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Structural Evaluation of Asphalt Pavements with Full-Depth Reclaimed Base

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16. Abstract (Limit: 250 words)

Currently, MnDOT pavement design recommends granular equivalency, GE = 1.0 for non-stabilized full-depth reclamation (FDR) material, which is equivalent to class 5 material. For stabilized full-depth reclamation (SFDR), there was no guideline for GE at the time this project was initiated (2009). Some local engineers believe that GE of FDR material should be greater than 1.0 (Class 5), especially for SFDR. In addition, very little information is available on seasonal effects on FDR base, especially on SFDR base. Because it is known from laboratory studies that SFDR contains less moisture and has higher stiffness (modulus) than aggregate base, it is assumed that SFDR should be less susceptible to springtime thawing.

Falling Weight Deflectometer (FWD) tests were performed on seven selected test sections on county roads in Minnesota over a period of three years. During spring thaw of each year, FWD testing was conducted daily during the first week of thawing in an attempt to capture spring thaw weakening of the aggregate base. After the spring thaw period, FWD testing was conducted monthly to study base recovery and stiffness changes through the seasons.

GE of SFDR was estimated using a method established by MnDOT using FWD deflections, and the GE of SFDR is about 1.5. The value varies from project to project as construction and material varies from project to project. All the materials tested showed seasonal effects on stiffness. In general, the stiffness is weaker in spring than that in summer and fall.

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Chapter 1. Literature Review

Full-depth reclamation (FDR) is a recycling technique where the existing asphalt pavement and a predetermined portion of the underlying granular material are blended to produce an improved base course. FDR is an attractive alternative in road rehabilitation: resources are conserved, and material and transportation costs are reduced as recycling eliminates the need for purchasing and hauling new materials and disposing of old materials. An additive is sometimes used, and this process is referred to as stabilized full-depth reclamation (SFDR).

Two approaches in pavement design involve the Structural Number and Granular Equivalency. The Structural Number, which is used widely throughout the United States, has been applied in many FDR projects. The Granular Equivalency (GE), which is popular in Minnesota, has no known reclaimed asphalt pavement designs. In this work, GE will be evaluated for both stabilized and standard FDR sections through Falling Weight Deflectometer (FWD) testing.

1.1 Background

It is inevitable that, over time, asphalt pavements degrade due to a variety of reasons including thermal cracking, traffic loading, or poor construction. In past years, common methods to rehabilitate failed asphalt pavements were to either apply hot-mix asphalt (HMA) overlays or to perform complete reconstruction of the pavement section. However, to fully reconstruct a pavement is expensive and time consuming, and although the overlay method is fast and less expensive, it does not always provide a long lasting solution. With overlays, previous distresses and cracks eventually reflect up to the new layer of pavement, thus requiring further rehabilitation. In the past few decades, in-place asphalt recycling has become cost-effective and has gained popularity. The Asphalt Recycling and Reclaiming Association has categorized recycling into five methods: cold planning, hot recycling, hot-in-place recycling, cold recycling, and full-depth reclamation [1].

1.2 Recycling Methods

Both cold and hot recycling can be performed on-site or off-site at a central facility. There are advantages and disadvantages for in-situ recycling, which will be briefly discussed in this section.

For hot-in-place recycling, a vehicle train with four components is used. The first two units in the vehicle soften the existing asphalt. Then the next unit mills the softened pavement, adds a rejuvenator, and then “windrows” the mix. Last, a mixing unit combines the rejuvenated blend with a virgin material and then agitates the combination in a pugmill. After the final mix is discharged, conventional methods of HMA paving proceed. Because all material is mixed and blended on-site, no material must be transported. However, the equipment used for hot-in-place recycling is expensive and specialized [2].

In hot recycling, the pavement is milled and then transported to a central facility where the material is heated and blended into a recycled mix and then hauled back to be re-used. Although the equipment is more common and less expensive, higher transportation costs are incurred.

An obvious benefit of cold recycling is that energy is conserved by not heating the materials. There are two methods of cold recycling commonly used: cold planing and cold-in-place recycling. During cold planing, the existing asphalt course is cold milled to achieve a specified vertical profile. Any surface distresses and irregularities are removed, leaving a uniform surface. The benefits of cold planing include an improved pavement cross-section, minimal traffic interruption, and a low cost [1].

Cold in-place recycling is also performed by using specialized train of equipment. First, while the existing pavement is milled, an asphalt or chemical is injected to achieve the proper compaction moisture content. Next, the newly mixed pavement is profiled with a grader, and then compacted with a vibratory roller. Last, a fresh surface is applied [3].

It is important to determine if the cause of pavement distress is structural. There are many distresses, such as fatigue in wheel paths, rutting, and reflective cracking that can indicate structural inadequacy [4]. The first four methods are very effective for fixing minor pavement distresses, but do not address structural or base problems because only the top layer of bituminous material is recycled [5]. The fifth method, full-depth reclamation (FDR) eliminates more serious base and structural pavement issues by recycling the entire asphalt section and a predetermined amount of granular base material.

1.3 Overview of Full Depth Reclamation

The advantages of FDR include, (a) improving the pavement structure without changing the geometry of the road, and (b) restoring the pavement to its desired profile while eliminating rutting, thermal cracking, and reflective cracking. Also, because FDR can include stabilizing the base, frost susceptibility can be reduced [2]. It has been stated that FDR is 25-50% lower in costs than conventional pavement rehabilitation efforts [6]. FDR is sustainable by preserving virgin materials and preventing the disposal of used material, and air quality issues, such as dust and smoke, are minimized due to the nature of the processes [2]. After the FDR process is performed, a surface layer is applied.

The process for FDR is very similar to cold-in-place recycling, and often times the same specialized vehicle is used for the two methods. There are five main steps in FDR: pulverization, blending of materials, shaping, compaction, and application of the surface course [7]. In addition, there are four types of operations used to reclaim pavement: multistep sequence, two step sequence, single machine, and single pass equipment [2].

In a multiple-step sequence, there are different machines used for each step. To mill the pavement, a modified roadheader can be used, and to blend mixed materials, a rotary mixer can be used. The advantages of the multiple-step sequence are that the equipment is readily available and costs are relatively low. The disadvantages are that there is a lack of uniformity in the depth of the cut, multiple passes are required to achieve the desired size of granular material, the production rate is low, and the multiple machines can pose traffic control issues [2].

In a two-step sequence, a cold milling machine is used to grind existing pavement. Next, soil stabilization mixing equipment is used to blend materials. The advantages of using the two-step sequence are that the machines can accurately control the depth of removal, and they can perform pulverizing and grading in a single pass, which results in less traffic interference. A disadvantage is the possible production of oversized aggregates [2].

By using a single machine, the breaking, pulverizing, and blending of the pavement and stabilizing agents can be performed in a single pass. The single machine operation is advantageous because of the high production rate, but disadvantages include the possibility of yielding oversized aggregates, a depth limitation for cutting, and the use of specialized equipment [2]. The single-pass equipment train uses a special vehicle that first mills the existing pavement, crushes the material to a desired gradation, adds stabilizing agents and blends the mixed materials, and finally paves. There are many advantages to this operation, including a high production rate, the ability to accurately remove the desired quantity of asphalt, and the elimination of oversized particles. However, the equipment necessary is specialized and very expensive [2].

1.4 Stabilizing Methods

As previously mentioned, a stabilizer may be added to increase the strength of the base. There are three types of stabilizing additives: chemical, bituminous, and non-traditional, including enzymes [5]. Compaction densifies the material and is sometimes considered as stabilization, but in this work it will be called FDR, as the reclaimed material should always be compacted in appropriate lifts prior to paving [8].

1.4.1 Chemical Stabilizers

Chemical stabilizers include Portland cement, hydrated lime, calcium chloride, and coal fly ash [5]. Portland cement reacts with moisture in the soil, and the cementitious, hydration reaction causes the particles to bond. Also, because the cementitious reaction continues with time, long term strength improvement can be observed. In a study conducted by the Georgia Department of Transportation using a cement treated base, it was concluded that approximately 6 in. (152 mm) of cement treated base (CTB) was equivalent in strength and stiffness to 8 in. (203 mm) of a crushed stone base. In addition, using FDR with a CTB resulted in a 42% reduction in costs from the conventional rehabilitation methods [9].

Lime products used to stabilize soil include (1) quicklime, which is calcium oxide, (2) hydrated lime, which is calcium hydroxide, and (3) lime slurry, which is hydrated lime and water [10]. Lime, however, requires a minimum amount of clay to react favorably. Lime stabilization causes a significant improvement in soil texture and structure by reducing plasticity and by providing pozzolanic strength gain. The pozzolanic reaction is the formation of calcium silicate hydrates as the lime reacts with the aluminates and silicates in the clay minerals. The calcium silicate hydrates produced in the reaction exhibit high strength and are the molecules responsible for the strength gain in cementitious reactions. Like Portland cement, the pozzolanic reaction is time dependent, and long-term strength improvement in lime stabilized soils and aggregates are possible [11].

Both Portland cement and lime stabilized bases increase the shear strength of the base, decrease permanent deformation, and reduce moisture susceptibility. However, shrinkage can often be a problem. In addition, due to the high calcium contents in cement and lime, the materials are subject to sulfate attack [8].

Unlike lime, coal fly ash does not require clay to react. However, the quantity required to produce a similar outcome is three to four times that of lime [12]. Coal fly ash, which can often be used as a Portland cement substitute, is a by-product of coal manufacturing. During combustion, the fly ash becomes infused with inorganic particles. There are two classes of fly ash. Class F fly ash comes from bituminous coals that have lower levels of calcium content. Class F fly ash shows no self-cementing properties, and is used less often than Class C fly ash. Class C fly ash is produced from sub-bituminous coal which has higher concentrations of calcium carbonate and thus becomes self-cementing [13]. Also, because Class C fly ash exhibits such rapid rates of hydration, retarders are often required in construction. Like cement and lime, fly ash is also susceptible to sulfate attack due to the calcium concentrations.

Calcium chloride, another chemical stabilizer, is often used to control dust due to its hydrophilic and deliquescent properties, meaning it absorbs water and then dissolves. This reaction can produce high strength bonds [14]. Calcium chloride penetrates the aggregate in the base and coats the particles and binds them together. Calcium chloride is an alkaline earth metal salt and is most stable in liquid form with six molecules of water in its structure. However, it is also commercially available in a dry, flake form. In addition, it can reduce the plasticity index (PI) of a soil and improves workability while maintaining strength. Research has shown that calcium chloride used along with Class F fly ash leads to high early strength [15].

1.4.2 Bituminous Stabilizers

Bituminous stabilizers commonly used are slow or medium asphalt emulsions, which can be polymer modified. Asphalt emulsions work by reducing moisture susceptibility while maintaining strength and providing flexibility. However, using an emulsion in moist soil can increase the moisture content to detrimental levels [5]. In addition, asphalt emulsions take time to cure.

Another popular stabilizer is foamed asphalt. In one study, it was determined that foamed asphalt stabilized recycled asphalt pavement (RAP) outperformed RAP stabilized with asphalt emulsion. Foamed asphalt is less expensive than asphalt emulsions and it exhibits rapid strength gain [5, 16]. Foamed asphalt typically requires a higher percentage of fines, and cement is sometimes added to meet this requirement.

Foamed asphalt is a mixture of air, water, and hot asphalt. Cold water is introduced to hot asphalt, causing the asphalt to foam and expand by more than ten times its original volume. During this foaming action, the asphalt has a reduced viscosity making it much easier to mix with aggregates. Due to the immediate strength gain, traffic can operate on the stabilized base until a hot mix asphalt base and wearing surface is applied [18].

1.4.3 Nontraditional Stabilizers

Although lime, cement, fly ash, and bituminous materials are sufficient for improving the stability of granular materials, the cost has increased in recent years. The increased cost has pushed companies to develop new, enzyme additives [19]. Scholen has categorized nontraditional stabilizers into five categories: ionic, enzymatic, mineral filling, clay filling, and polymer. Ionic stabilizers catalyze a cation exchange and flocculation of clay and soil particles. Ionic stabilizers reduce plasticity and swell potential, which increase strength. In an enzymatic reaction, the enzymes bond and are attracted to the large, negatively charged organic molecules in the soil minerals [8, 20].

Stabilizers must be chosen with careful consideration of the base material to ensure proper strength gain and limited sulfate attack. Kearney [5] summarizes general guidelines for selecting an appropriate stabilizer. Other states have used SFDR, and the following are some examples.

1.5 Case History 1

In 2003, four projects in Maine were selected to determine the structural strength of stabilized FDR with foamed asphalt. The test plan consisted of performing FWD tests, obtaining samples, and conducting laboratory tests. The objective of the project was to determine, from test data, the structural layer coefficients of foamed asphalt layers and recommend appropriate strength of foamed asphalt mixes to be used in Maine [18].

The work involved pulverizing the existing HMA surface together with approximately 2 in. (50 mm) of the underlying gravel. After initial reclaiming, the material was then graded and compacted. The roadway was reclaimed with foamed asphalt to a depth of 5.9 in. (150 mm), and then surfaced with 1.2 in. (30 mm) of asphalt.

The subgrade resilient moduli of the four Maine projects were determined through the backcalculation of the falling weight deflectometer results using the following equation:

$$M_r = \frac{0.24P}{d_r \times r} \tag{1}$$

where,

P = applied load
d_r = deflection at a distance r from center of the load
r = distance from the center of the load
M_r = backcalculated subgrade resilient modulus

Table 1: Subgrade Resilient Modulus Results

	Belgrade	**Orient**	**Farmington**	**Macowahoc**
M_r, psi	21564	19383	15842	29342

After computing the resilient modulus of each section, the effective modulus above the subgrade was determined using the following equation:

$$d_o = 1.5Pa\left[\frac{1}{M_r\left[1+\left(\frac{D}{A}\left(\frac{E_p}{M_r}\right)^{1/3}\right)^2\right]}^{0.5}\right] + \frac{1-\frac{1}{1+\left(\frac{D}{a}\right)^2}}{E_p} \tag{2}$$

where,

d_0 = temperature corrected central deflection, in.
a = load plate radius, in.
A = load plate area, sq.in.
p = load pressure, psi
D = total thickness of all pavement layers above subgrade, in.
E_p = effective modulus of pavement layers above subgrade, psi

Table 2: FWD Data

	Belgrade	**Orient**	**Farmington**	**Macowahoc**
d_0,mils	8.52	16.6	11.37	8.1
a, in	6	6	6	6
A, sq. in	113.1	113.1	113.1	113.1
P	80.5	80.6	79.8	78.9
D, in	32.2	22	28	22
E_p, psi	135979	67615	109165	152375

Next, the structural coefficients for each layer were determined:

$$SN_{eff} = 0.0045DE_p^{1/3} \tag{3}$$

Because the subgrade resilient modulus was known, the layer coefficient for the subbase could be determined. Once the effective structural number was calculated, the layer coefficient for the base could be computed:

$$a_2 = \frac{SN_{eff} - a_1D_1 - a_3D_3}{D_2} \tag{4}$$

Table 3 lists the computed structural layer coefficients for the FDR foamed asphalt stabilized base.

