8	0-5	10-30	5-20	10-35
No. 200 (75 µm)	0-3.5	0-3	5-25	0-5 For class B or C; 0-6.5 for Class A	3-10

Chapter 2: Methods

2.1 Data

Most of the performance data were obtained from the MnDOT Pavement Management Network. MnDOT stores this data in a retrieval system that is dynamically segmented. Every year the Pavement Management Van runs the US Highways, the Interstate Highways, and the Minnesota Highways in the state (i.e., National Highway System), so the network is constantly being updated as new data is added to it.

MnDOT Pavement Management Data of the National Highway System (NHS) is of type 2 censoring. Unlike type 1 censoring, conditions are noted continuously or periodically until a critical value or failure condition is reached. In that case, the time to failure is part of the information garnered from the research type. In type 1 censoring, the concern is the number of times failure would occur in the system in each period. This is also derived from the available data but trends such as time to reach a threshold are more important in this research than the actual failures occurring (11). Additionally, "failure" is a conceptual term being used here because the trigger values used are not necessarily a failure but a threshold. It is noted that a fix or intervention type has reached terminal serviceability when the Ride Quality Index (RQI) is 2.5. Surface Rating (SR) is a measure of the surface defects of a section. The RQI and SR are measured by pavement management vans. RQI is determined by the rated comfort level of human riders chosen from various works of life, a wide age range and gender disparity as well as educational level. This has been performed every 10 years approximately although it would have been conducted more frequently but for the rigor involved in the use of human subjects for this type of research. The range of RQI is from 0 to 5 (very rough to very smooth). Subsequent measurements are conducted using the pavement management van that measures International Roughness Index (IRI).

IRI is the Average Rectified Value (ARV) of the Slope Power Spectrum Density. Based on the Quarter Car Dynamics, it assembles the effect of vertical acceleration of the quarter car over a stretch of road and provides what seems to be a summation of vertical displacements per unit horizontal displacement. This is not an average slope of the section otherwise the complex gain factors of the quarter car in emulating the ride comfort of the human system would be of no consequence (12). Correlation equations between IRI and RQI for concrete and for asphalt are developed and reviewed over time to ensure that there is no ignored effect of change in perception or expectations of the riding public as to the scale of appreciation of good, fair, bad, and poor. The equations to convert IRI to RQI are as follows:

For bituminous pavements:

$$\boldsymbol{RQI = 5.697 - 2.104} * \sqrt{\boldsymbol{IRI}} \text{ for IRI in m/km} \tag{4}$$

$$\boldsymbol{RQI = 5.697 - 0.264} * \sqrt{\boldsymbol{IRI}} \text{ for IRI in in/mile} \tag{5}$$

For concrete pavements:

$$\boldsymbol{RQI} = \mathbf{6.634} - \mathbf{2.183} * \sqrt{\boldsymbol{IRI}} \text{ for IRI in m/km} \tag{6}$$

$$\boldsymbol{RQI} = \mathbf{6.634} - \mathbf{0.353} * \sqrt{\boldsymbol{IRI}} \text{ for IRI in in/mile} \tag{7}$$

Performance curves were created by plotting RQI and SR against years since construction. Projects and repairs done on the section were marked in a separate graph below the original one. The remaining service life of each section were extrapolated from these performance curves.

2.2 Analytic Methods

2.2.1 Reliability (Weibull) Analysis

To study the performance of grouped sections, it is necessary to examine the individual component performances over time. In this report, data was classified according to the drainable bases that are studied (i.e., OGAB, PASB, Class 5Q, and PCSB) to determine their success and performance. To do this, the Remaining Service Life (RSL), preventative maintenance window, and failure pattern of certain populations of section parameters with certain drainable base treatments were compared to contiguous sections without treatment. The section RSL is how many years before pavement failure (i.e., pavement with an RQI of less than or equal to 2.5), and preventative maintenance window or threshold time to failure is how many years after the initial construction in which improvements to the life of the pavement through small changes such as chip seal can be done. Failure pattern shows how the population of sections fails, where a higher value means most sections in the given population are wearing out due to age and a lower value means most sections in the given population are wearing out early, likely due to construction flaws which cause the road to fail prematurely. Any failure pattern value less than 1 means premature failure, equal to 1 means random failure, and greater than 1 is due to pavement aging with specific categories ranging from 1-2, 2-3, 3-4, and 4+. Weibull analysis gives estimates for these parameters.

Weibull distribution is determined by parameters β, μ, and t_0, where the shape/threshold parameter is β, μ is the scale/characteristic life parameter, and t_0 is the location/shift parameter which is the time to failure. The data analyses determine the parameters that are most appropriate for the model with the following probability and cumulative density functions:

$$\textbf{For the Probability Density function, } \boldsymbol{f(t)} = \frac{\boldsymbol{\beta}}{\boldsymbol{\mu}}\left(\frac{\boldsymbol{t}-\boldsymbol{t_0}}{\boldsymbol{\mu}}\right)^{\boldsymbol{\beta}-\mathbf{1}} \boldsymbol{e}^{-\left(\frac{\boldsymbol{t}-\boldsymbol{t_0}}{\boldsymbol{\mu}}\right)^{\boldsymbol{\beta}}} \text{ for } \boldsymbol{t} \geq \mathbf{0} \tag{8}$$

$$\textbf{For the Cumulative Density Function } \boldsymbol{F(t)} = \mathbf{1} - \boldsymbol{e}^{-\left(\frac{\boldsymbol{t}-\boldsymbol{t_0}}{\boldsymbol{\mu}}\right)^{\boldsymbol{\beta}}} \tag{9}$$

where f(t) is the probability density function (PDF), and F(t) is the cumulative density function (CDF). To find these parameters and to solve for the Weibull parameters, the Weibull plot, which is a plot of the empirical cumulative distribution function of data on special axes in a type of Quantile vs Quantile plot can be visually assessed or in this case solved analytically. Linearizing the Weibull cumulative density function, $ln(ln(\hat{F}(t)-1))$ *vs* $ln(t)$, where $\hat{F}(t)$ is calculated from the data, the cumulative density function F(t) is expressed as

$$\boldsymbol{ln(1-F(t)) = ln(\, e^{-\left(\frac{t-t_0}{\mu}\right)^{\beta}}\,)} \tag{10}$$

$$\boldsymbol{ln(-ln(1-F(t))) = \beta ln(t-t_0) - \beta ln(\mu)} \tag{11}$$

(12)

Letting $y = \ln\left(\ln\left(\frac{1}{1-F(t)}\right)\right)$ and $x = \ln(t - t_0)$, Eq 12 can be rewritten as:

$$y \;=\; \beta x - \beta ln(\mu) \tag{13}$$

which is linear with a slope of $m \;=\; \beta$ and intercept of $b = -\beta ln(\mu)$, i.e., it is in the standard form of a straight line. Thus, if the data comes from a Weibull distribution, then a straight line is expected on a Weibull plot, meaning the parameters are found via a best-fit if linear model described by Eq 13.

2.2.2 Logistic Regression

A reliable method of representing the probability that a drainable section in the network will perform better than a contiguous non-drainable section is logistic regression. The method of logistic regression estimates the parameters of a binary logistic model. The values are “0” or “1,” where “0” indicates that the drainable base section has a lower RSL than the corresponding contiguous non-drainable increasing (or decreasing) and “1” meaning the drainable base section has a larger RSL. The logistic regression gives estimates for φ_0 and φ_1 in **Equations 14-16** which are used to make a continuous function as in the following derivation:

$$\text{Logit(p)} = \log \boldsymbol{\frac{p}{1-p}} = \boldsymbol{\varphi_0 + \varphi_1 x} \tag{14}$$

$$\boldsymbol{\frac{p}{1-p} = e^{\varphi_0+\varphi_1 x}} \tag{15}$$

$$\boldsymbol{p = \frac{e^{\varphi_0+\varphi_1 x}}{1+ e^{\varphi_0+\varphi_1 x}\varphi} = \frac{1}{e^{-(\varphi_0+\varphi_1 x)}+1}} \tag{16}$$

The “p” in the above equation is the probability of a drainable base exceeding the RSL of its coterminous non-drainable base section.

2.2.3 Hypothesis Testing, Sample Size, and Model Power

A significance test (or t-test) can determine whether two population means are statistically equal. The null hypothesis for a two-tailed test is $H_0\colon \mu_1 - \mu_2 = D_0$ for given D_0. The alternative hypothesis is $H_a\colon \mu_1 - \mu_2 \neq D_0$. The p-value can be calculated as in (14) and if it is smaller than the significance level, H_0 is rejected, otherwise, fail to reject H_0.

To justify t-test conclusions, there must be a large enough sample size. Given that data is drawn as independent samples from two populations sections (drainable base and non-drainable base) which are assumed to be normal with the same variance, the required sample size for given values of significance level (α), power (1- β), standard deviation of the population (σ) and value needed to detect a change (δ) can be calculated using the following equation:

$$\boldsymbol{S} = \mathbf{2} * \left(\boldsymbol{Z}_{\mathbf{1}-\frac{\alpha}{2}} + \boldsymbol{Z}_{\mathbf{1}-\frac{\beta}{2}}\right)^{\mathbf{2}} * \left(\frac{\sigma}{\delta}\right)^{\mathbf{2}} \tag{17}$$

where $Z_{1-\frac{\alpha}{2}}, Z_{1-\frac{\beta}{2}}$ are the critical values dependent on the values of α, β in $\mathcal{N}$(0,1), the normal distribution with mean of 0 and variance of 1.

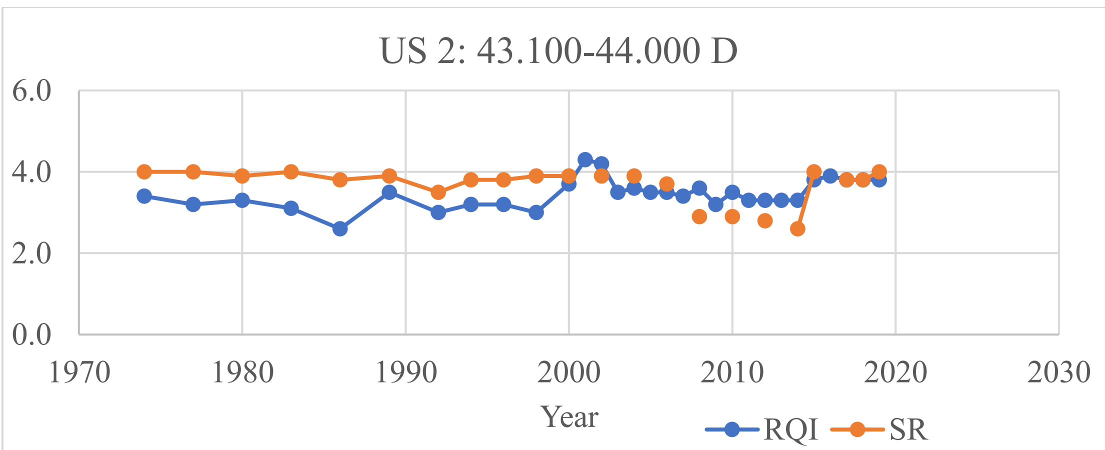
2.3 Strategy

First, drainable base sections were generated from MnDOT’s Pavement Management system and sent to district engineers to get confirmation of these sections as well as changes to the list. The data was then compiled from historical RQI data (first as design records and then as mile records) and plotted as performance curves. The performance curves were extrapolated to an RQI of 2.5 to obtain the RSL estimate. Similarly, the non-drainable contiguous section was compiled by taking the sections on either side of the confirmed drainable base sections and the RSL estimate was found as above. Then, all the data was separated by district and Weibull analysis was done on the drainable base sections for all the roads in each district and for the non-drainable contiguous sections to obtain the threshold time to failure, characteristic life, and mode of failure parameter values. A significance test was done between the average RSL of drainable and the contiguous section. These parameters and the p-value from the significance test are used to distinguish between the RSL of drainable and non-drainable base within each district. Calculating the minimal sample size to give a certain amount of model power was used to justify the results of the significance tests and emphasized the lack of data that was obtained. The logistic regression was done by comparing the extrapolated RSL of a drainable base section to either its increasing contiguous section or the decreasing contiguous section (two logistic regressions were done for each dataset), and if the RSL of the drainable base was larger, it is a one, otherwise a zero.

Chapter 3: Evaluation of Mile Records (m-records)

3.1 Results

Both mile records and design records were used to create datasets which were analyzed separately. The mile records (M-records) represent data arranged according to milepost irrespective of design while the design records (D-records) represent data arranged according to design irrespective of milepost markers. A consequence of this is the ability to observe the behavior of design clusters in D-records and the behavior of mile post fragmentation to allow variability within designs. Figures **3.1a and b** show the example layout of the M-record and D-records arrangement of data as received from the Pavement Management Unit.



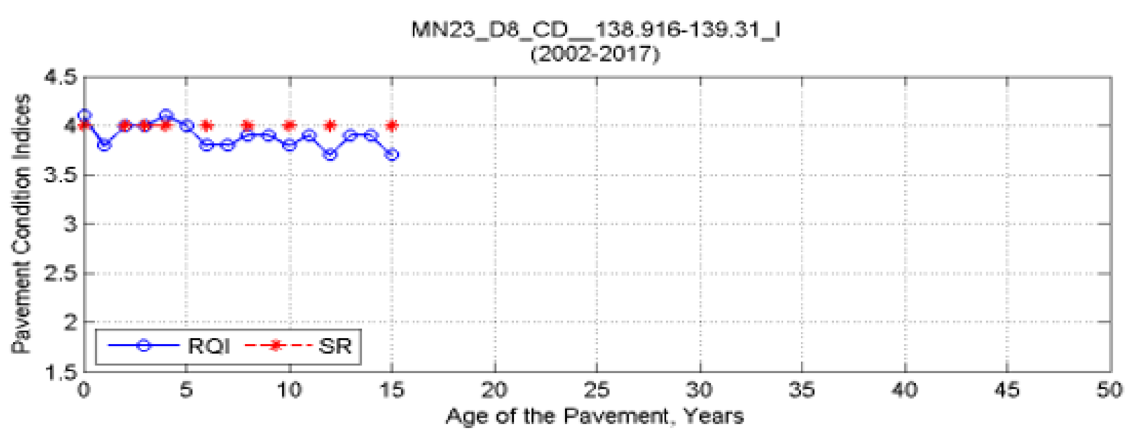


Figure 3.1 Example layouts of the M-record and D-records.

Table 3.1 shows an example of a record received from the Pavement Management Unit.

Table 3.1 Example of a record

Route		Dir.	Begin RP	End RP	Year	RQI	SR
Type	Num.						
US	2	D	43.122	44.023	1974	3.4	4.0
US	2	D	43.122	44.023	1977	3.2	4.0
US	2	D	43.122	44.023	1980	3.3	3.9
US	2	D	43.122	44.023	1983	3.1	4.0
US	2	D	43.122	44.023	1986	2.6	3.8
US	2	D	43.122	44.023	1989	3.5	3.9
US	2	D	43.122	44.023	1992	3.0	3.5
US	2	D	43.122	44.023	1994	3.2	3.8
US	2	D	43.122	44.023	1996	3.2	3.8
US	2	D	43.122	44.023	1998	3.0	3.9
US	2	D	43.122	44.023	2000	3.7	3.9
US	2	D	43.122	44.023	2001	4.3	3.9
US	2	D	43.122	44.023	2002	4.2	3.9

Table 3.2 illustrates how the data was sorted into categories. First, all the sections identified as using a drainable base during construction were split by district and then sorted into the specific type of drainable base or their respective contiguous sections. It then shows the Weibull characteristic life.

Table 3.2 Weibull parameters from Reliability Analysis

Type	District	1	2	3	4	5	6	7	8	Districts 1,2,3,4,5,7,8
OGAB	Threshold time to failure (Years)						0	0	2	1
	Characteristic Life (Years)	0				17	13.78	12.6	12.7	12.5
	Mode of failure						2.242	3.2	2.7	2.8
Non-OGAB Contiguous Section	Threshold time to failure (Years)	5					2	1	0	0
	Characteristic Life (Years)	12.0				11	10.2	13.1	12.0	12.5
	Mode of failure	3.7					1.4	3.7	2.8	3.3
PASB	Threshold time to failure (Years)	8	8		3			1	0	1
	Characteristic Life (Years)	13.2	17.7	17.0	15.3	10.5		10.0	12	15.3
	Mode of failure	4.5	14.0		3.6			2.0	3.83	2.8
Non-PASB Contiguous Section	Threshold time to failure	3	6					0	3	4
	Characteristic Life	11.3	14.5	19	12	16		14.6	10.2	14.0
	Mode of failure	1.5	6.9					2.6	1.1	1.5
Class 5Q	Threshold time to failure	5	0	0	0	0	2	0		2
	Characteristic Life	12.6	16.5	11.0	13.7	10.5	9.5	13.2		11.6
	Mode of failure	3.4	3.0	2.5	2.7	2.3	1.3	2.6		1.5
Non-Class 5Q Contiguous Section	Threshold time to failure (Years)	3	0	4	0	3	2	0		1
	Characteristic Life (Years)	14.1	16.5	17.7	13.7	9.2	16.7	11.3		14.8
	Mode of failure	3.5	3.3	3.6	2.9	1.2	3.3	1.9		2.2

In **Table 3.3**, while the p-values are generally low (< 0.2), only 17 of the 36 significance tests showed p-values < 0.05 (i.e., selected significance level), indicating that about half of the drainable base sections had RSL significantly different than the RSL of the contiguous non-drainable sections.

Table 3.3 Drainable base vs. non-drainable base RSL t-test results

	p-value (alpha=0.05) from t-test comparing Drainable Section to Contiguous Non-Drainable Section in District								
District	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**Statewide**
OGAB	0.0009				0.1667	0.0004	0.1909	0.1299	0.4438
PASB	0.1555	0.0667	0.0011	0.0449	0.4717		0.0020	0.0012	0.0243
Non-Overlay OGAB	0.0009				0.1667	0.0004	0.1210	0.1299	0.0107
Non-Overlay PASB	0.1555	0.0181	0.1465	0.2190	0.0680		0.0020	0.0496	0.0710
Class 5Q	0.1910	0.4950	6.6E-05	0.0991	0.2305	0.0002	0.1287		0.0002

3.2 Discussion

3.2.1 Reliability Parameters

From **Table 3.2**, non-OGAB contiguous segments have characteristic life values ranging from 10.2 to 13.1 years with District 6 having the lowest and District 7 having the highest. District 6 has the mode of failure parameter value of 1.4 and the combined districts have a value of 3.3. For OGAB segments, characteristic life values vary between 0 (District 1) to 13.78 years (District 6). The threshold time to failure goes from 5 to 0 for non-OGAB and 0 2 to 0 for OGAB. Thus, half of the non-OGAB segments adjacent to the OGAB sections have smaller characteristic lives compared to the OGAB segments, but the difference is not large and the other half have larger characteristic lives compared to the OGAB segments, which indicates that there is no statistically significant difference between the performance of non-OGAB and OGAB sections (i.e., both segments seem to have about the same characteristic life as indicated by t-test results in **Table 3.3)**. For most of the t-tests, a larger sample size would provide more reliable results, as some of the datasets had sample sizes far below what the minimal size should be, calculated using **Equation 17.**

For PASB segments, the characteristic life parameter varies from 10.0 (District 7) to 17.74 (District 2). There is no PASB data in District 6, leaving nothing to compare with the combined PASB data. The only comparison that can be made is to the non-PASB contiguous segments. Districts 1, 2, 4, and 8 all had PASB characteristic life values which were larger than the non-PASB contiguous segments, but Districts 3, 5, and 7 have PASB characteristic life less than the non-PASB contiguous segments.