Table 3: Foamed Asphalt Base Layer Coefficients

Project Section	Layer Coefficient
Belgrade - Rt 8	0.22
Orient Cary - Rt 1	0.23
Farmington - Rt 156	0.22
Macwahoc - Rt 2A	0.35

1.6 Case History 2

Romanosci [16] conducted an experiment to determine the effective structural coefficients of foamed asphalt SFDR. The experiment consisted of four pavement sections—all with a silty clay subgrade and three inches of HMA. Out of the four sections, one was constructed with a 9 in. (229 mm) conventional AB-3 granular base and three constructed with a foamed asphalt SFDR base of 6, 9, and 12 inches. The blended base material was composed of 50% RAP, 37% AB-3 granular material, 12% A7-6 soil, and 1% Portland cement (to offset the high plasticity of the clay). The optimum water content was determined to be 3% and asphalt content was determined to be 3% of PG 64-28 [16].

Following the preparation of the pavement mixes and sections, accelerated testing was performed. A single axle and dual axle wheel were used to simulate loading, and then in addition FWD and weight drop tests were performed. Weight drop tests are similar to FWD while using a smaller load, a larger load plate, and longer spacing between sensors.

The effective structural number was computed using the AASHTO method and the measured deflections. In this experiment, the layer coefficients needed to be determined for the asphalt wearing course and the base. Romanosci assumed a structural coefficient for the asphalt surface was assumed to be 0.42, a typical value. Because the layer coefficient for the asphalt course and the depths of each layer were known, the coefficient for the base could be determined:

$$a_2 = \frac{SN_{eff} - a_1 D_1}{D_2} \tag{5}$$

By using the AASHTO method of pavement design, the average structural layer coefficient of the foamed asphalt stabilized base was estimated to be 0.18.

1.7 Case History 3

In another study conducted by Wen et al. [25], Class C fly ash was incorporated into the FDR. The asphalt layer was milled and blended with 8% fly ash and 5% water in-place to become the rejuvenated base material. To evaluate the structural contribution of the fly ash, FWD tests were performed both four days and one year after construction [26].

Twenty-three FWD tests were conducted on CTH JK in 2001 and 22 tests in 2002 at an interval of 100 ft along the roadway. The data were then used in the programs Modulus 5.1 and Michback to backcalculate the moduli of each pavement layer. It is important to note that the backcalculated modulus of the fly ash stabilized base increased 49% from 2001 to 2002, which reinforces the observation that fly ash reactions continue over time.

Using the backcalculated moduli values, the structural number was estimated using:

$$SN = [1.49 \times ET^3]^{1/3} \tag{6}$$

$$\log(ET)^3 = 5.03 - 1.309\log(AUPP) \tag{7}$$

$$AUPP = \frac{1}{2}\left(5D_0 - 2D_1 - 2D_3 - D_4\right) \tag{8}$$

where,

SN = structural number of pavement (mm),
ET = flexural rigidity of pavement (mm),
AUPP = area under pavement profile (mm), and
D_i = surface deflection (mm).

With SN, the structural coefficient of the asphalt layer, a_1, was estimated using the AASHTO method, and then the layer coefficient could be determined for the base layer using equation (5). The coefficient of the fly ash stabilized base was 0.16 (2001) and 0.23 (2002).

1.8 Summary

Full-depth reclamation (FDR) is a recycling technique gaining popularity as it decreases or eliminates the need for purchasing and transporting new material. In this literature review, an overview of the FDR process was presented along with methods and materials used for stabilization. In addition, the structural layer coefficients were identified in various projects for SFDR pavement.

Chapter 2. Analysis Methods

2.1 The AASHTO Method and Structural Number

In the American Association of State Highway and Transportation Officials (AASHTO) method of flexible pavement design, the structural number (SN) is an AASHTO index of pavement strength based on layer thickness and material properties. SN is commonly used in pavement design practices and expresses the capacity of pavements to carry loads for a given combination of soil support, estimated traffic, terminal serviceability, and environment.

On the other hand, the "Minnesota Method" of pavement design incorporates the Granular Equivalency (GE), which indicates the contribution of a given layer of pavement material relative to the performance of the entire pavement section. It is dependent upon the properties of that layer in relation to the properties of the other layers. The relative thickness between the layers is known as the granular equivalency factor. The layer equivalency can be determined by laboratory and field tests.

Layer coefficients used in the AASHTO pavement design method are also used to describe material stiffness, which is similar to the GE factor. Therefore, layer coefficients of the base materials of the tested project were calculated and used to estimate GE values.

The AASHTO Guide for the Design of Pavement Structures, originally published in 1961, has been the primary pavement design approach in the United States. The AASHTO Guide is based on the results of the AASHO (American Association of State Highway Officials) Road Test conducted in Illinois in the late 1950's. The first interim design guide was published in 1961, and subsequent revisions occurred in 1972 and 1981, with the current edition expanded and revised in 1986 and 1993. This method incorporates several design variables such as traffic loading, environmental effects, serviceability, pavement layer thickness, and pavement layer materials. In addition, the AASHTO method incorporates a level of uncertainty in the process to ensure that the design will last; the level of reliability must increase as the traffic volume increases [21].

The Guide for Design of Pavement Structures first requires the desired terminal serviceability to be determined. The serviceability is expressed as an index from 4.2 to 0, where 4.2 is a newly constructed flexible pavement and 2.0 is a pavement in need of rehabilitation. Next, the known traffic volumes must be converted to the number of equivalent 18 kip single axle loads (ESAL). Then the structural number (SN) can be determined by using design charts or a computer program. After the SN is known, the layer coefficients are evaluated and the required layer thicknesses are computed.

From the AASHO Road Test, the original equation is as follows:

$$\log W_{t18} = 9.36\log(SN+1) - 0.2 + \frac{\log[(4.2 - p_t)/(4.2 - 1.5)]}{0.4 + 1094/(SN+1)^{5.19}} \tag{9}$$

where,

W_{t18} = Number of 18-kip single axle load applications at time, t
SN = Structural Number of pavement
p_t = Serviceability at time, t

In the original test, the Structural Number was determined from the following equation:

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{10}$$

where,

a_1, a_2, a_3 = layer coefficients for the surface, base, and subbase, respectively
D_1, D_2, D_3 = layer thickness, respectively

The layer coefficients are a measure of the relative ability of a unit thickness of a material to function as a structural component of the pavement. The coefficients can be computed with regression equations. For the asphalt layer, the coefficient, a_1, used in AASHO road tests are determined from a design chart or equation, but is often taken as 0.44. For a base and subbase courses, the coefficients are determined from the following equations:

$$a_2 = 0.249(\log E_2) - 0.977 \tag{11}$$

$$a_3 = 0.227(\log E_3) - 0.839 \tag{12}$$

where E_2 and E_3 are the resilient moduli for the base and subbase courses can be determined either by testing or from an AASHTO equation as a function of the stress state and moisture conditions:

$$E_i = K_1 \theta^{K_2} \tag{13}$$

The constants K and θ can be determined from AASHTO tables [22].

Equation (9) is only applicable for pavements with an effective subgrade resilient modulus of 3000 psi (20.7 MPa). To apply the equation to other subgrade conditions, and incorporate reliability, the equation was modified:

$$\log W_{t18} = Z_R S_{0+} 9.36 \log(SN+1) - 0.2 + \frac{\log[\Delta PSI/(4.2-1.5)]}{0.4 + 1094/(SN+1)^{5.19}} + 2.32 \log M_R - 8.07 \tag{14}$$

where,

ΔPSI = Loss of serviceability (represents the level of serviceability loss the designer is willing to accept due to traffic loads)
M_R = Effective subgrade soil resilient modulus

Z_R = Normal deviate for a given reliability, R
S_0 = Standard deviation

In the modified equation, the Structural Number includes drainage conditions:

$$SN = a_1 D_1 + a_2 D_2 m_3 + a_3 D_3 m_3 \qquad (15)$$

where,

a_1, a_2, a_3 = layer coefficients for the surface, base, and subbase, respectively
D_1, D_2, D_3 = layer thickness for the surface, base, and subbase, respectively
m_1, m_2, m_3 = drainage coefficients for the surface, base, and subbase, respectively

The following are typical AASHTO structural layer coefficients obtained from a variety of recycled test sections using several types of recycled materials. Layer coefficients for cold-recycled mixes can be determined from these values [4]:

- Coefficients for foamed-asphalt recycled layers range from 0.20 - 0.42
- Coefficients for emulsion recycled layers range from 0.17 - 0.41
- Coefficients for cold recycled mixes are between 0.30 - 0.35

2.2 The Minnesota Method and Granular Equivalency

In Minnesota, a different method is used in flexible pavement design, which is based on the subgrade R-value determined by laboratory testing [23]. The design method includes traffic loading (ESALs) and material properties (R-value) [27].

The R-value can be determined in a laboratory with the Hveem Stabilometer test, which is a type of triaxial test performed by measuring a compacted soil's resistance to deformation [24]. In addition, in Minnesota Investigation 201 conducted by the Minnesota Department of Highways, a relationship between the subgrade modulus and R-value was determined:

$$\text{R-value} = (0.41 + 0.873 M_R)^{1.28} \qquad (16)$$

After the R-value is computed, and the design number of ESALs is known, the required total Granular Equivalency can be determined from Figure 1.

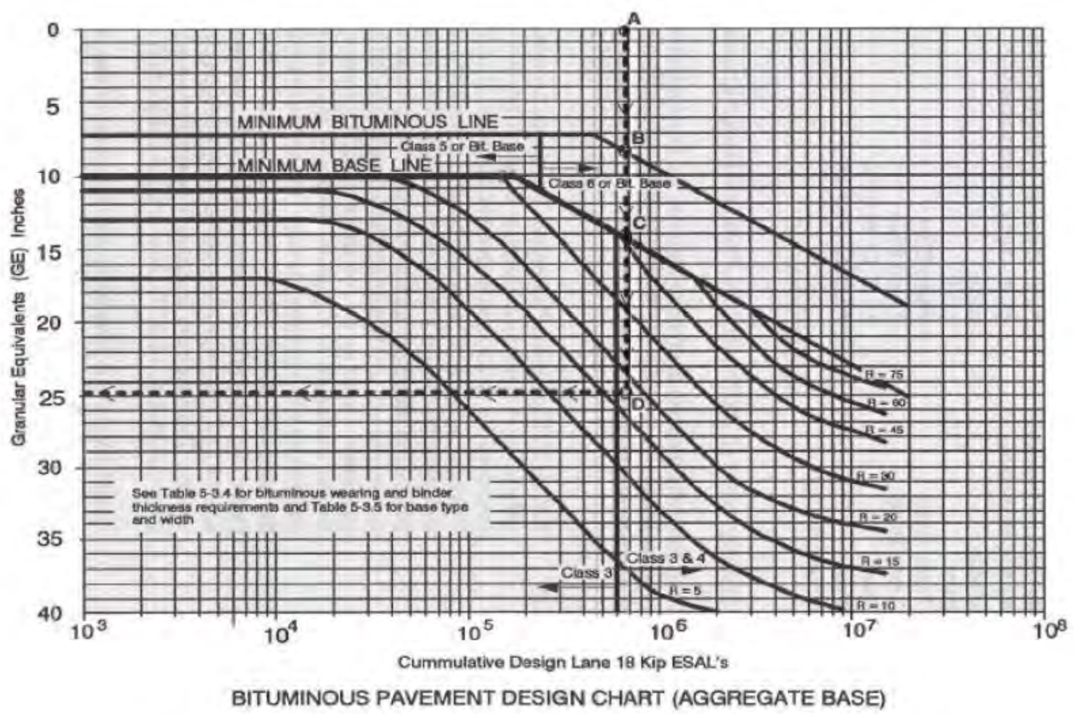

Figure 1: Minnesota Method Design Chart [23]

The design chart not only provides the required Granular Equivalent (GE) for the entire pavement section, but it provides the minimum GE for the asphalt and base courses. Once the required GE is known, the pavement can be designed by using

$$G.E. = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{17}$$

where,

GE = Granular Equivalent,

a_i = granular equivalent factors for surface, base, and subbase, respectively,

D_i = thickness of respective layers.

In equation (17), the constants, a_1, a_2, and a_3 represent the required depth of a given material to replace a class 5 or 6 base. The granular equivalent factors, a_i, were determined in the late 1960's through extensive testing and data analysis.

In order to determine the GE, the relative effect of the layers, based on deflections, was established using the Benkelman Beam. The following equations express the relationship between load deflection and thickness.

$$\log(d_s) = 0.76 + 1.09\log(L_1) - 3.32\log(0.056D_1 + 0.016D_2 + 0.026D_3 + 1) \tag{18}$$

$$\log(d_f) = 1.06 + 1.54\log(L_1) - 4.60\log(0.140D_1 + 0.021D_2 + 0.031D_3 + 1) \tag{19}$$

where,

d_s = spring rebound deflection, in units of 0.001 in.
d_f = fall rebound deflection, in units of 0.001 in.
L_1 = axle load, in units of kips
D = thickness of surface, base, and subbase.

Using equations (18) and (19), the thickness indices can be converted to gravel equivalent factors.