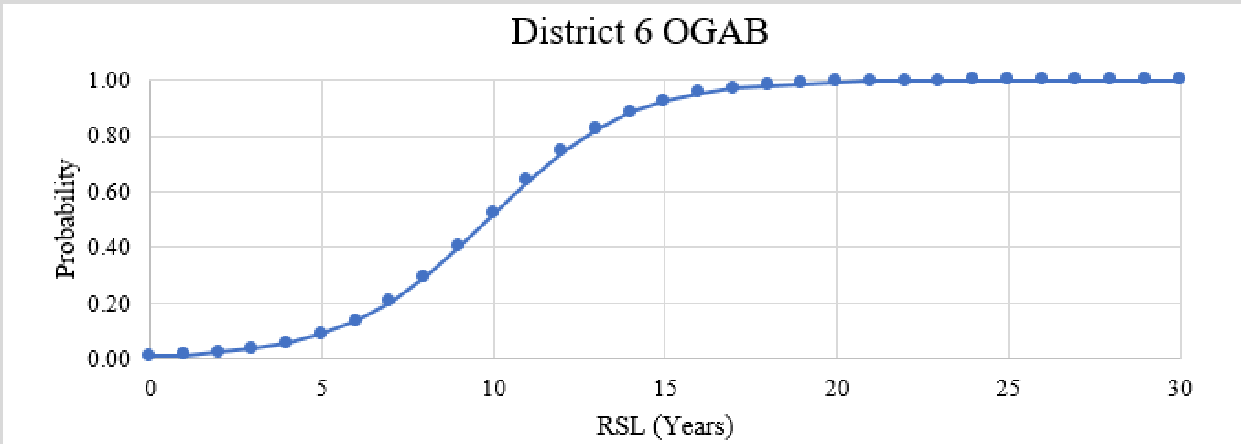
For non-PASB contiguous segments, the calculated characteristic life values ranged from 10.2 to 19. As mentioned above, Districts 1, 4, and 8 have PASB segments with larger characteristic life parameter values, but Districts 3, 5, and 7 have the non-PASB contiguous segments with larger characteristic life parameter values. For Districts 3 and 5 this can most likely be attributed to a lack of data (there were only 2 sections of PASB, and thus Weibull analysis could not be applied), but District 7 had 36 sections that were considered. The mode of failure parameter for the combined data is 1.5, meaning that the failure pattern is >1. Consequently, there is no premature or random failure, and many of the sections made up of non-PASB contiguous segments are wearing out due to age instead of construction flaws.

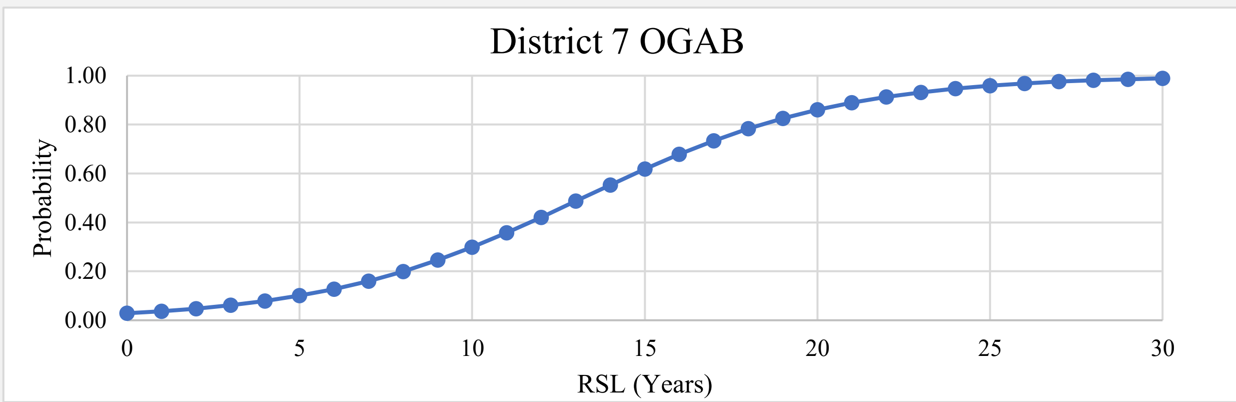
The sections which contained Class 5Q aggregate had threshold time to failure parameters ranging from 0 to 5, and 0 to 4 for the non-Class 5Q contiguous sections. The characteristic life parameters ranged from 9.5 to 16.5 for the Class 5Q sections and 9.2 to 17.7 for the contiguous sections, and mode of failure was greater than 1 for all districts, indicating a tendency to end of life failure. As with the PASB and OGAB sections, the RSL of the drainable base was larger than the contiguous counterpart in Districts 2,4, and 5, but less than in Districts 1,3,6, and 7. The t-test data shows that these differences are not significant in many of the cases, but in the case where all of the districts data is combined together, the sample size is large enough and it can be concluded that the difference is statistically significant.

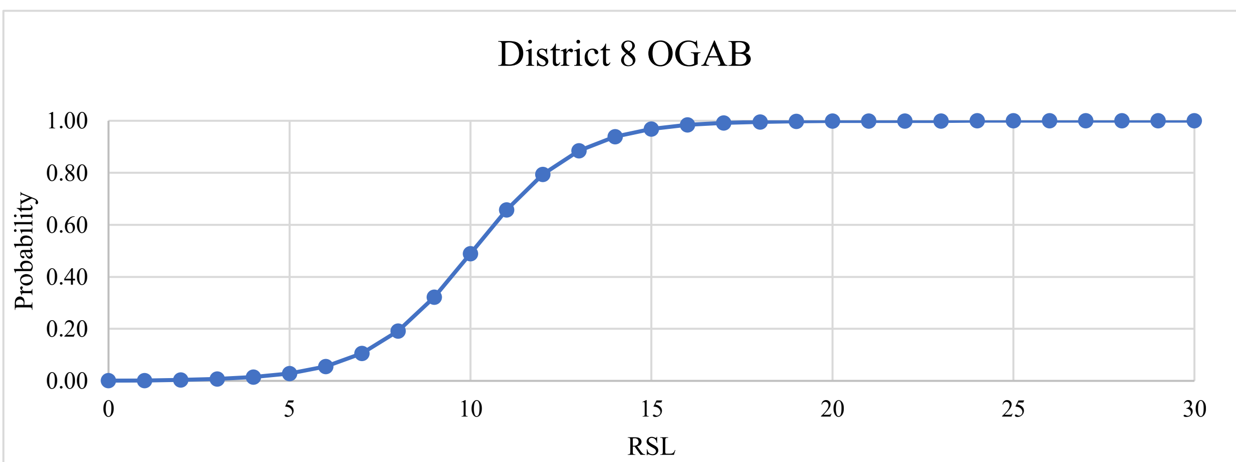
Overall, the estimated parameter values for all the data and the overlay-excluded data seems to be the same general trend. There does not seem to be any distinct connection between the remaining service/characteristic life parameter value for PASB, Class 5Q, or OGAB and that of the non-drainable base contiguous sections. For each data set, about half of the districts had Class 5Q/PASB/OGAB characteristic life parameter larger than the Non-OGAB/PASB/Class 5Q characteristic life parameter, but the other half had the same or the drainable base parameter smaller than the non-drainable base parameter. Thus, while the benefits of the drainable bases include better flow of water through the pavement and less clogging, the overall remaining service life does not seem to improve by a statistically significant amount. It is therefore questionable if RSL is a sufficient measure of the effect of drainable bases.

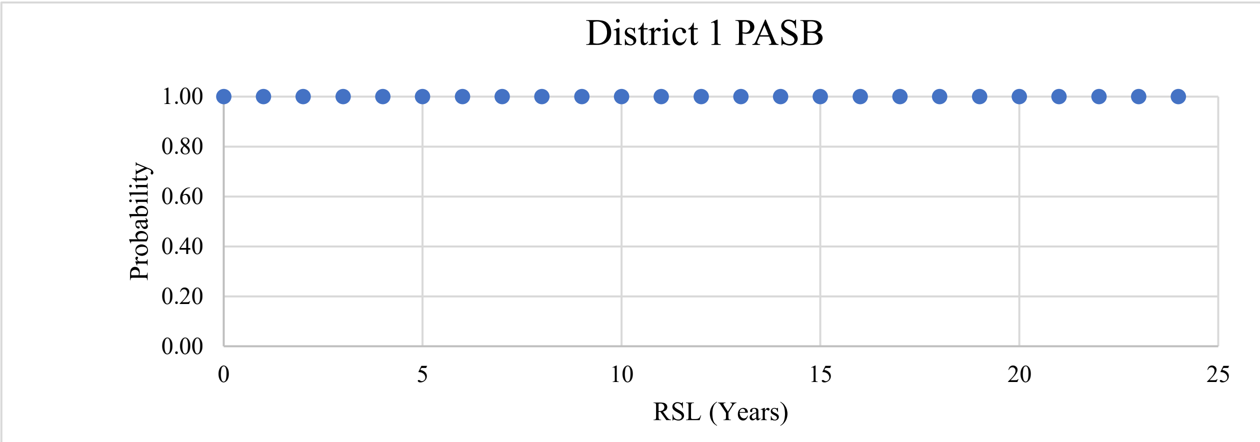
3.2.2 Logistic Regression Plots

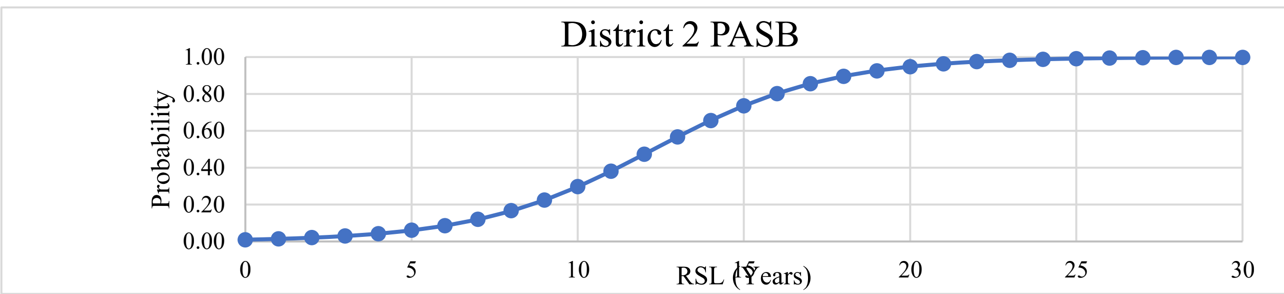
Using the continuous equation derived in the analytic methods section about logistic regression, the plots are created using the parameters in the equation. The plots give the likelihood that the remaining service life of the drainable base sections exceeds that of the non-drainable base sections. It is evident that if factors forced the OGAB RSL to 15 years in District 6, it would have a 50% likelihood of still being higher than the non-drainable coterminous section. However, that same factor that would result in a 50% probability would only occur if in District 7 the RSL is more than 14 years (**Figures 3.2-3.3**). Most of the data has that 50^{th} percentile in the range of 7-15 years, indicating that all districts have similar performances and the pivot of 50% probability of the drainable base having the larger RSL compared to the non-drainable contiguous sections is RSL > 10 years.