From elastic theory, pavement deflections can be predicted if the elastic properties of the materials are determined under the same conditions of test as in the field. The following equation can be fit to the elastic theory prediction of deflection.

$$\log(d) = a_0 - a_1 D_1 - a_2 D_2 - a_3 \log(E) \tag{20}$$

where,

d = deflection, 0.001 in.
D_1 = surface thickness, in.
D_2 = granular base plus subbase thickness, in.
E = Young's modulus of embankment soil, psi.
a_0, a_1, a_2, a_3 = constants determined from regression analysis, which correspond with pavement layers

The deflections measured were correlated with the thickness of the pavement layers and the stiffness of the pavement using equation (20). For each test section, the Benkelman Beam deflection for surface, base, and subbase thicknesses were used in a multiple regression analysis to determine the values of constants, a_i. The following equations are a result of the regression analysis.

$$\log(d_s) = 3.125 - 0.070D_1 - 0.027D_2 - 0.024D_3 - 0.601\log(R) \tag{21}$$

$$\log(d_s) = 2.781 - 0.056D_1 - 0.016D_2 - 0.019D_3 - 0.507\log(R) \tag{22}$$

$$\log(d_s) = 2.733 - 0.056D_1 - 0.019D_2 - 0.025D_3 - 0.416\log(R) \tag{23}$$

where Log(d_s) = Benkelman beam deflection, 0.001 in. using a 9-ton axle load.

To determine the granular equivalency for the top surface, the surface constant, a_1, was divided by the granular base constant, a_2. The lower base equivalency was determined by dividing a_3 by a_2. For equation (15), the granular equivalency for the asphalt surface is determined by using the regression analysis constants, a_1 = 0.07 and a_2 = 0.027. Thus, the G.E. factor for asphalt is 2.59 (=0.07/0.027). The GE factor for Class 3 and 4 Base is 0.89 (=0.024/0.027). Using equations (21 - 23), the equivalency factors in Table 4 for the surface, base, and subbase were determined.

Table 4: GE Factors

	Equivalency Factors		
Year	**Surface**	**Base**	**Subbase**
1965	2.59	1	0.89
1966	3.5	1	1.2
1967	2.95	1	1.32

The Minnesota Department of Highways evaluated several GE factors in the Minnesota Investigation 183 for various pavement layers [23]:

- plant-mix surface layers: 2.25;
- plant-mix base: 2.0;
- road-mix surface: 1.5;
- road-mix base: 1.5;
- bituminous treated bases range for lean and rich mixes, respectively: 1.25-1.5;
- gravel bases: 0.9-1.0
- crushed rock base: 1.0
- sand gravel subbase: 0.75

2.3 Backcalculation and Falling Weight Deflectometer

Another method that can be used to estimate GE is backcalculation to obtain resilient modulus of the base materials. It is known that GE of Class 5 material is 1. Therefore, the ratio of resilient modulus between base materials and Cl5 should give an estimate of the GE factors of these materials. To measure pavement deflection, impulse load tests are commonly performed by using a Falling Weight Deflectometer (FWD). The FWD is a device capable of applying dynamic loads to the pavement surface, similar in magnitude and duration to that of a single heavy moving wheel load. The FWD test measures the pavement response with seismometers and generates a deflection basin that provides valuable information [24].

In an FWD test, an impulse load is applied by dropping a weight (usually 9000 lb) on the pavement, and the resulting deflections are measured at specified distances from the point of load application. The number of load applications can be adjusted. After obtaining the deflections, Young's moduli of the different layers are determined by backcalculation, which involves an estimate of the elastic properties from the measured surface deflections for an assumed layer profile [25]. In this study, EVERCALC software is used to backcalculate modulus of base materials.

Chapter 3. Selected Sections for Field Testing

FWD testing was performed on the selected sections (Figs. 2-5) over a three year period. During spring thaw of each year, FWD was conducted daily in the first week of thawing in an attempt to capture spring thaw weakening of base. After spring thaw, FWD was conducted monthly to study base recovery and stiffness changes through seasons. Table 5 contains the FWD testing schedule. Ground penetrating radar (GPR) was also conducted on the sections to obtain pavement thickness profile. GPR is a non-destructive testing tool that has wide applications in pavements. It detects changes in the underground profile due to contrasts in the electromagnetic conductivity across material interfaces. It can be used at relatively high speeds and gives a continuous pavement profile. GPR surveys have been successful in determining stripping zones in asphalt pavements, detecting subsurface voids, detecting subsurface anomalies (bedrock/peat), bridge deck delamination, tie bar locations, underground utility locates, sub-grade profiling, and pavement thickness. Figure 6 shows the GPR equipment.

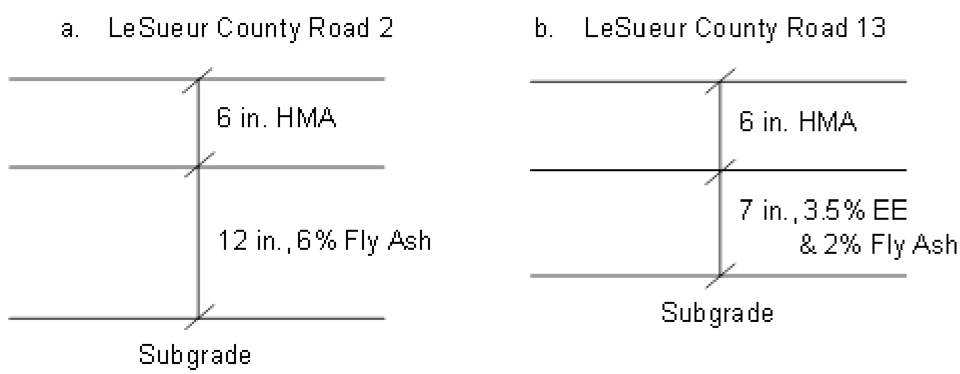

Figure 2: LeSueur County Road Sections: a. CR 2; b. Road 13

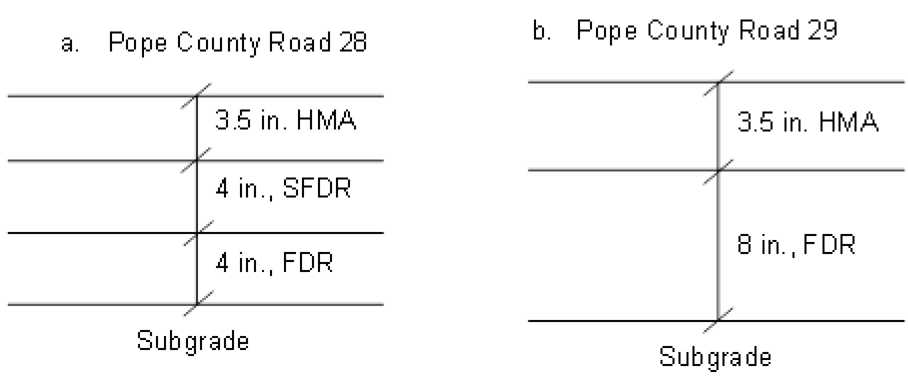

Figure 3: Pope County Road Sections: a. CR 28; b. CR 29

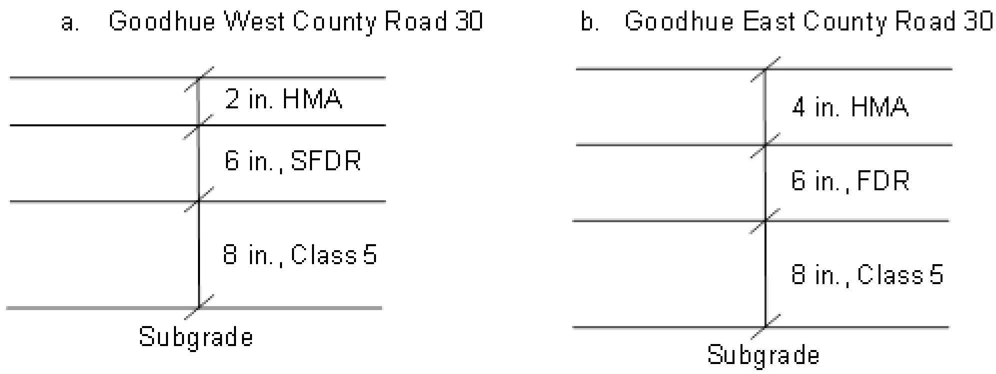

Figure 4: Goodhue East and West County Road 30 Sections: a. West; b. East

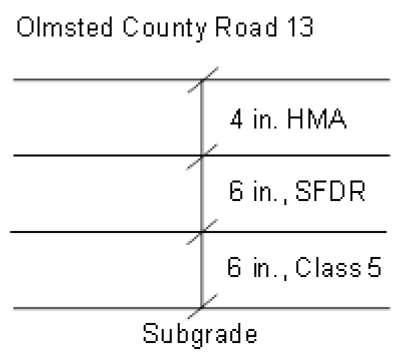

Figure 5: Olmsted County Road 13 Section

Figure 6: GPR Equipment

Table 5: FWD Testing Schedule

	Goodhue	Goodhue	LeSueur	LeSueur	Olmsted	Pope	Pope
	30E	30W	2	13	13	28	29
2009 FWD Testing	3/5/2009	3/5/2009	3/5/2009	3/5/2009		3/6/2009	3/6/2009
	3/9/2009	3/9/2009	3/9/2009	3/9/2009	3/9/2009	3/16/2009	3/16/2009
	3/10/2009	3/10/2009		3/10/2009	3/10/2009	3/17/2009	3/17/2009
	3/13/2009	3/13/2009	3/13/2009	3/13/2009	3/13/2009	3/19/2009	3/19/2009
	3/16/2009	3/16/2009	3/16/2009	3/16/2009	3/16/2009	3/20/2009	3/20/2009
	3/18/2009	3/18/2009	3/17/2009	3/17/2009	3/18/2009	3/23/2009	3/23/2009
	3/19/2009	3/19/2009	3/19/2009	3/19/2009	3/19/2009	3/24/2009	3/24/2009
	3/20/2009	3/20/2009	3/20/2009	3/20/2009	3/20/2009		
	3/23/2009	3/23/2009	3/23/2009	3/23/2009	3/23/2009		
	4/30/2009	4/30/2009	4/30/2009	4/30/2009	4/30/2009	4/27/2009	4/27/2009
	7/10/2009	7/10/2009	7/13/2009	7/13/2009	7/10/2009	7/30/2009	7/30/2009
	8/7/2009	8/7/2009					
	10/19/2009	10/19/2009	10/19/2009	10/19/2009	10/19/2009	10/18/2009	10/18/2009
	11/5/2009	11/5/2009	11/5/2009	11/5/2009	11/5/2009	11/9/2009	11/9/2009
2010 FWD Testing	3/4/2010	3/4/2010	3/4/2010	3/4/2010	3/4/2010	3/5/2010	3/5/2010
	3/8/2010	3/8/2010	3/8/2010	3/8/2010	3/8/2010	3/9/2010	3/9/2010
	3/9/2010	3/9/2010	3/9/2010	3/9/2010	3/9/2010	3/10/2010	3/10/2010
	3/10/2010	3/10/2010	3/10/2010	3/10/2010	3/10/2010	3/13/2010	3/13/2010
	3/11/2010	3/11/2010	3/11/2010	3/11/2010	3/11/2010	3/14/2010	3/14/2010
	3/12/2010	3/12/2010	3/12/2010	3/12/2010	3/12/2010	3/16/2010	3/16/2010
	3/18/2010	3/18/2010	3/18/2010	3/18/2010	3/18/2010	3/19/2010	3/19/2010
	3/25/2010	3/25/2010	3/25/2010	3/25/2010	3/25/2010	3/26/2010	3/26/2010
	3/31/2010	3/31/2010	3/31/2010	3/31/2010	3/31/2010	4/1/2010	4/1/2010
	4/7/2010	4/7/2010	4/7/2010	4/7/2010	4/7/2010	4/6/2010	4/6/2010
	4/14/2010	4/14/2010	4/14/2010	4/14/2010	4/14/2010	4/15/2010	4/15/2010
	5/24/2010	5/24/2010	5/24/2010	5/24/2010	5/24/2010	5/13/2010	5/13/2010
	6/16/2010	6/16/2010	6/16/2010	6/16/2010	6/16/2010	6/17/2010	6/17/2010
	7/14/2010	7/14/2010	7/14/2010	7/14/2010	7/15/2010	7/16/2010	7/16/2010
	8/12/2010	8/12/2010	8/12/2010	8/12/2010	8/12/2010	8/20/2010	8/20/2010
	9/22/2010	9/22/2010	9/22/2010	9/22/2010	9/22/2010	9/29/2010	9/29/2010
	10/12/2010	10/12/2010	10/12/2010	10/12/2010	10/12/2010	10/11/2010	10/11/2010
	11/4/2010	11/4/2010	11/4/2010	11/4/2010	11/4/2010	11/3/2010	11/3/2010
2011 FWD Testing	2/17/2011	3/8/2011	2/17/2011	2/17/2011	2/17/2011	3/4/2011	3/4/2011
	3/14/2011	3/14/2011	3/15/2011	3/15/2011	3/14/2011	3/17/2011	3/17/2011
	3/16/2011	3/16/2011	3/16/2011	3/16/2011	3/16/2011	3/18/2011	3/18/2011
	3/17/2011	3/17/2011	3/17/2011	3/17/2011	3/17/2011	3/19/2011	3/19/2011
	3/18/2011	3/18/2011	3/18/2011	3/18/2011	3/18/2011	3/21/2011	3/21/2011
	3/19/2011	3/19/2011	3/19/2011	3/19/2011	3/19/2011	3/22/2011	3/22/2011
	3/30/2011	3/30/2011	3/30/2011	3/30/2011	3/30/2011	3/28/2011	3/28/2011
	4/7/2011	4/7/2011	4/7/2011	4/7/2011	4/7/2011	4/4/2011	4/4/2011
	4/15/2011	4/15/2011	4/15/2011	4/15/2011	4/15/2011	4/13/2011	4/13/2011
	5/18/2011	5/18/2011	5/18/2011	5/18/2011	5/18/2011	4/18/2011	4/18/2011
	9/1/2011	9/1/2011	9/1/2011	9/1/2011	6/27/2011	5/12/2011	5/12/2011
	9/28/2011	9/28/2011	9/28/2011	9/28/2011	9/1/2011	8/30/2011	8/30/2011
	10/17/2011	10/17/2011	10/17/2011	10/17/2011	9/28/2011	9/26/2011	9/26/2011
					10/17/2011	10/14/2011	10/14/2011

FWD data were used to obtain base stiffness through backcalculation. Figure 7 shows examples of base stiffness change during the year. GPR data were analyzed to obtain pavement layer thickness profiles. The layer thickness is determined at every 0.1 miles (Figs. 8-13).

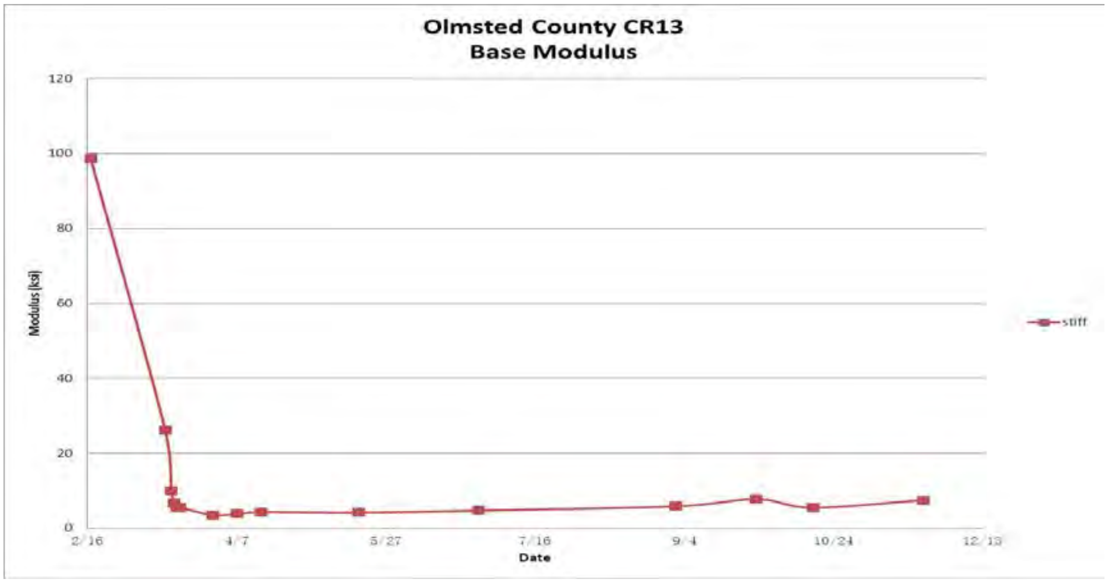

Figure 7: Example of Base Stiffness Changes during the Year

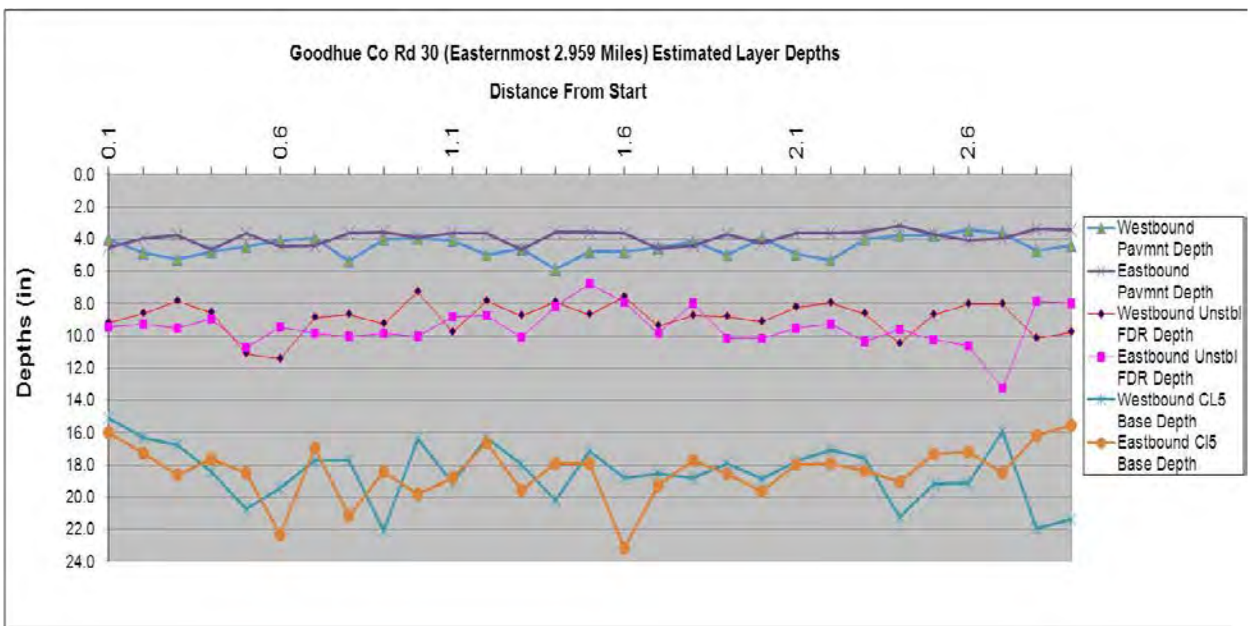

Figure 8: Goodhue County Road 30 Pavement Layer Thickness from GPR Survey

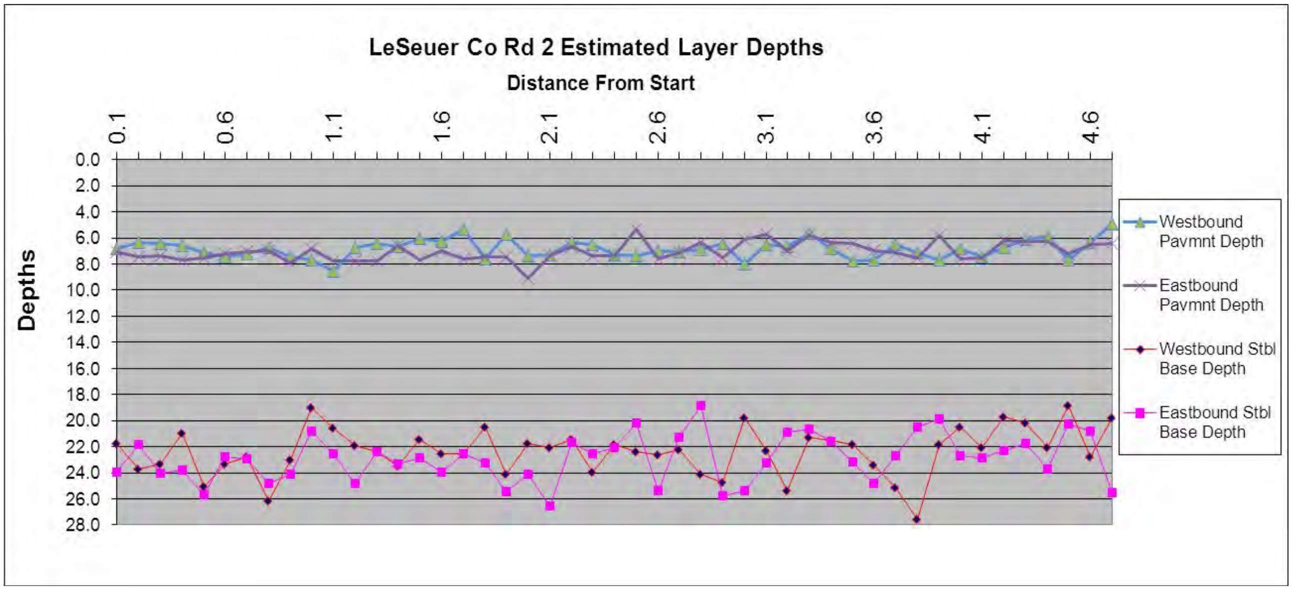

Figure 9: LeSeuer County Road 2 Pavement Layer Thickness from GPR Survey

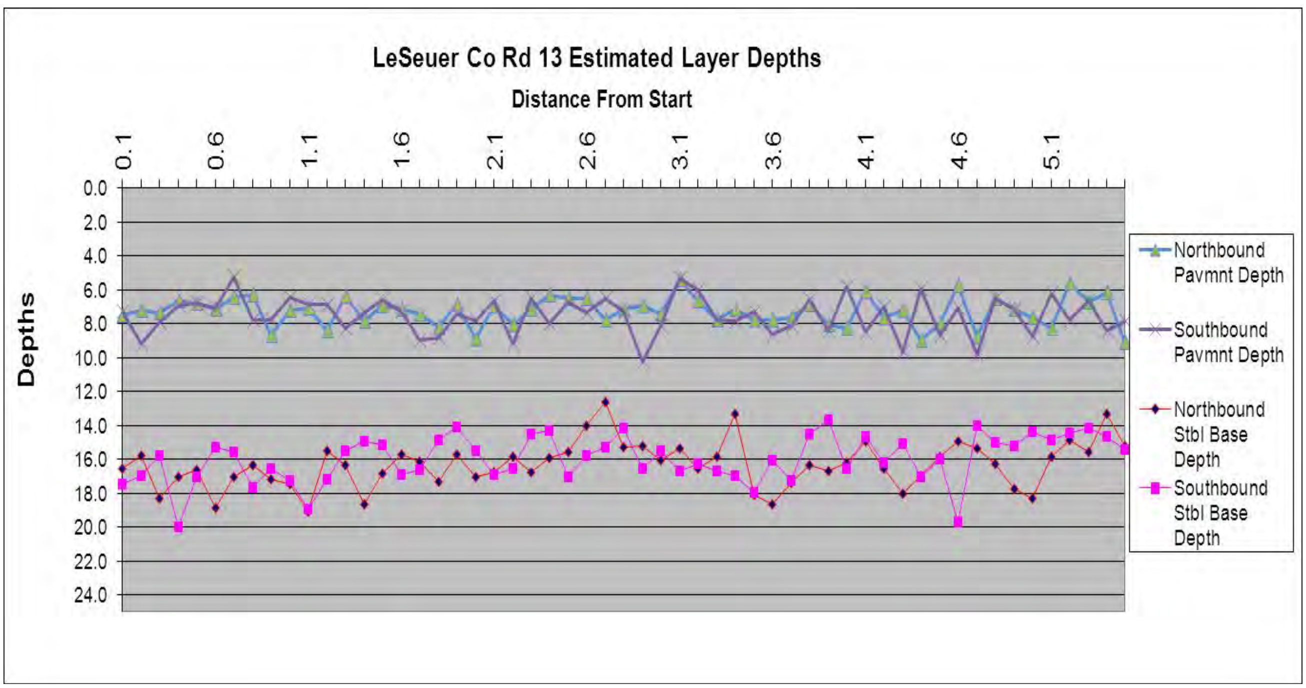

Figure 10: LeSeuer County Road 13 Pavement Layer Thickness from GPR Survey

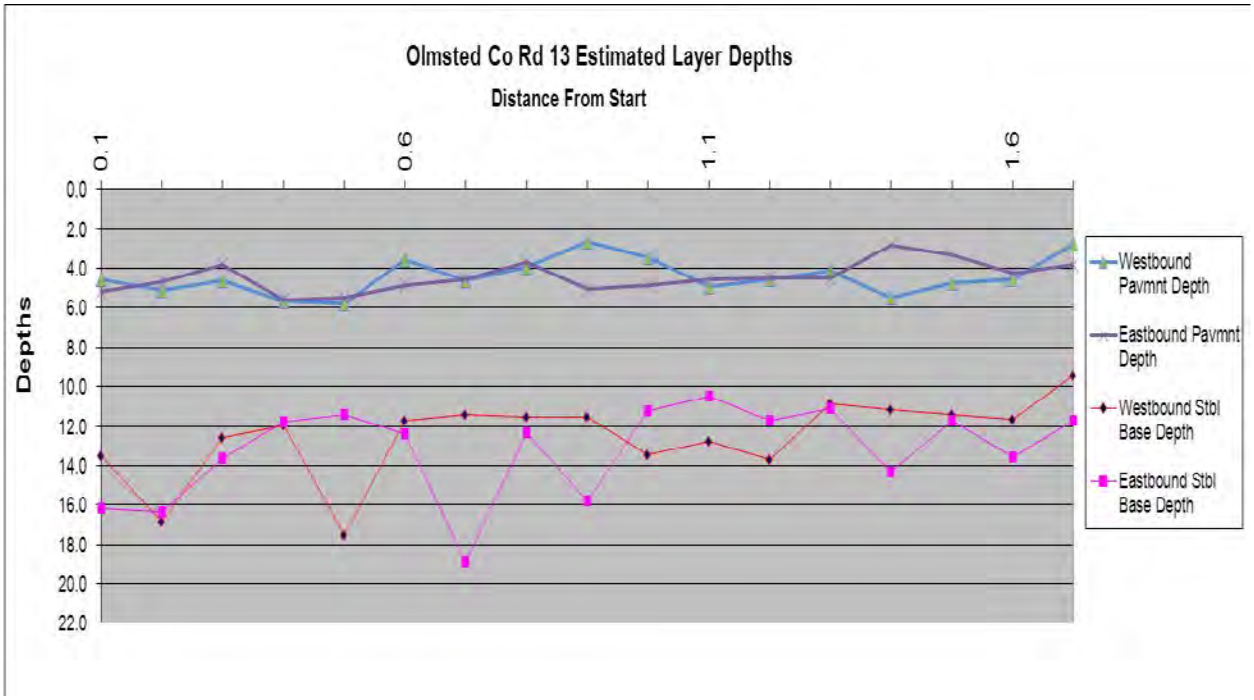

Figure 11: Olmsted County Road 13 Pavement Layer Thickness from GPR Survey

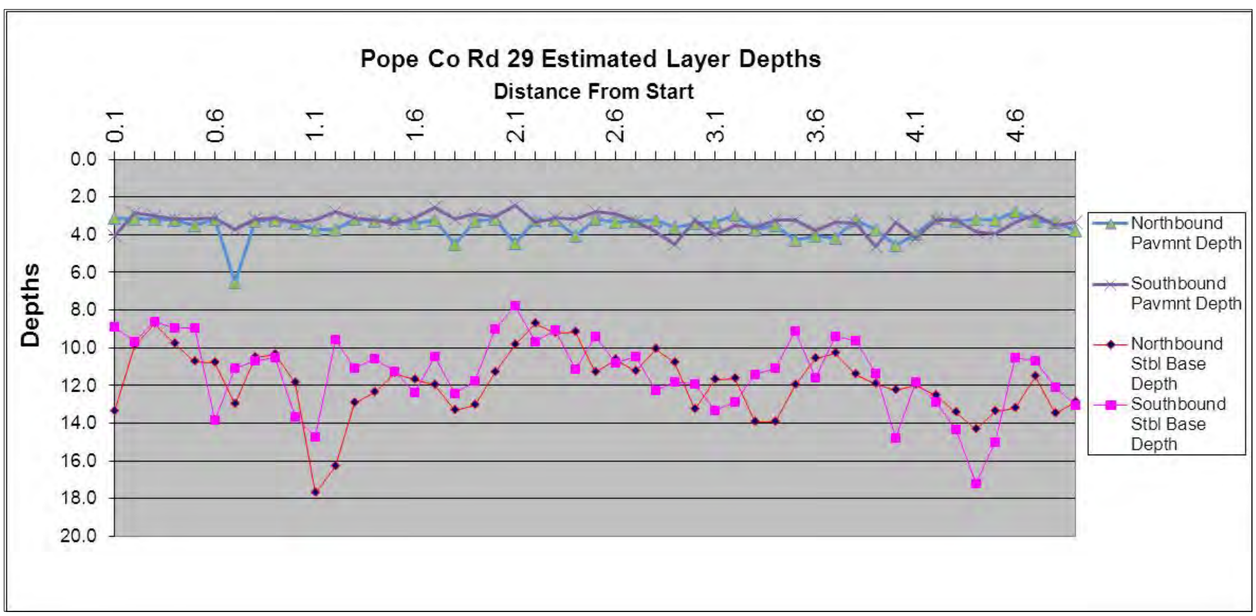

Figure 12: Pope County Road 29 Pavement Layer Thickness from GPR Survey

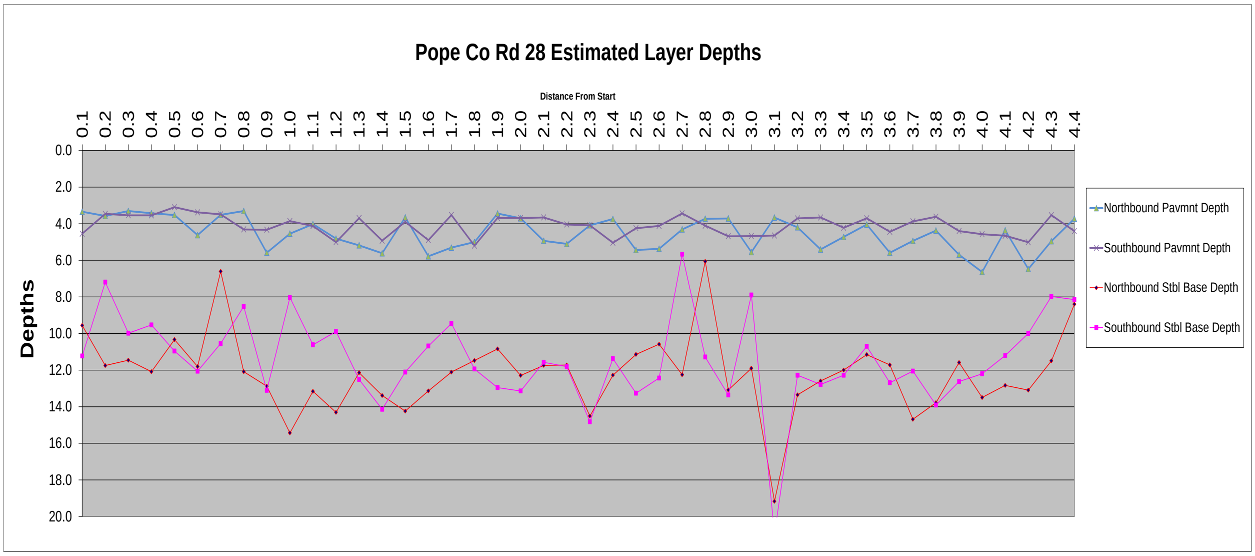

Figure 13: Pope County Road 28 Pavement Layer Thickness from GPR Survey

Chapter 4. Analysis Results

4.1 MnROAD Cell 21, Class 5 Base Analysis

In order to analyze stiffness values of the SFDR projects, moduli from Class 5 projects must be compared to determine if the SFDR roads yield higher performance.

The Young's modulus for the base course was determined using an FWD analysis program, EVERCALC, for one project using Class 5 as a base, and seven other projects using both stabilized and standard FDR base. Because the geology under the pavement subgrade was not specified for all projects, it is unknown whether a stiff layer (location of zero deflection) is present. In EVERCALC, the modulus values were computed twice, once with a stiff layer and once with no stiff layer (semi-infinite space). The modulus values with the lowest RMS (root mean square) error were used for the analysis.

MnROAD cell 21 consists of 8 in. (203 mm) of asphalt and 23 in. (584 mm) of Class 5 base. The stiffness values for the road section constructed with Class 5 base are detailed as follows: Unless noted otherwise, only one modulus value is shown per testing season. Spring testing was conducted from January to May. Summer testing was conducted from June to August. Fall testing was conducted from September to November. No testing was performed in December. Unless noted, each modulus is an average of several measurements and is measured in ksi (1000 lb/in.^{2}). MnROAD cell 21 consists of 8 in. (203 mm) of asphalt and 23 in. (584 mm) of Class 5 base.

Table 6: Young's Modulus Values in ksi (1000 lb/in.^{2}) for MnROAD Cell 21

Cell 21				