District 6 OGAB
Probability
1.00
0.80
0.60
0.40
0.20
0.00
0
5
10
15
20
25
30
RSL (Years)


District 7 OGAB
Probability
1.00
0.80
0.60
0.40
0.20
0.00
0
5
10
15
20
25
30
RSL (Years)


District 8 OGAB
Probability
1.00
0.80
0.60
0.40
0.20
0.00
0
5
10
15
20
25
30
RSL





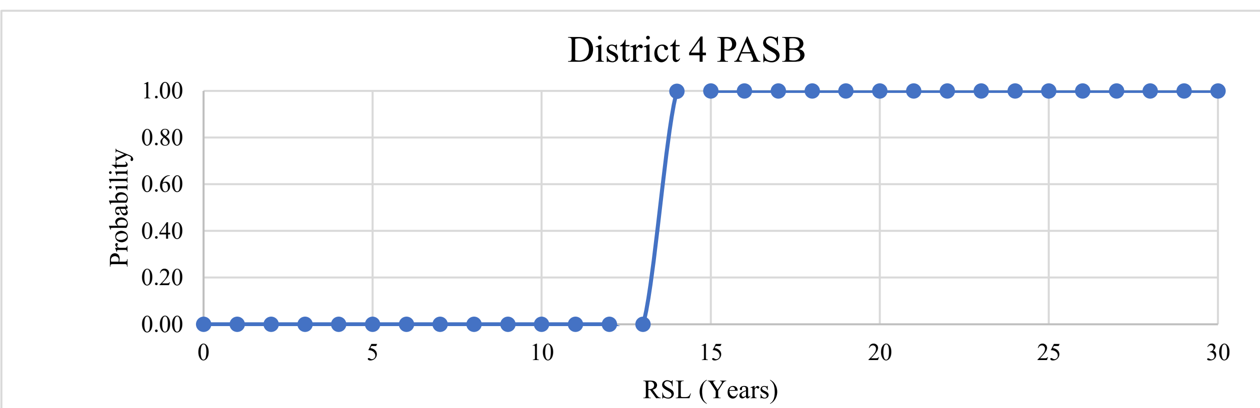


Figure 3.2 Decreasing OGAB Logistic Regression Plots

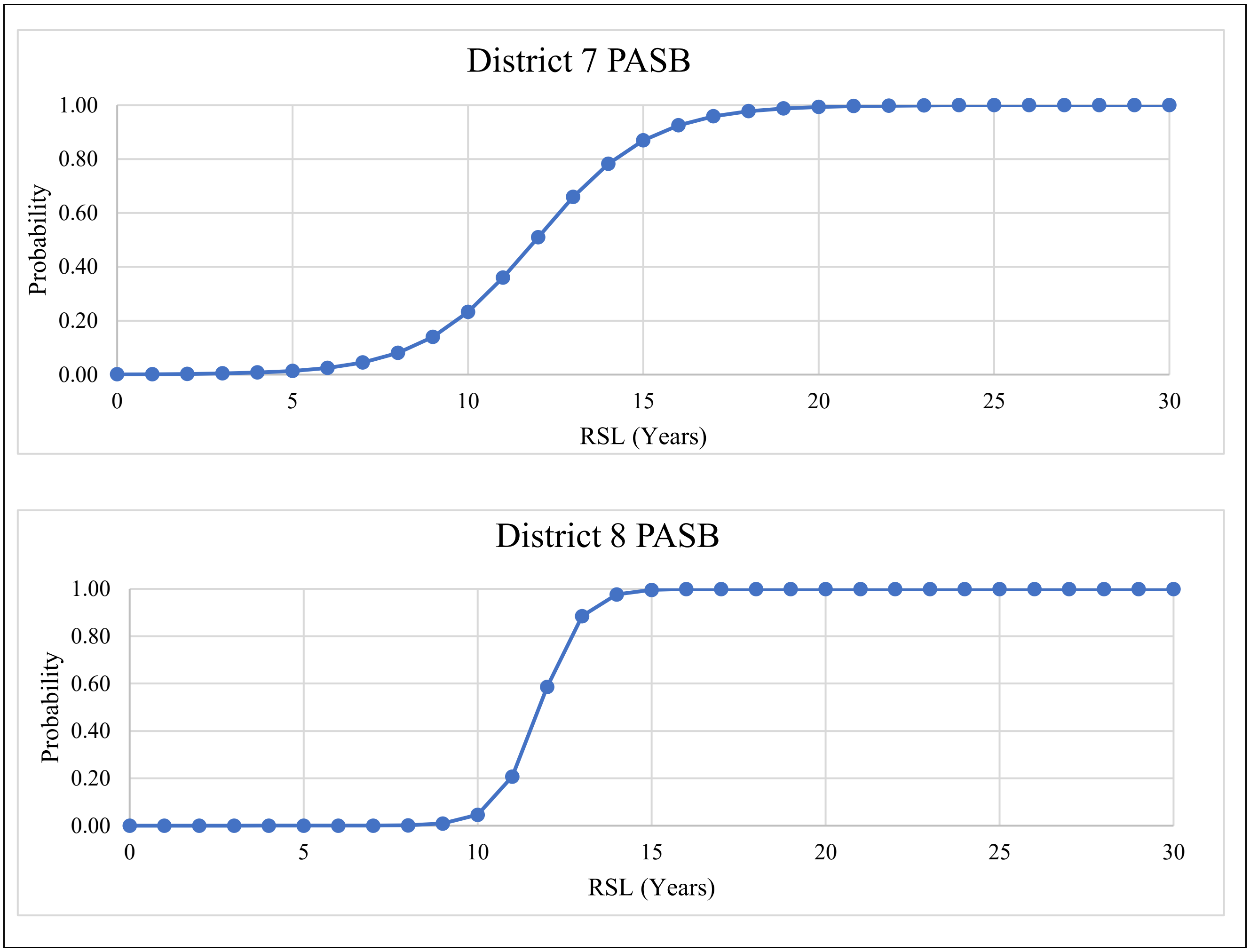


Figure 3.3 PASB Logistic Regression Plots

3.2.3 Significance Tests

The significance test results in **Table 3.3** confirm what was found from the Weibull analysis. The individual districts have varying results, where only about half of the districts have significant differences between drainable base and non-drainable base remaining service lives. If all data for PASB, OGAB, and other drainable bases are compiled based on their individual base type, there is a significant difference between the remaining service lives, indicating that the drainable base does improve the service life of the pavement, as expected from previous studies.

The calculations for minimum number of data points in **Table 3.4** indicate that for the OGAB sections, $n \geq 18$ and there are 451 OGAB sections and 128 non-OGAB base sections used in the t-test. Similarly, for PASB sections it was calculated that $n \geq 46$ and there are 167 PASB sections and 61 non-PASB sections which are used in the t-tests. The parameters used were $\alpha = 0.05, \beta = 0.2, \delta = 3$ years which means that this is a 95% confidence limit with 80% certainty of the correlation. Different values of α, β change the smallest sample size required, but with these parameter values there is enough data being analyzed to comprise a usable sample space. The data for t-tests in the individual districts was often not large enough, but the results are still useful.

Table 3.4 Minimal expected sample vs actual sample size for two-sample t-test calculations.

	Actual Sample size vs Minimum Sample Size in District								
District	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**Combined**
OGAB Expected Sample Size	7				11	34	16	21	18
OGAB Actual Sample Size	2				1	171	185	92	451
PASB Expected Sample Size	38	47	3	11	11		37	67	46
PASB Actual Sample Size	4	93	2	14	2		36	12	167
Class 5Q Expected Sample Size	15	16	22	12	48	29	39		35
Class 5Q Actual Sample Size	8	14	33	12	12	74	52		205

Chapter 4: Evaluation of Design Records (D-Records)

4.1 Data

First, data was obtained from each district and performance curves plotting RQI against time from construction (in years) were created for each segment identified as using OGAB or PASB. The only data considered is from projects not labeled as overlay.

District 1 has two sections on US 169 identified and confirmed to use OGAB and four sections for PASB. District 2 has no sections confirmed to use OGAB, and 10 sections confirmed to use PASB on highways US 2, US 71, US 75, and MN 200. District 3 has no sections confirmed to use OGAB, and two sections on MN 23 confirmed to use PASB. District 4 has one section on IS 94 confirmed to use OGAB, and no sections which use PASB. District 5 has one section on IS 35 confirmed to use OGAB, and two sections on IS 494 which use PASB. District 6 has 89 sections on US 14, US 52, US 63, and MN 13 which use OGAB, and none which use PASB. District 7 has 41 sections on MN 23, MN 60, and US 14 which use OGAB, as well as 12 on US 14, US 169, and MN 13 using PASB. District 8 has 31 sections on US 12, US 212, US 71, MN 15, MN 19, MN 22, MN 23, and MN 7 which are confirmed to use OGAB, and four sections on US 71 and MN 15 which use PASB.

RSL for each segment of data was obtained via extrapolation of the graph of RQI vs year until RQI is 2.5. Then, Weibull analysis was done for each district and the sections that had been identified as OGAB or PASB, as well as the adjacent segments which are not OGAB or PASB. These are used as a comparison to the OGAB and PASB sections. Weibull analysis can only be done when there are at least three distinct data points, so some of the data points only have the Characteristic Life parameter (μ) (not the other two parameters) obtained by calculating the 63.2 percentile of the CDF characterized by the mean and standard deviation of the RSL of the segments. The results of these analyses on all OGAB data as well as only the non-overlay projects are shown in Tables 4.1 and 4.2. In the tables below, if the cell is blank, there was either not enough data to perform Weibull analysis on it or there was no data at all, as some sections had very few, if any, of the PASB or OGAB sections.

4.2 Results

Table 4.1 illustrates how the data was sorted into categories. First, all the sections identified as using a drainable base during construction were split by district and then sorted into the specific type of drainable base or their respective contiguous sections. The projects were sorted by overlay vs non-overlay to make sure the data illustrates the impact of the drainable base on the remaining service life and not the impact of an overlay.

Table 4.1 Weibull parameters from D-record data.

Type	**District**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**Districts 1.2.3.4.5.7.8**
OGAB	***Threshold time to failure***						3	5	4	4
	Characteristic Life	0			24	18	11.56	17.67	18.20	16.06
	Mode of failure						1.42	5.14	1.87	1.98
Non-OGAB Contiguous Section	***Threshold time to failure***	8					2	0	3	3
	Characteristic Life	18.01			16.65	11.95	10.25	16.60	15.44	16.11
	Mode of failure	5.66					1.42	2.53	1.09	1.33
PASB	***Threshold time to failure***		8					3	0	3
	Characteristic Life	19.47	20.21	24.22		13.74		15.53	15.46	18.45
	Mode of failure		4.18					1.44	1.99	2.14
Non-PASB Contiguous Section	***Threshold time to failure***	5	4			6		4	8	3
	Characteristic Life	12.89	13.63	23.48		11.70		16.54	16.31	14.91
	Mode of failure	1.19	1.04			1.48		1.28	83.57	1.21

Note, with the data in terms of D-records, there was not enough data to do a logistic regression analysis.

4.3 Discussion

The results from **Table 4.1** show that, as in the analysis of the mile records, there is an almost equal number of districts that have RSL of the drainable base higher than that of the non-drainable counterparts.

For the OGAB data, Districts 4,5,6,7, and 8 all have the RSL of the OGAB sections larger than the RSL of the non-drainable contiguous sections, and District 1 is not very trustworthy since there were only 2 sections of data analyzed, both with an RSL of 0. The combined districts have a much closer ratio of the RSL of OGAB to non-OGAB contiguous sections, both around 16.

For PASB projects, Districts 1,2,3, and 4 all have the RSL of PASB larger than the contiguous sections, but Districts 7 and 8 have the RSL of the drainable base lower. The combined sections seem to show a larger difference between the RSLs, with the drainable base having much larger RSL at 18.454 compared to 14.907.

In the districts which have projects using OGAB, all districts except the combined districts have the drainable base sections with higher RSL compared to their non-drainable contiguous sections. The combined districts dataset most likely gives the best overall estimate of how OGAB behaves as a drainable base in terms of its RSL. It averages near 16, similar to the non-OGAB contiguous sections. This indicates that although the data seems to show that OGAB extends the service life of the pavement, this conclusion should not be drawn based on the shortage of data in most situations, and the combined districts gives a better estimate of how OGAB contributes to the RSL of a pavement.

Chapter 5: Geocomposite Joint Drain

5.1 Background

Sustainability of concrete pavement was a significant objective of a 2013 MnROAD cell construction initiative. Originally, the concept included crushing the concrete pavement from an existing concrete cell for coarse aggregates in the new cell and researchers sought to construct the base from recycled concrete aggregates. Eventually, it was determined that it is not possible to design or place a recycled drainable and stable base within the design period. Alternative means of achieving efficient subsurface drainage at the joints were investigated, thus leading to the proposal to use geocomposite joint drains (GJD) at the transverse joints.

Research conducted by Rohne & Burnham in 2011 (5) on MnROAD concrete test cells and some network test sections revealed widespread mid-depth scouring of the concrete pavements that were constructed over non-drainable bases such as MnDOT Class 5 aggregate base (MnDOT 2016) (15). Some test cells showed three types of degradation: spalling at the surface, belling or beveling at the bottom, and scouring at mid depth. While the first two were predictable, the third distress type was unexpected. It was explained by the hysteretic phenomenon that occurs with the sequence of load application and release being at a phase-difference from the pavement response. That delay results in periods of negative pressures that bring about cavitation erosion of the concrete at mid depth. Nevertheless, this phenomenon did not occur in pavements that were built on permeable asphalt stabilized bases and some open graded aggregate bases (OGAB). To address this problem, a cost-effective subsurface drainage solution was investigated. This effort included the use of a special open graded aggregate base (OGAB) in 2001 at MnROAD. Unfortunately, the OGAB test cell turned out to be a classic study in loss of support due to material gradation instability. Consequently, GJD was installed in a MnROAD test cell (Cell 613) in 2013 in search for a sustainable subsurface drainage.

As certain open graded aggregate bases may experience instability causing loss of support (3), a joint drainage system placed directly under the transverse joints and above a non-drainable base providing drainability as well as stability is a necessity. By providing savings in pavement thickness that could otherwise have created bridge clearance issues, GJD facilitates many benefits. A drainable base providing lateral transmissivity enhances financial effectiveness when such bases are daylighted beyond the shoulder in lieu of edge drains. Additional cost savings are realized when the drainage layer is thin. Subsurface drainage is an important investment towards the durability of rigid pavements in the purview of financial effectiveness.

5.2 Lateral Transmissivity

Izevbekhai (3) idealized flow in a geocomposite joint drain. In an unsaturated flow condition, the GJD is idealized as a rectangular dorsal-ventrally flattened box section enclosing the flowing fluid. In this case (16):

$$R = \left(\frac{bd}{b+2d}\right); V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{18}$$

where *V* is flow velocity, *n* is a constant characteristic of the medium, *R* is hydraulic radius which is the ratio of the cross-sectional area to the wetted perimeter as expressed for *b* and *d* as width and thickness respectively of the GJD (3/8 inch and 12 inches approximately). *S* is slope which is the camber of the roadway (0.04) in cell 613. Izevbekhai (3) argues that the water intrusion test (discussed in the next section) followed a forced pipe flow which will be idealized by a Hagen Poiseuille flow. Various attempts (20,21) have been made to characterize flow through geosynthetic media. Since the geonet has a honeycomb structure, a modified Kozeny–Carman equation (20,21) is applicable. Kozeny-Carman flow analyzes pressure drop in a medium of packed spheroidal materials with a tractable or determinable sphericity. In using the Kozeny-Carman equation, a porous media within the rectangular cross section is idealized with glass beads and the Poiseuille flow equation is derived. The Kozeny-Carman equation will ordinarily be given (3,16) as

$$\frac{\Delta P}{L} = \frac{180\mu}{\varepsilon^3 D^3} \frac{(1-\varepsilon)^2}{\varphi e^2} v_s \tag{19}$$

where ΔP is pressure drop; *L* is length of drain, μ is dynamic viscosity, ε is porosity of the section; *D* is the diameter of glass beads assumed to be 35 x 10^{-3} inches and φe is the factored glass bead sphericity providing the same head loss as the GJD; v_S is base velocity. For glass beads φe is 0.92. Permeability prediction entails selecting a model expressing *k* in terms of other measurable aggregate properties. Historically, the first approaches were based on a tube-like model of rock pore space known as the Kozeny-Carman relationship. The derivation of this equivalent channel model though mentioned by (Izevbekhai & Akkari 2012) for pervious concrete is relevant to GJD normalized to a channel of glass beads. It was similarly assumed that flow through a porous medium can be represented by flow through a bundle of tubes of different radii (17). Within each tube, the flow rate is low enough that flow is laminar rather than turbulent. The assumption is that each flow path forms a twisted, tortuous, yet independent route through the medium. From considerations of flow through tubes, if *f* is the shape factor and if specific surface area is expressed as $\propto_r$ (the ratio of pore surface area to volume), it can be shown that

$$k = \frac{\varepsilon^3}{f\tau\alpha_r^2} \tag{20}$$

where τ is tortuosity. The equation depends on which volume is used to normalize the pore surface area, *f* and if specific surface area is defined as the ratio of pore surface area to grain volume $\propto_g$, the expression changes. If *k* is permeability and if Formation Factor (F), which has become very important in porosimetry, is introduced as $F = \frac{\tau}{\varepsilon}$ the expression becomes

$$k = \frac{\varepsilon^3}{(1-\varepsilon)^2 f \tau \alpha_G^2} \text{ or } k = \frac{R^2}{Ff} = fF_p^2 \quad (21)$$

Thus, relationship between permeability and porosity depends on the definition of specific surface area (16). It is important to idealize this non-granular obstructive but porous flow medium as an equivalent porous medium of known particulate dimensions as determined by flow experiment. This modifies the Kozeny-Carman equation for GJD by introducing an equivalency factor in comparison to a flow through a glass bead medium of known geometric characteristics. In this case, the GJD is normalized to a flow through a media containing glass beads. This concept will simplify subsequent analysis of clogging in a later analysis outside of the scope of this paper. Based on the proviso postulated by (18) in pervious pavements, predicting the clogged porosity as may also help when clogging occurs in the media. It states that when the clogging agent whose natural porosity is **Va/Vp** introduces a void system **Va** into the cavities, porosity before clogging = **Vp/V** and porosity after clogging (**Va/V) is (Va/Vp) (Vp/V)** (18). This can be expressed as:

Porosity of medium after Clogging = (Natural porosity of clogging agent) *(Porosity of medium before clogging) (22). This can be extended to drainable bases.

5.3 Construction of Test Cells

5.3.1 Original Projects

MnDOT built some test sections with different designs in Interstate 90 in 1998 in Project SP 6780-74 (Eastbound Reference Post 13+0.151 to 20+0.470).

For a test section that was not perfectly engineered, the Interstate Highway 90 test section sufficiently indicated at that time that GJD may be a pavement enhancement based on the RQI observed (**Figure 5.1 and Figure 5.2**). MnROAD subsequently provided the site for an engineered and controlled experiment.

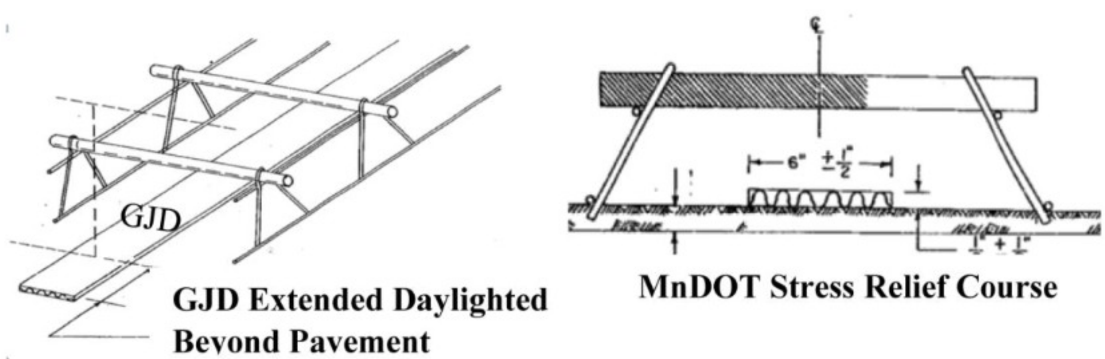


Figure 5.1 Old Interstate 90 Geocomposite Joint Drain Performance Evaluation and Construction