		Spring	**Summer**	**Fall**
HMA	nostiff	1008.15	311.26	1520.56
	stiff	907.65	278.34	1423.04
BASE	nostiff	14.76	20.36	18.39
	stiff	24.00	26.23	28.29

4.2 LeSueur County Road 2

The 4.8 mile section of LeSueur County Road (CR) 2 is constructed of 6 in. (152 mm) asphalt followed by 12 in. (305 mm) of 6% Class C fly ash SFDR base. The subgrade material is classified as "plastic."

Table 7: Young's Modulus Values in ksi (1000 lb/in.^{2}) for LeSueur CR 2

		Spring		Summer		Fall	
2011			**w/r Cell21**		**w/r Cell21**		**w/r Cell21**
HMA	nostiff	942.27	0.93	536.03	1.72	1152.79	0.76
	stiff	822.71	0.91	369.82	1.33	917.89	0.65
BASE	nostiff	23.78	1.61	46.57	2.29	37.88	2.06
	stiff	51.16	2.13	73.61	2.81	62.91	2.22
2010							
HMA	nostiff	1246.89	1.24	291.12	0.94	1003.56	0.66
	stiff	1106.47	1.22	196.54	0.71	788.66	0.55
BASE	nostiff	42.09	2.85	48.19	2.37	49.03	2.67
	stiff	59.26	2.47	75.5	2.88	73.19	2.59
2009							
HMA	nostiff	835.89	0.83	313.02	1.01	1094.04	0.72
	stiff	752.06	0.83	215.85	0.78	880.48	0.62
BASE	nostiff	41.17	2.79	39.89	1.96	46.39	2.52
	stiff	71.57	2.98	62.76	2.39	69.05	2.44

The white columns represent the ratio of the modulus of the FDR base over the modulus of the Cell 21 Class 5 base. The Young's modulus values were calculated for each individual pavement layer for SFDR roads and the Class 5 base roads in spring, summer, and fall. The moduli were backcalculated using EVERCALC, both with a stiff layer and without a stiff layer. A ratio greater than 1.0 designates that the FDR section is stiffer. Observing the ratios between LeSueur CR 2 and Class 5 roads, it can be seen that, for the most part, the base modulus values calculated for LeSueur CR 2 are generally higher than those of Class 5 base roads.

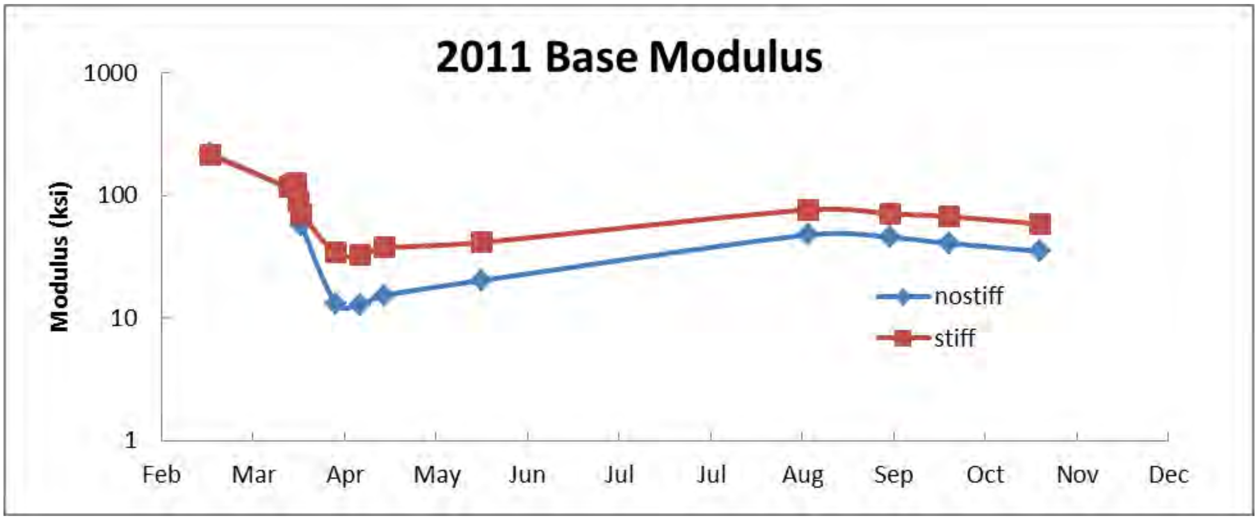

Figure 14: Young's Modulus Values for LeSueur CR2 in 2011

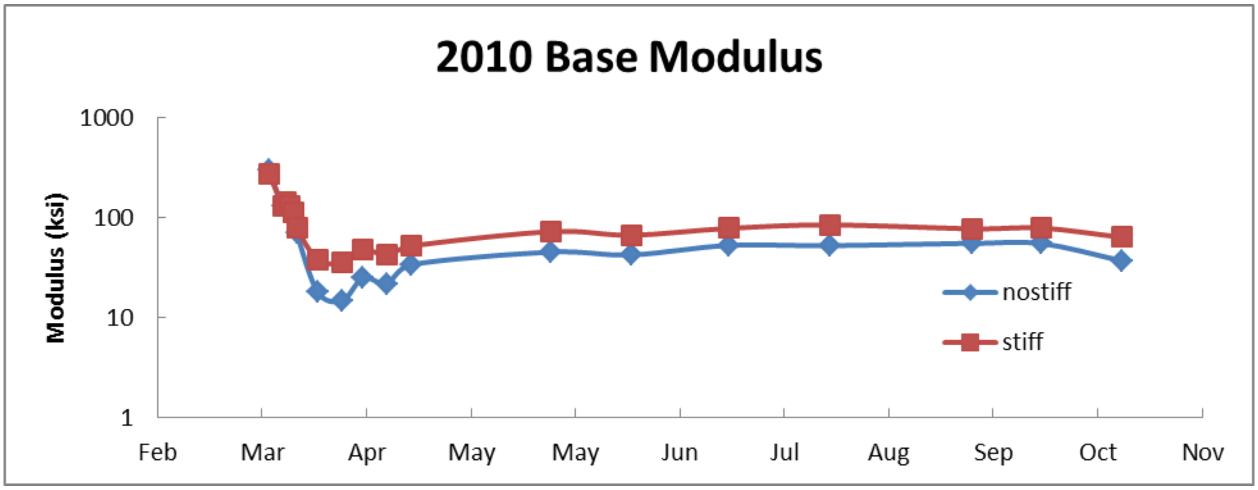

Figure 15: Young's Modulus values for LeSueur CR2 in 2010

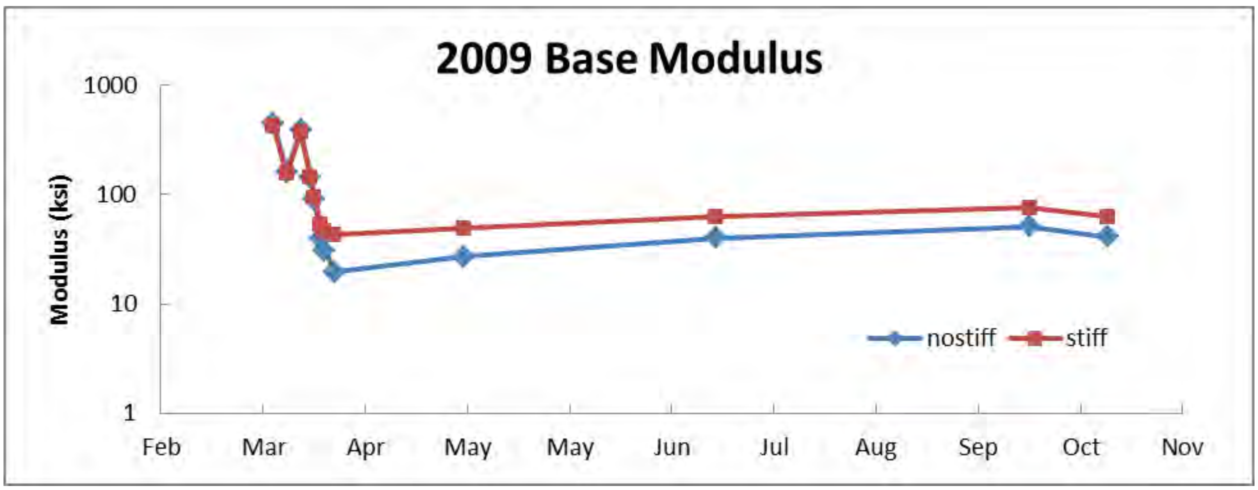

Figure 16: Young's Modulus values for LeSueur CR2 in 2009

It can be observed from Figures 14-16 that when the modulus is calculated with a stiff layer, the modulus values are consecutively higher throughout the year compared to when calculated without a stiff layer. In the spring months there is a rapid decrease in both sets of moduli due to the spring thawing effect on the roads.