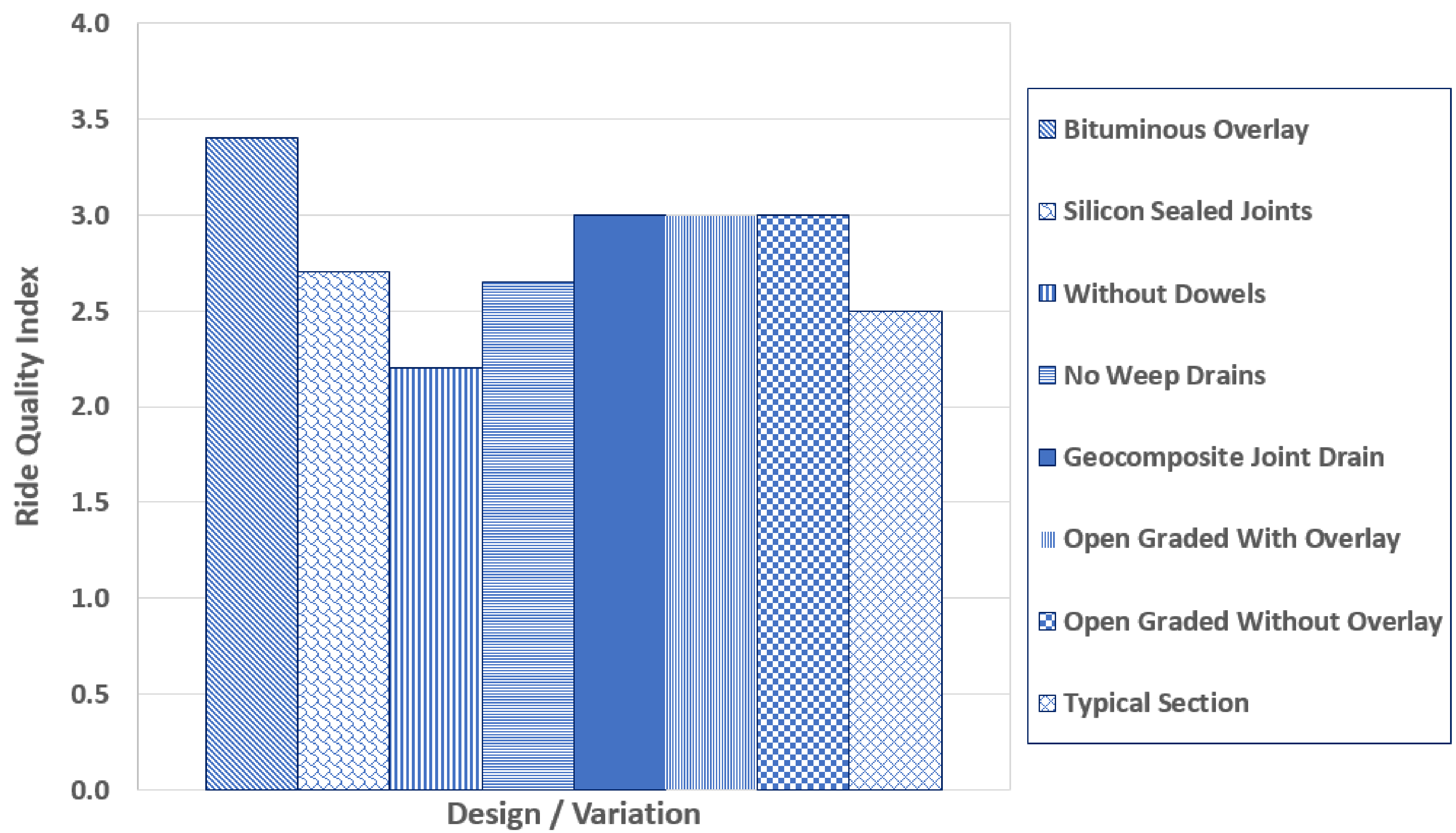
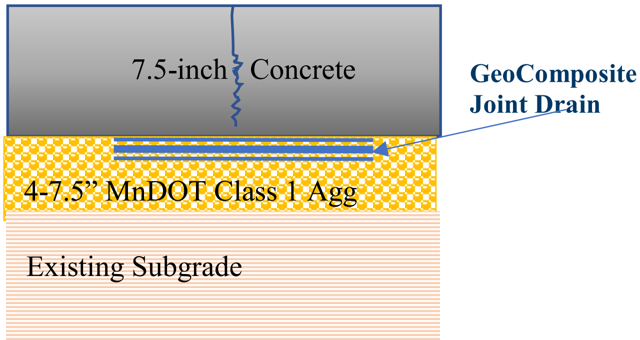


Figure 5.2 Performance of Various Designs Compared to GJD

5.3.2 MnROAD Research 2013 Test Cell

This research was conducted at the MnROAD Research facility. MnROAD, located near Albertville, Minnesota (40 miles northwest of the Twin Cities) is a unique cold region outdoor testing laboratory. MnROAD was originally constructed in 1994 at a cost of $25 million of Minnesota State and United States Federal funding.

MnROAD 2013 Cell 613 was constructed as a 203 mm (8 inch) thick concrete pavement with 75% recycled coarse aggregate and 25% virgin coarse aggregate. The structure is composed of 7.5-inch-thick Portland cement concrete with 15-ft non-skewed joint spacing built on an existing Class 1 aggregate base. The base was prepared by under-cutting and compacting the in situ MnDOT Class 1 base material to accommodate thicker pavement without grade increase. Pavement was constructed with 1-inch dowels in all transverse joints and 30-inch-long ties in the longitudinal joints. Bituminous shoulders were constructed. GJD material was enclosed by the dowel basket, beneath the concrete pavement and above the base material at some joint locations. Splicing was performed twice between the centerline of the pavement and the shoulder by improvising a butt-joint in the geonet layer and lap joints in the non-woven geofabric layers. Dowels baskets and drain were compositely anchored at short (1 ft) interval (**Figure 5.3**). **Figure 5.4** shows an exaggerated view of the GJD.

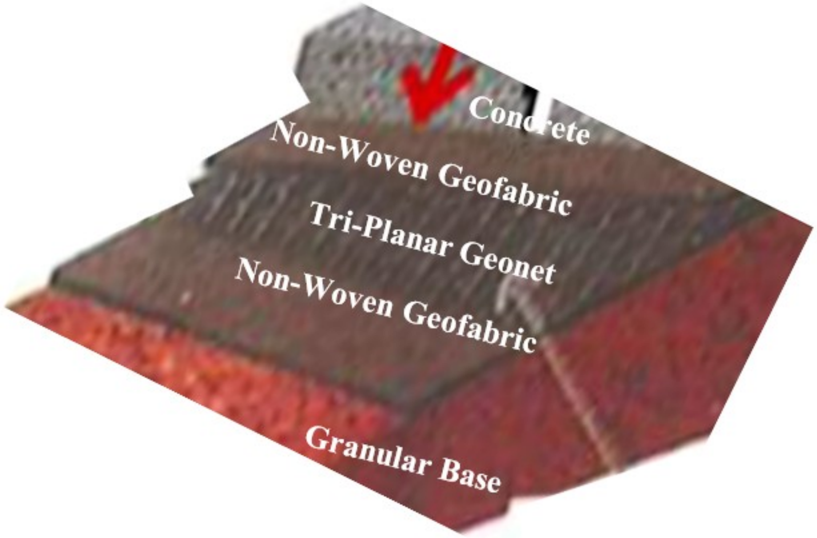


Schematic Layout of the GJD in MnROAD

Mainline Cell 613

Installation of GJD Beneath Dowel Basket

Figure 5.3 GJD Installation at MnROAD



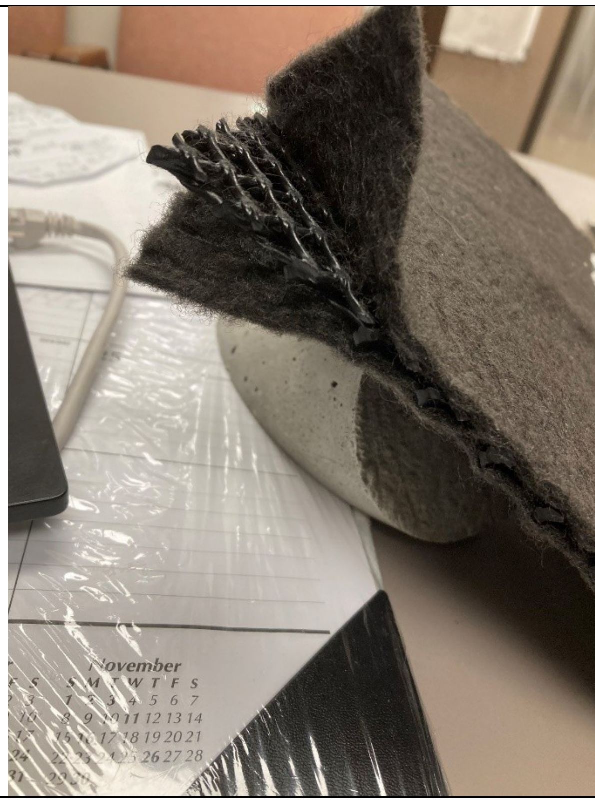


Figure 5.4 Distorted / Fragmented view of the Geocomposite Joint Drain

5.4 Evaluation of GJD

5.4.1 Ponding and Ground Penetrating Radar

In summer of 2014, the installed drains at MnROAD test cell 613 were subjected to quantitative evaluation of lateral transmissivity under load to observe and measure joint drainage enhancement. . A constant head permeameter device designed by MnDOT concrete research team was used (Figure 5.5a). A sequence of ponding and moisture-sensor readings was accompanied by a Ground Penetrating Radar (GPR) survey to detect sub-surface saturation levels. The study scanned the joints with the FHWA's sophisticated 3-D GPR (**Figure 5.5b**) before and after ponding.

(a)

(b)

Figure 5.5 (a) Constant Head Permeameter Testing. (MnDOT Permeameter was designed By MnDOT Concrete Research Group: Bernard Izevbekhai, Steven C. Olson, and Alexandra K Akkari in 2011.) (b) Ponding Tests

Figure 5.6 shows the outflow beyond the shoulder where the GJD daylighted.

Figure 5.6 Sealed Joint Testing (left) and Observation hole (right) (3).

A channel measuring 6 inches high by 10 inches wide was uniformly perforated at the bottom directly above the joints and maintained at a constant hydraulic head of 4 inches.

Higher percentage reflection of a GPR signal was caused by the presence of water in the base at the interface with the PCC (due to the high relative dielectric constant of water). This provided a potential opportunity to "map" the saturation condition in the base based on the amount of reflection at the PCC/base interface depth. Presence of water throughout the PCC layer decreases the percentage of the GPR signal that makes it to the PCC/base interface depth. **Figure 5.7** shows a cross section reflectivity view (top) and plan view (bottom) of the reflectivity at the approximate depth of the dowels. An increase reflectivity at the level of the dowels significantly blocks the GPR signal from being able to evaluate the base saturation. The flooding was conducted by attempting to send the water directly into the joint. Results (**Figure 5.8**) showed contrast of reflectance during and after flooding indicating that the GJD effectively drained the water away. That contrast was not exhibited by the joints without GJD.