4.3 LeSueur County Road 13

The 5.5 mile section of CR13 in LeSueur County is constructed of 6 in. (152 mm) of asphalt course followed by 7 in. (178 mm) of 3.5% emulsion and 2% Class C fly ash SFDR base, and 1-3 in. (25-76 mm) of non-stabilized FDR base; the stabilized and non-stabilized were treated as one layer in Evercalc. The subgrade material is classified as "plastic."

Table 8: Young's Modulus Values in ksi (1000 lb/in.^{2}) for LeSueur CR 13

		Spring		Summer		Fall	
2011			w/r Cell21		w/r Cell21		w/r Cell21
HMA	nostiff	1311.82	1.30	250.60	0.81	1028.73	0.68
	stiff	1385.39	1.53	297.61	1.07	1130.52	0.79
BASE	nostiff	155.45	10.53	135.90	6.67	230.17	12.51
	stiff	144.86	6.04	90.50	3.45	193.15	6.83
2010							
HMA	nostiff	1814.90	1.80	173.52	0.56	776.37	0.51
	stiff	1888.96	2.08	226.88	0.82	865.67	0.61
BASE	nostiff	154.37	10.46	165.18	8.11	252.49	13.73
	stiff	139.26	5.80	108.35	4.13	221.83	7.84
2009							
HMA	nostiff	1100.75	1.09	160.01	0.51	1253.12	0.82
	stiff	1218.97	1.34	217.13	0.78	1380.82	0.97
BASE	nostiff	69.29	4.69	93.83	4.61	171.09	9.30
	stiff	42.82	1.78	41.07	1.57	119.20	4.21

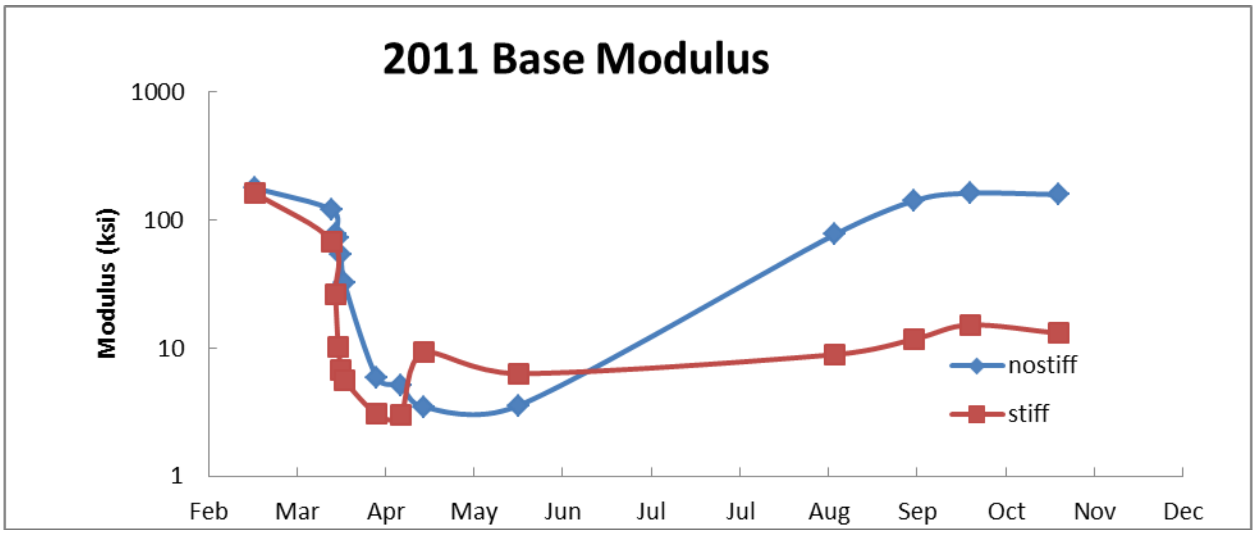

Figure 17: Young's Modulus Values for LeSueur CR 13 in 2011

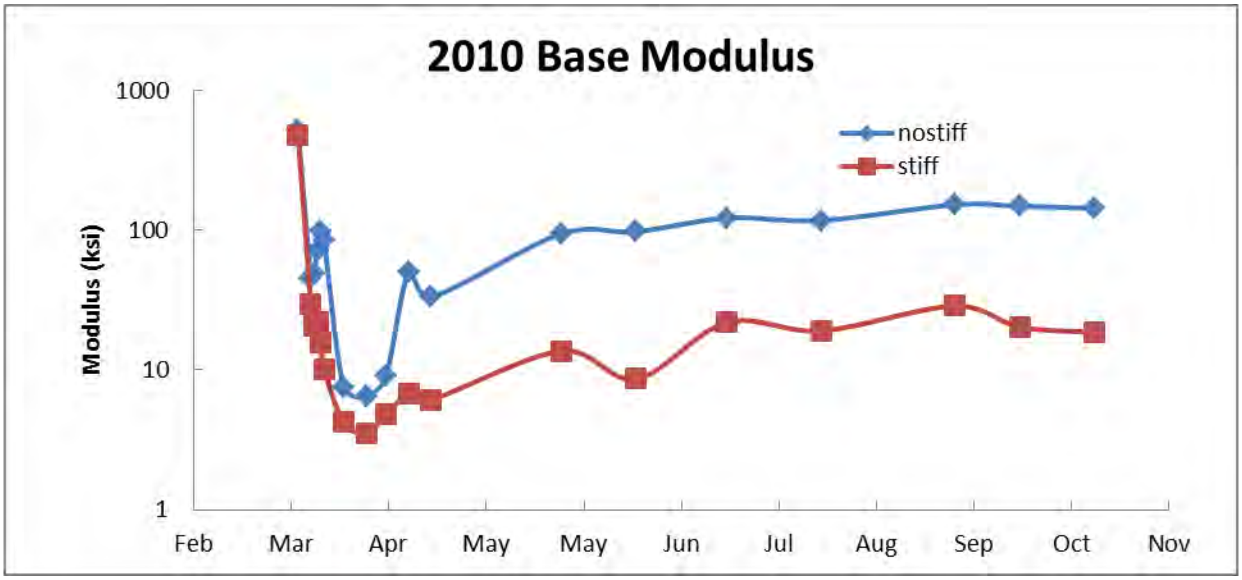

Figure 18: Young's Modulus Values for LeSueur CR 13 in 2010

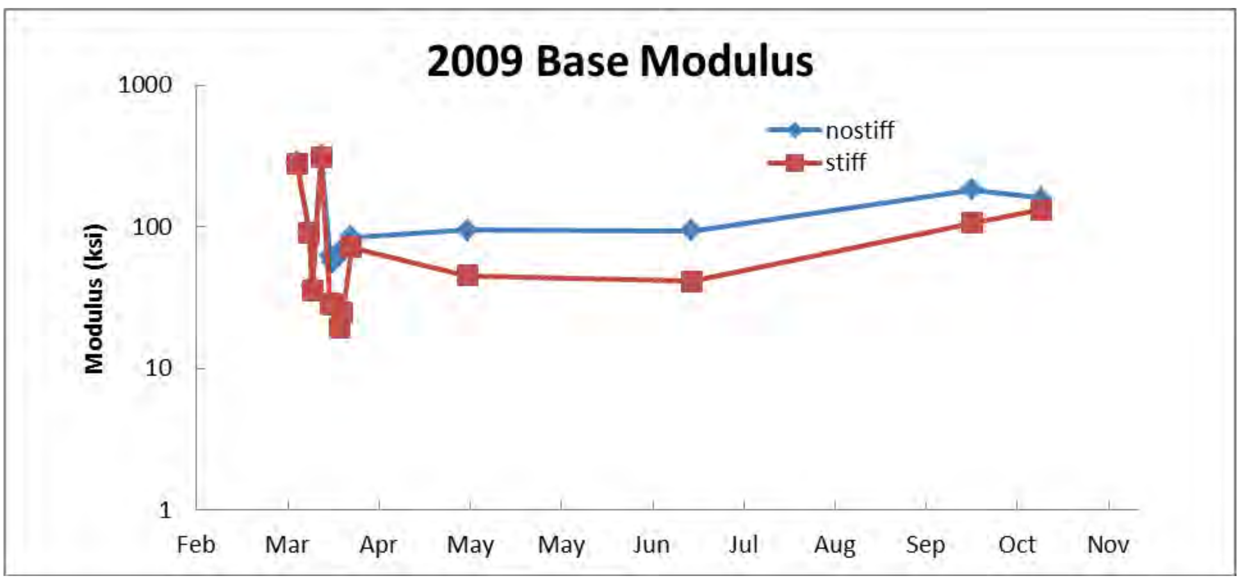

Figure 19: Young's Modulus Values for LeSueur CR 13 in 2009

Looking at the ratios between LeSueur CR 13 and Class 5 roads, it is clear that the base modulus for CR 13 is much higher than the values calculated for Class 5 base roads (Table 6). The moduli calculated without a stiff layer are generally higher throughout the year on CR 13. A spring thawing effect can be observed as well, which decreased the stiffness in late spring.

4.4 Pope County Road 28

The 4.2 mile section of CR 28 is constructed of 3.5 in. (89 mm) of asphalt followed by 4 in. (102 mm) of 0.004 gal/yd^2/in., T15 Base One SFDR, followed by 4 in. (102 mm) of FDR base course. The subgrade soil is classified as Class 4 material.

Table 9: Young's Modulus Values in ksi (1000 lb/in.^{2}) for CR 28

		Spring		Summer		Fall	
2011			**w/r Cell21**		**w/r Cell21**		**w/r Cell21**
HMA	nostiff	1674.06	1.66	403.28	1.30	1022.71	0.67
	stiff	2102.68	2.32	753.63	2.71	1524.97	1.07
BASE	nostiff	51.32	3.48	125.38	6.16	161.51	8.78
	stiff	12.23	0.51	31.26	1.19	53.53	1.89
2010							
HMA	nostiff	2062.22	2.05	667.97	2.15	658.69	0.43
	stiff	1963.20	2.16	699.35	2.51	1017.78	0.72
BASE	nostiff	11.12	0.75	76.97	3.78	134.63	7.32
	stiff	11.64	0.49	28.28	1.08	45.75	1.62
2009							
HMA	nostiff	1457.01	1.45	369.92	1.19	624.79	0.41
	stiff	1641.56	1.81	664.67	2.39	1071.24	0.75
BASE	nostiff	61.34	4.16	167.50	8.23	162.56	8.84
	stiff	15.03	0.63	39.31	1.50	39.54	1.40