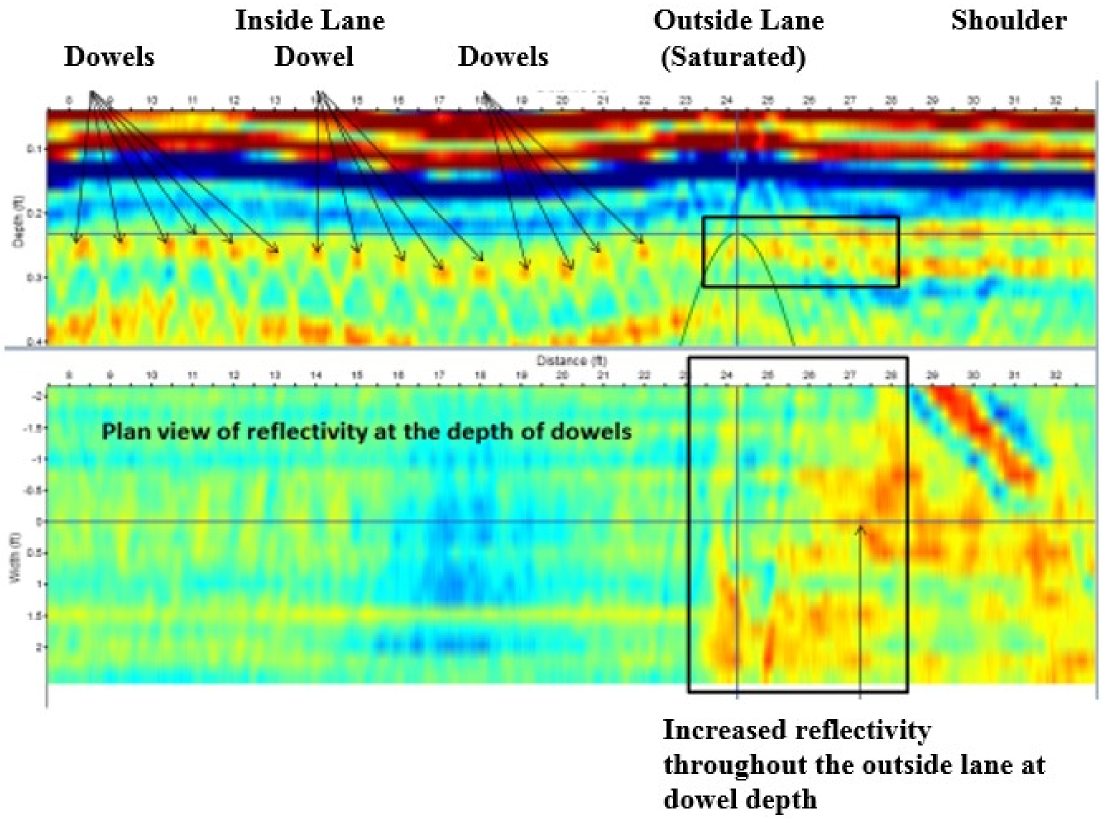


Figure 5.7 GJD 3-D GPR Evaluation (3).

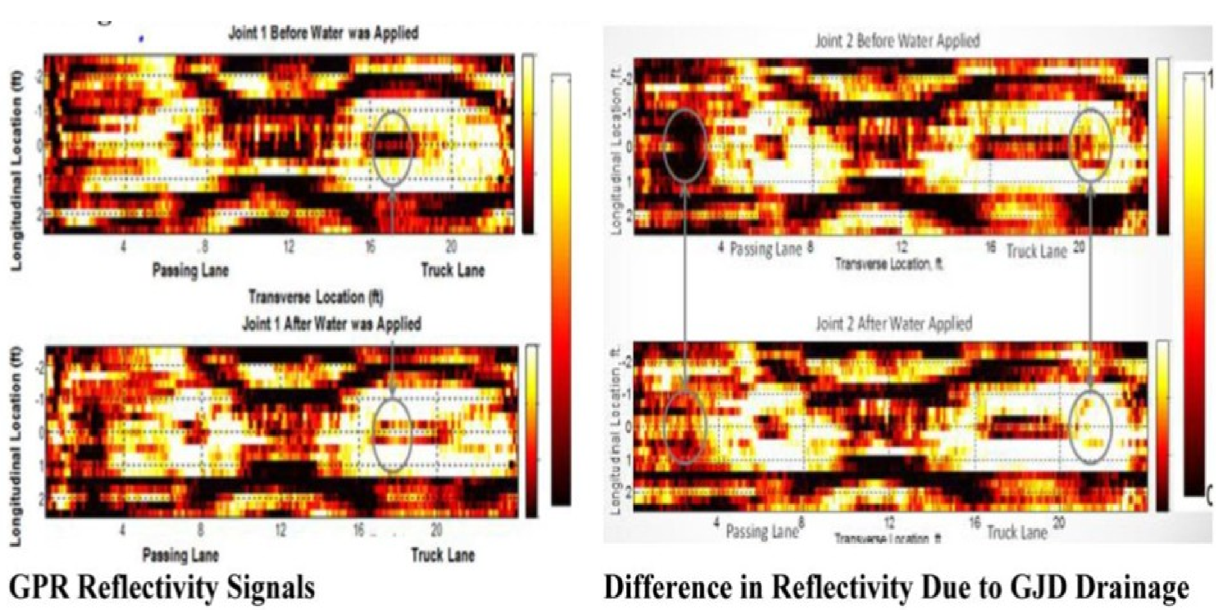
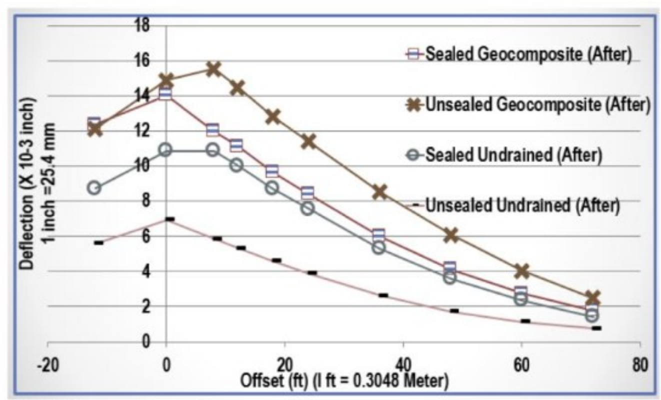


Figure 5.8 GJD flow near the pavement shoulder (3).

5.4.2 Effect of GJD on Load Transfer Efficiency

Researchers had been concerned that use of an additional material beneath the joints may potentially reduce load transfer efficiency (LTE) and increase deflection. As part of the MnROAD GJD experiment, LTE and deflections were measured in the GJD joints and the adjacent non-GJD joints. FWD measurements were also conducted to ascertain if there were excessive deflections at the drained joints. For a comparative analysis, the entire experiment was repeated on joints without the GJD in the same test cell. Limited FWD testing data in **Figure 5.9** shows that there was no evidence of excessive deflections in the GJD joints in comparison to the non GJD joints. Results in **Figure 5.9** show that the curves were not clustered according to GJD vs Non GJD. Moreover, **Figure 5.9** shows that the GJD joints were not distinctively associated with excessive deflections as often feared by some prospective users though in some cases, deflections were occasionally increased by GJD.

Falling Weight Deflectometer (FWD)



FWD Deflections Observed

Figure 5.9 FWD Deflection equipment and experimental results.

5.4.3 Other (Non-Quantitative) Evaluation

Test cell 613 at the MnROAD facility was constructed in 2013 to include a GJD material under some of the transverse joints. After 5 months of service the drains were flooded to determine if they retained the required in–service lateral transmissivity (under load). Four sets of stations each with a water injection point and an observation hole were set up. Two locations unlike the other two joints were sealed with elastomeric preformed sealant and the other two were not sealed. By gravity, water was supplied from a mobile 400-gallon water vessel to a 6-inch diameter single ring cylindrical tube located at a joint 914 mm (3 ft) from centerline. A static head of 18 inches (457 mm) was maintained over the unsealed joints (**Figure 5.5).**

Three out of four test points had a 3-minute run time. Run time was defined as the time taken for the wetting front in the GJD to reach the observation hole or the time the test would be aborted if there was no observable flow. As an outlier, run time of the 4th test point located at a sealed joint (#5215, station 1185+70) was 35 minutes. No evidence of drainage could be seen at an observation hole 25 feet away from the centerline. Another hole was dug closer to the left end of the shoulder, 21 feet away from the centerline. The shoulder had some poorly compacted soil that caused downward flow of the water from the GJD, which explains why lateral flow was not quickly detected at the day-lighted shoulder.

5.5 Network Deployment of GJD

5.5.1 Historical Deployments

In Minnesota, GJD has been used in a few projects. However, in most of these applications it has been used full length along the pavement. MnDOT District 7 used this material directly under the joints 25 years ago on Highway 90. In CSAH 34 in Owatonna, Minnesota, high water table encountered during previous construction had resulted in early deterioration of the pavement material. Subsequent introduction of GJD during reconstruction has since resulted in improved pavement performance. Similar problems were encountered in Interstate Highway 494 at Highway 169 where improvements to the highway proved difficult since the proposed construction left the roadway just a couple feet above the surrounding water table. The GJD facilitated subsurface drainage, and, in addition, improved pavement performance was observed in all 3 cases.

5.5.2 Recent Use

Based on results of the studies done at MnROAD, MnDOT has written a special provisions language for the use of GJD. Currently, MnDOT has developed a Special Provision language for the use of the GJD in program delivery and titled: "(2105) GEOCOMPOSITE JOINT DRAIN FOR USE UNDER CONCRETE PAVEMENT". It provides guidelines for installing a GJD under the proposed concrete pavement in accordance with the provisions of MnDOT 2105, 2301 but with plans modified by the special provision. This is followed by a table that provides limits for lateral transmissivity, permittivity, and flow rate for the composite; density, rib spacing net core thickness and unsupported aperture for the triplanar core and permeability coefficient, strength, and water flow rate for the non-woven geotextile. The special provision recommends extension (without splicing) beyond shoulder and for daylighting. The Engineer measures the number of square yards [square meters] of satisfactorily installed GJD. The Engineer measures for payment according to Item 2105.604 (Geotextile Fabric Special) (MnDOT 2016).

It was used on three state projects in 2016: SP 2771-37 Minnesota Highway 610 new alignment to connect Interstate-94 and existing Minnesota Highway 610. It was also deployed in Project 6285-143 I-694 from Rice Street to Lexington Ave as well as Project SP 8206-45 on US Highway 61 from Trunk Highway 97 to Trunk Highway 37 in Forest Lake Minnesota. Moreover, in 2016 GJD was considered but not used in project SP 5880-186/5880-191 Interstate Highway 35 from Pine City to Hinckley but the factors associated with the decision not to use it were neither material nor performance related.