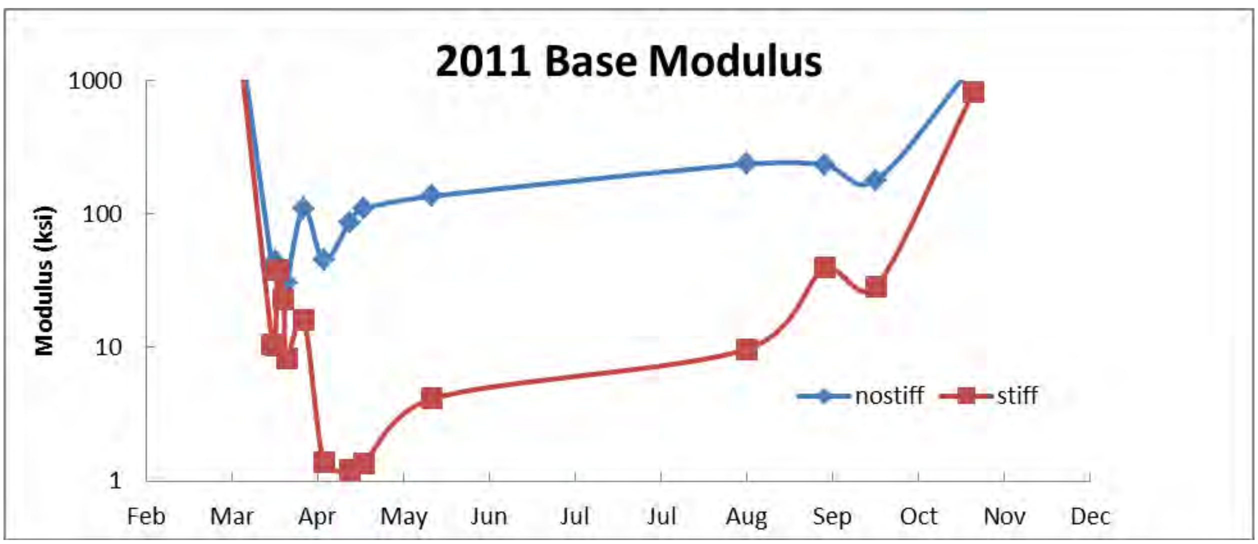

Figure 20: Young's Modulus Values for Pope CR28 in 2011

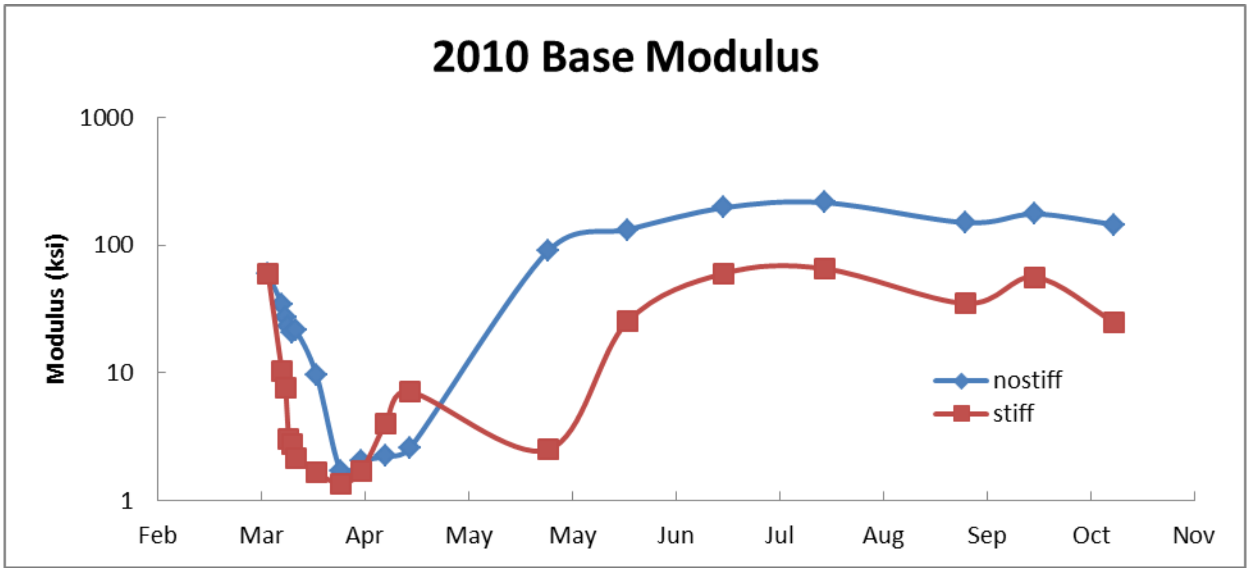

Figure 21: Young's Modulus Values for Pope CR28 in 2010

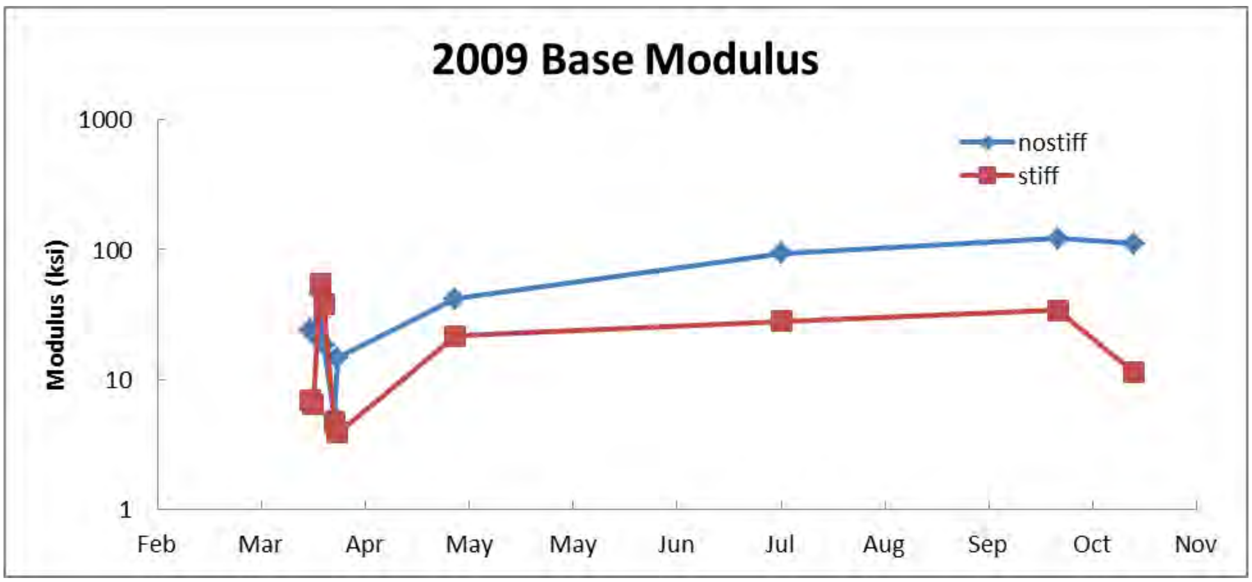

Figure 22: Young's Modulus Values for Pope CR 28 in 2009

Looking at the ratios for CR 28, it can be observed that the base modulus values computed for CR 28 are higher compared to the values computed for Class 5 roads (Table 6). Although there are some minor variations in which set of data shows a higher modulus, those calculated without a stiff layer are generally higher in the three years of testing. It can also be seen that in 2011 and 2010 there were several short periods of thawing and then refreezing again in the spring months that caused fluctuations in stiffness. Furthermore, in 2011 and 2010, there appears to have been a short freezing period in September that increased the stiffness as well.

4.5 Pope County Road 29

The 5.0 mile section of CSAH 29 is constructed of 3.5 (89 mm) in. of asphalt followed by 8 in. (204 mm) of FDR base course. The subgrade soil is classified as Class 4 material.

Table 10: Young's Modulus Values in ksi (1000 lb/in.^{2}) for Pope CR29

		Spring		Summer		Fall	
2011			**w/r Cell21**		**w/r Cell21**		**w/r Cell21**
HMA	nostiff	2293.59	2.28	1784.39	5.73	1311.76	0.86
	stiff	2488.87	2.74	988.92	3.55	3319.05	2.33
BASE	nostiff	15.22	1.03	36.24	1.78	33.53	1.82
	stiff	15.32	0.64	48.53	1.85	53.38	1.89
2010							
HMA	nostiff	2355.84	2.34	650.83	2.09	1707.17	1.12
	stiff	2144.76	2.36	398.51	1.43	1254.01	0.88
BASE	nostiff	12.44	0.84	29.05	1.43	25.86	1.41
	stiff	16.86	0.70	43.80	1.67	38.97	1.38
2009							
HMA	nostiff	2345.98	2.33	945.60	3.04	2016.34	1.33
	stiff	1339.11	1.48	333.61	1.20	940.41	0.66
BASE	nostiff	12.75	0.86	23.21	1.14	20.78	1.13
	stiff	44.18	1.84	54.52	2.08	53.05	1.88

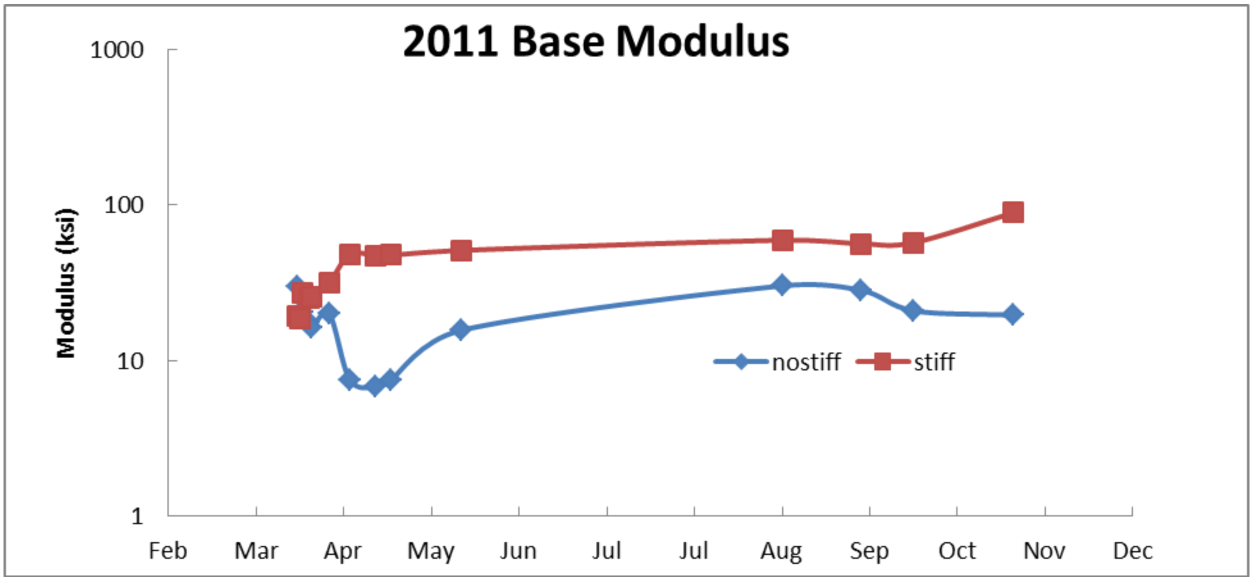

Figure 23: Young's Modulus Values for Pope CR 29 in 2011

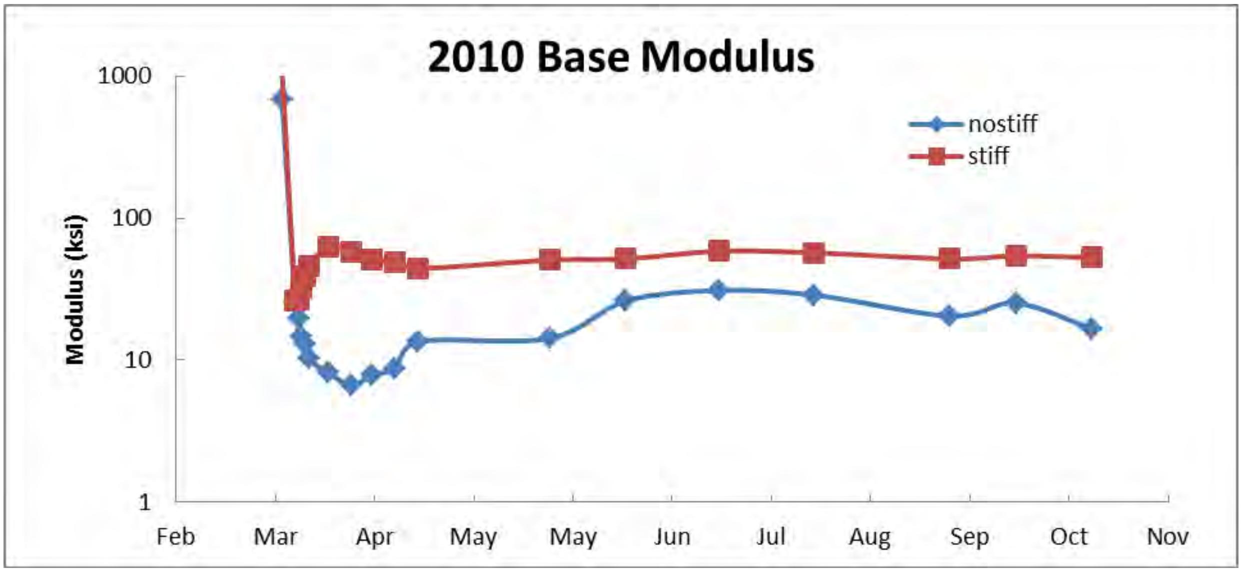

Figure 24: Young's Modulus Values for Pope CR 29 in 2010

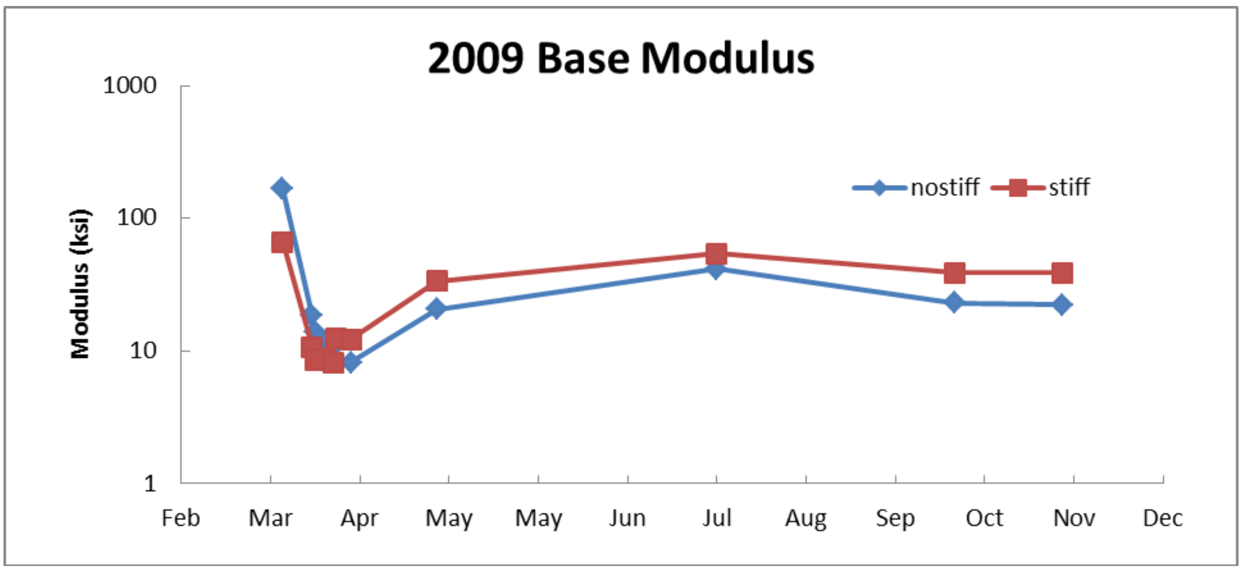

Figure 25: Young's Modulus Values for Pope CR 29 in 2009

Observing the ratios between CR 29 and the Class 5 roads, it can be seen that the calculated base moduli are higher for CR 29 than those calculated for Class 5 roads. The moduli calculated with a stiff layer are consecutively higher than those without a stiff layer. A spring thawing effect can only be observed in 2010 and 2009, while in 2011 the road section has higher stiffness in the summer compared to spring, and an even greater stiffness in the fall.

4.6 Goodhue County Road 30 Eastern Section

About 3 miles of CR 30 East is constructed of 4 in. (102 mm) of asphalt followed by 6 in. (152 mm) of FDR, and 8 in. (203 mm) of aggregate. The subgrade soil was not classified.