5.6 Final Notes on GJD

Subsurface drainage is a necessity in pavements. Test sections examined and discussed included full-length application, actual GJD application in Interstate 90 Test Sections and the MnROAD Test section. Each exhibited desirable pavement performance, low IRI, high RQI, and high LTE and low deflection. Some of the joint sealants did not provide a hermetic seal which buttresses the need for sub-surface drainage even when joints are sealed. At some locations of the MnROAD cells shoulder, material being partially compacted may have influenced downward flow from the drains thus complicating water detection. A poorly compacted base or subgrade at the shoulder may confound the efficacy of the GJD in lateral drainage. GJD showed potential for use as an alternative to drainable bases as it is inexpensive in comparison to a Permeable Asphalt Stabilized Base (PASB) or Open Graded Aggregate Base (OGAB). Overall, it appears to be a cost-effective solution to the challenge of subsurface drainage and stability. FWD testing allayed fears of presumably high deflections expected from reversible compressibility of GJD. On a large scale, GJD is cost-beneficial in lieu of Permeable Asphalt Stabilized Base (PASB).

Chapter 6: Operations Research

6.1 Survey Questions

Certain important questions were sent out to the Districts and MnROAD facility. The first question enquired if OGAB in district typically follows/has historically followed the course of Value Engineering/ Special Provision or Specification. The second question enquired if during construction of OGAB the district typically uses quality compaction. Occasionally, it is required that specific percentage proctor compaction is achieved. However, MnDOT exercise higher quality of care with other construction practices especially moisture control. Respondents were requested to assign a percentage to the accuracy of the statement: “Compared to regular base construction we ensure that additional steps are taken to prevent contamination of OGAB or DSB or PASB. For Geo-Composite Joint Drain (GJD) we minimize splicing but in doing this inevitably, we strive to ensure that there is absolute integrity of the butt-joint of the Geo-net and the lap-joint of the non-woven geofabric.” The third question requested percentages of the accuracy of the following statement: “In District (or MnROAD) we actively seek to utilize the drainable bases because we have found them to be associated with good performance in the district network. From my experience I believe the return on investment or cost effectiveness happens after X years of construction roughly”. “If there will be reconstruction in less than X years, I will not use OGAB/DSB/PASB.” This attempted to establish traditionally known break-even year for the additional investment.

The fourth question probed into the known percentage cost increase due to additional cost compared to a regular class 5 aggregate base (percentage increase approximately). Finally, the survey enquired about instability (Some unstabilized drainable bases have been associated with pavement subsidence and loss of support from instability of aggregates): “What steps do you take to ensure that the sources are not associated with unstable aggregates? What extra steps (tests) do you take to ensure that the drainable base is drainable?”

6.2 Questionnaire Responses and Conclusions

The answers from the districts to the questionnaire indicated that the main driver for drainable base construction was the specifications as well as special provisions, whereas the priorities for MnROAD were different. MnROAD gave 80% to value engineering (possibly research needs) and 20% to the specifications, indicating that the research aspect is more important. Also, the answers show that District 6 actively seeks to use OGAB and Class 5Q aggregates in its projects 66-99% of the time but was unsure of the percentage for PASB. District 3 avoids OGAB, PASB, GJD, and Class 5Q, instead preferring to use DSB 66-99% of the time. District 4 prefers PASB, and the other aggregate bases are not used frequently enough to decide. MnROAD actively seeks to use OGAB, DSB, PASB, and GJD in its projects 11-40% of the time, and Class 5Q aggregates in its projects 41-65%. Similarly, most of the drainable bases are more expensive, with District 6 saying OGAB is 80% more upfront, DSB is 54%, Class 5Q is 46%, whereas District 3 says DSB is 30-59% more upfront. MnROAD confirms this, with OGAB and DSB at 30-59% more upfront, GJD and Class 5Q as 0-30% more, and PASB is 60-79% more compared to the usual Class 5.

Districts 3,4, and 6 emphasize the necessity of drainable bases being placed for a long-term solution. The desired break-even year is greater than 20 years for the drainable base sections. Interestingly, none of the predicted characteristic life parameters were near 20 years, indicating that the drainable bases may not be reaching that desired break-even year. It should be noted that the characteristic life parameter estimations themselves are conservative, so the pavement may last longer than the given value. District 3 commented that the drainable bases are often used with unbonded overlays, the data of which was excluded to determine the effect of the drainable base instead of the combined effect of UBOLs/drainable bases. Based on previous experiments done on OGAB pavement, MnROAD said that the break-even year for OGAB is 0-3 years although early failure due to aggregate instability was found. On the other hand, DSB, PASB and Class 5Q all have predicted break-even years between 3-10, but that this is only limited to the MnROAD construction cycles meaning it is difficult to provide this assessment. This agrees with (or is lower than) what was found for the characteristic life parameter for the PASB and OGAB data in all the districts. MnROAD comments that there has not been enough done to give an evaluation of the GJD. District 6 has much larger characteristic life parameters compared to the other districts. This may be due to the fact that there is more data and therefore gives a more accurate estimation of the true characteristic life, but it also may factors beyond their scope of this research. Districts 3,4, and 6 all emphasize the necessity of drainable bases for a long-term solution. This lends credence to take more precautions and give more emphasis on care of the bases themselves, as can be seen in the results from the second question of the survey, where District 6 reported additional steps are taken 66-99% of the time for OGAB, PASB, and Class 5Q to prevent contamination, District 3 reported the same 66-99% for OGAB and DSB, and District 4 reported the same 66-99% for PASB.

It is necessary to ensure that sources of sections are not associated with unstable aggregates, which can lead to compromised subsurface drainage or pavement subsidence and loss of support. To forestall base instability, various tests can be performed to ensure that the base is drainable and stable. District 6 practices Los Angeles Rattler (LAR), QA sampling, and Infrared-Thermography (IR) gradation testing for their OGAB, PASB, and Class 5Q bases and avoids using recycled concrete for their OGAB and PASB sections. District 3 uses LAR, QA sampling, and avoidance of recycled concrete for their OGAB and DSB sections. MnROAD performs LAR and QA Sampling on their OGAB, DSB, and Class 5Q sections. For PASB sections, they only use LAR testing, while for GJD sections, only QA Sampling is used. They also use the stability equation to test their OGAB sections. Because District 6 performs more testing on the aggregates compared to District 3 or District 4, District 6 has a more comprehensive understanding of how the aggregate will behave once placed in the road. Therefore, it is expected that District 6 would have better drainage performance because the more information available about the aggregate means they wield some proficiency and experience in OGAB in that district. There was no real evidence of the same proficiency in other drainable base types in this district.

The characteristic life of pavements is dependent on many factors, including the aggregate itself as well as maintenance practices. The questionnaire shows that District 6 does more testing on the aggregate compared to other districts, thereby giving a more comprehensive understanding of aggregate behavior and how to better maintain the base. All districts emphasized the need for the pavement to be a long-term solution and that taking care of the aggregates was a vital step to avoid contamination and other issues for the districts' drainable base of choice. It can be inferred that the extra knowledge obtained from testing and care given to avoid contamination enables districts with such practices to have longer RSL. Overall, there is not a significant difference between the RSL of OGAB/PASB versus the non-drainable base contiguous section statewide for only some districts, indicating that the RSL being significantly larger must be due to some factors of proficiency in the design and construction of the drainable base in certain districts.

Chapter 7: Conclusions and Recommendations

Analyzing the RQI of the drainable base sections compared to their contiguous counterparts showed that, for all sections, the majority in the given population are wearing out due to age, not failing prematurely due to early wear-and-tear, construction flaws, or randomly. The other parameter values obtained from the reliability analysis and the t-tests showed a significant difference between the RSL of the drainable base and non-drainable base sections for about half of the districts, which is essentially inconclusive from a statewide perspective. That is, half of the districts concluded that the drainable base sections have larger RSLs, and the other half was the opposite. Therefore, the difference between the systems was advantageous for some of the districts, and this distinction was related to the results found in the survey in which the districts that obtained as much knowledge and understanding of certain drainable base types had greater success in those types.

In terms of GJD, each test section at MnROAD exhibited desirable pavement performance. It was noted that some of the joint sealants of the GJD material did not provide a hermetic seal, which provides support for the need of sub-surface drainage even when joints are sealed. At some locations of the MnROAD cells shoulder, material being partially compacted may have influenced downward flow from the drains thus complicating water detection. A poorly compacted base or subgrade at the shoulder may have confounded the efficacy of the GJD in lateral drainage. This means that GJD shows potential as an alternative to drainable bases as it is inexpensive in comparison to other drainable bases and, in addition, has good performance. FWD results put to rest fears of high deflections expected from reversible compressibility of GJD. Additional testing and monitoring should continue, to determine if these conclusions continue to be a valid as the pavement ages.

Weibull analysis was used to study the success of each subsurface type/section and to study how performance characteristics changed over time. In this report, the performance of Open Graded Aggregate Base and Permeable Asphalt Stabilized Base was studied. The study compared the RSL, Preventative Maintenance Window or threshold Time to Failure (TTTF), and failure pattern (shape parameter) of base types were compared to contiguous bases without that unique drainability feature. The RSL followed the institutional definition of how many years until pavement degrades to an RQI of 2.5. The TTTF indicates how many years after the initial construction are available to improve the life of the pavement through minor preventive maintenance activities when the pavement is still in a relatively pristine condition. Failure pattern shows how the population of a network or group of pavements fails, where a higher value means most roads in the given population are wearing out due to age (the goal of a good design), and a lower value means most of the sections in that network in the given population are wearing out early, likely due to construction flaws or characteristics of the design and construction techniques that cause the road to fail prematurely. This is referred to as “Burn-in” or “Infantile Failure.” This is most undesirable in pavement design. Performing Weibull analysis allowed the identification of the parameters μ, t_0, and β with those respective performance measures (the shape/threshold parameter is β, μ is the scale/characteristic life parameter, and t_0 is the location/shift parameter, which is the time to failure).

The Weibull analysis on drainable base sections versus their contiguous non-drainable counterparts did not give a definitive answer as to whether the RSL of the drainable base sections were extended significantly, because about half of the districts had an RSL of the drainable base that was less than the non-drainable sections. This was confirmed with the t-test results. The evaluation of GJD seemed to show that it was a feasible and cost-effective subsurface drainage option.

Operations research of district drainable base practices was also conducted. Although the population was limited, there was sufficient evidence to associate good performance of certain drainable bases with the experience and proficiency in the engineering of those base types. This calls for a cross-training or a drainable base forum where experience and ideas can be shared to maximize the potential benefits of drainable bases.

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