Table 11: Young's Modulus Values in ksi (1000 lb/in.^{2}) for CR 30 East

		Spring		Summer		Fall	
2011			**w/r Cell21**		**w/r Cell21**		**w/r Cell21**
HMA	nostiff	1808.91	1.79	550.45	1.77	1327.37	0.87
	stiff	1867.66	2.06	699.07	2.51	1521.12	1.07
BASE	nostiff	14.58	0.99	58.26	2.86	53.94	2.93
	stiff	9.11	0.38	38.03	1.45	28.64	1.01
2010							
HMA	nostiff	1831.94	1.82	383.83	1.23	1212.20	0.80
	stiff	1915.57	2.11	479.69	1.72	1355.86	0.95
BASE	nostiff	16.54	1.12	46.11	2.26	59.73	3.25
	stiff	7.97	0.33	29.00	1.11	38.68	1.37
2009							
HMA	nostiff	1875.05	1.86	338.49	1.09	1811.55	1.19
	stiff	1777.05	1.96	371.27	1.33	1882.70	1.32
BASE	nostiff	17.94	1.22	38.33	1.88	22.08	1.20
	stiff	19.85	0.83	30.40	1.16	13.93	0.49

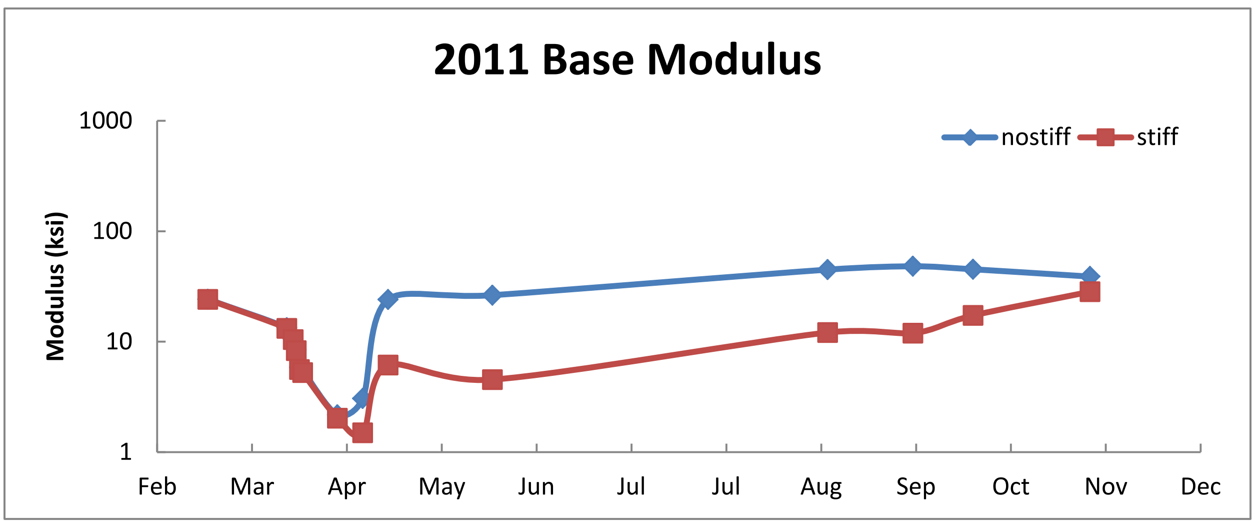

Figure 26: Young's Modulus Values for Goodhue CR 30 East in 2011

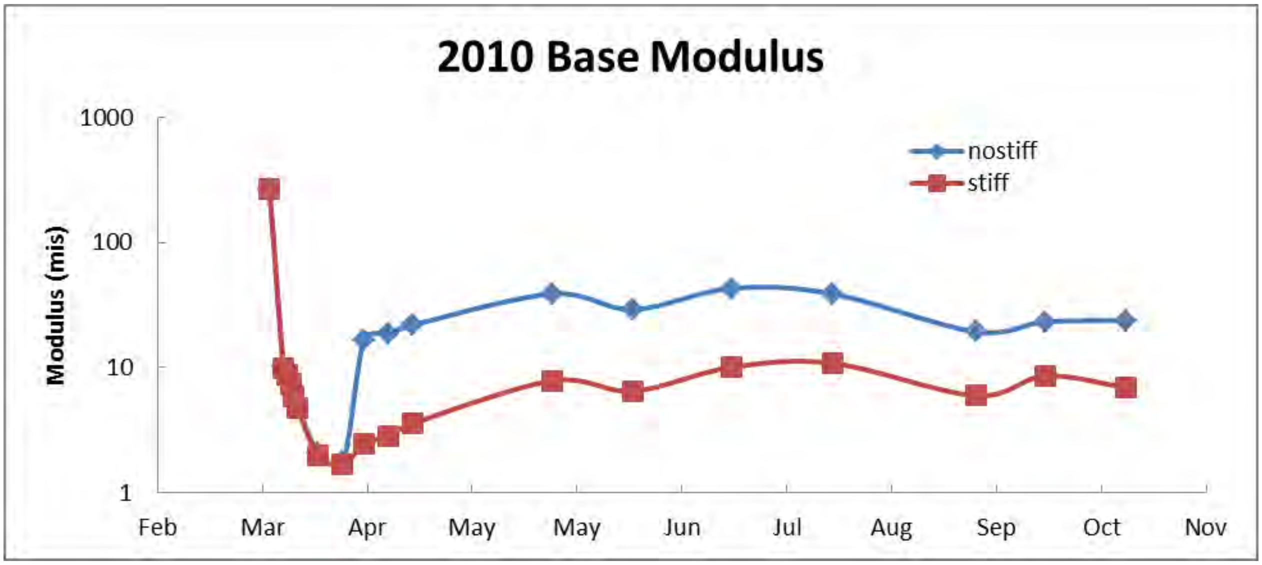

Figure 27: Young's Modulus Values for Goodhue CR 30 East in 2010

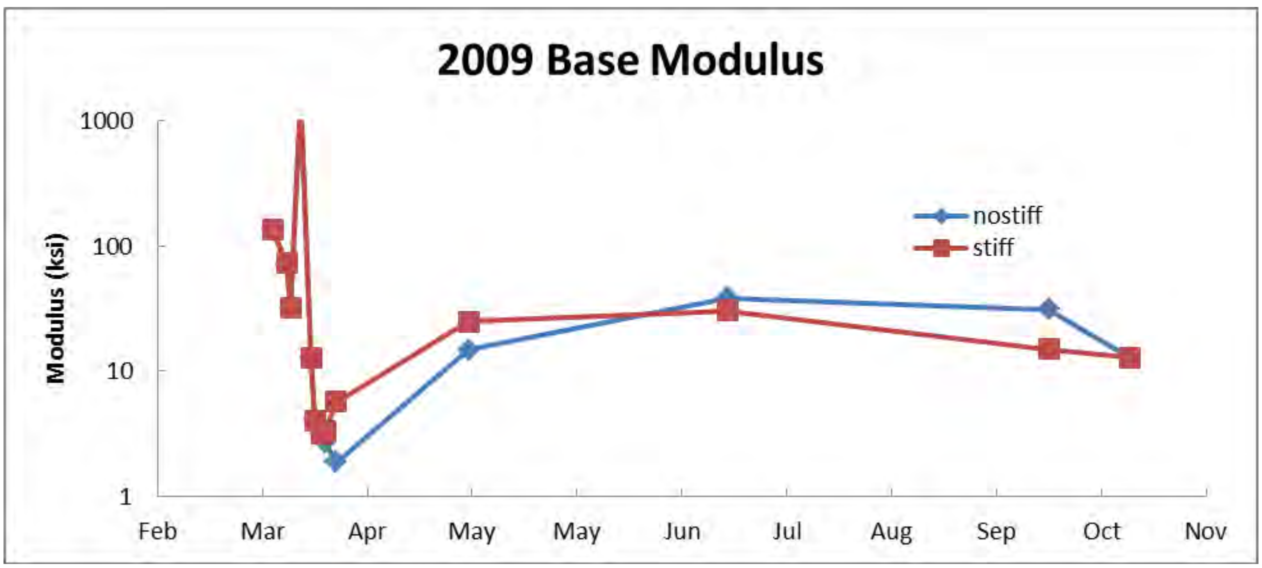

Figure 28: Young's Modulus Values for Goodhue CR 30 East in 2009

Observing the ratios between Goodhue CR 30 East and Class 5 roads, it can be seen that the base moduli calculated for CR 30 are, for the most part, higher than those values calculated for Class 5 roads. In 2011 and 2010, it can be clearly seen that the moduli calculated without a stiff layer are higher than those calculated with a stiff layer. However, in 2009, the results change halfway through the year. In spring and early summer, the values calculated with a stiff layer are higher, while in late summer and fall, the opposite is true. A spring thawing effect can be observed each year, decreasing the stiffness. In 2009, there was a late spring freeze that caused the stiffness to rise once again before the summer months.

4.7 Goodhue County Road 30 Western Section

The 2.8 mile section of CR 30 West is constructed of 2 in. (52 mm) of asphalt followed by 6 in. (152 mm) of 4.5% Fortress SFDR, and 8 in. (203 mm) of aggregate. The subgrade soil was not classified.

Table 12: Young's Modulus Values for Goodhue CR 30 West

		Spring		Summer		Fall	
2011			w/r Cell21		w/r Cell21		w/r Cell21
HMA	nostiff	2597.23	2.58	850.32	2.73	2181.99	1.43
	stiff	4686.98	5.16	2335.22	8.39	4089.24	2.87
BASE	nostiff	38.99	2.64	73.17	3.59	134.58	7.32
	stiff	13.97	0.58	47.28	1.80	89.56	3.17
2010							
HMA	nostiff	4108.65	4.08	616.45	1.98	1772.56	1.17
	stiff	5357.84	5.90	1607.88	5.78	3138.07	2.21
BASE	nostiff	56.68	3.84	72.82	3.58	123.48	6.71
	stiff	27.74	1.16	34.53	1.32	76.93	2.72
2009							
HMA	nostiff	4767.63	4.73	530.81	1.71	2179.49	1.43
	stiff	6037.35	6.65	781.27	2.81	3357.11	2.36
BASE	nostiff	41.93	2.84	63.93	3.14	141.53	7.69
	stiff	29.02	1.21	52.30	1.99	99.31	3.51

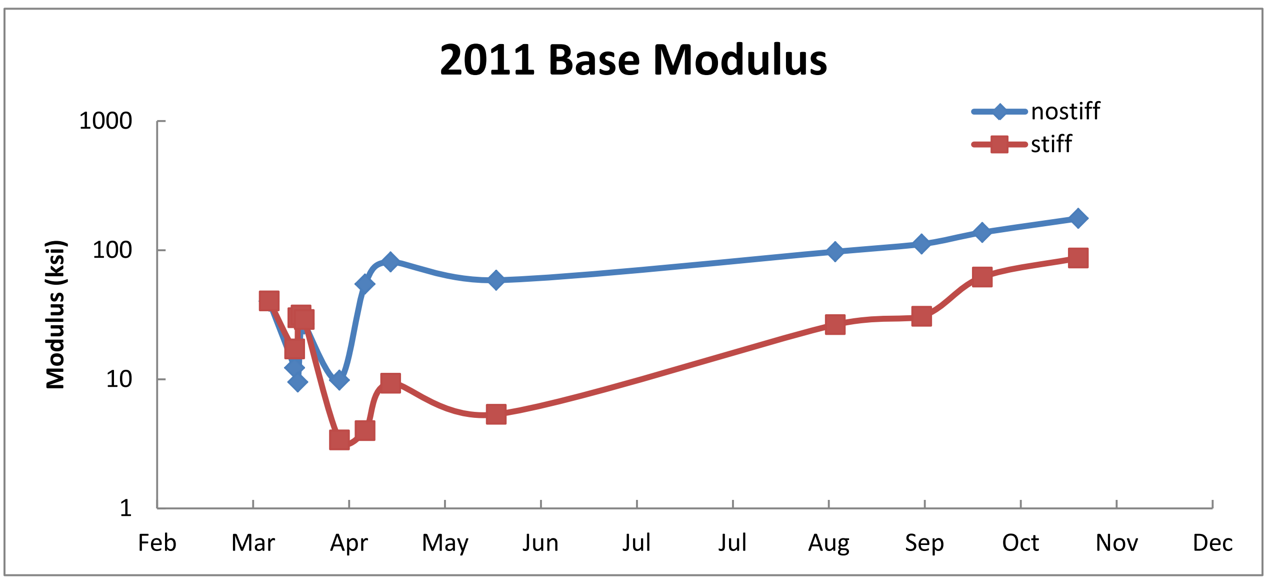

Figure 29: Young's Modulus Values for Goodhue CR 30 West in 2011

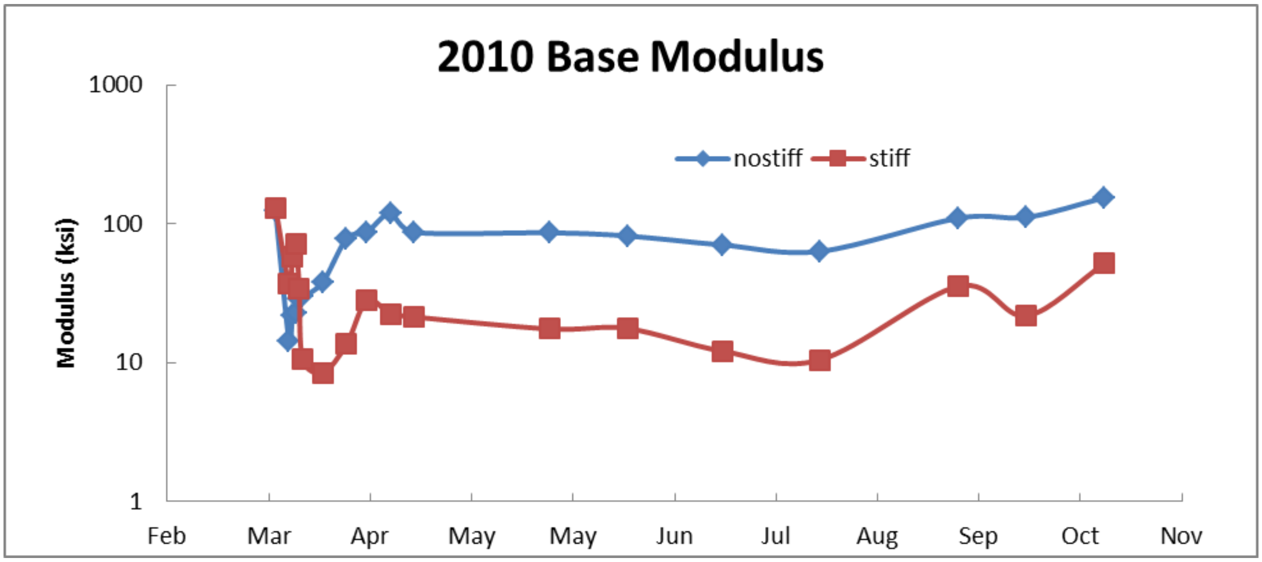

Figure 30: Young's Modulus Values for Goodhue CR30 West in 2010

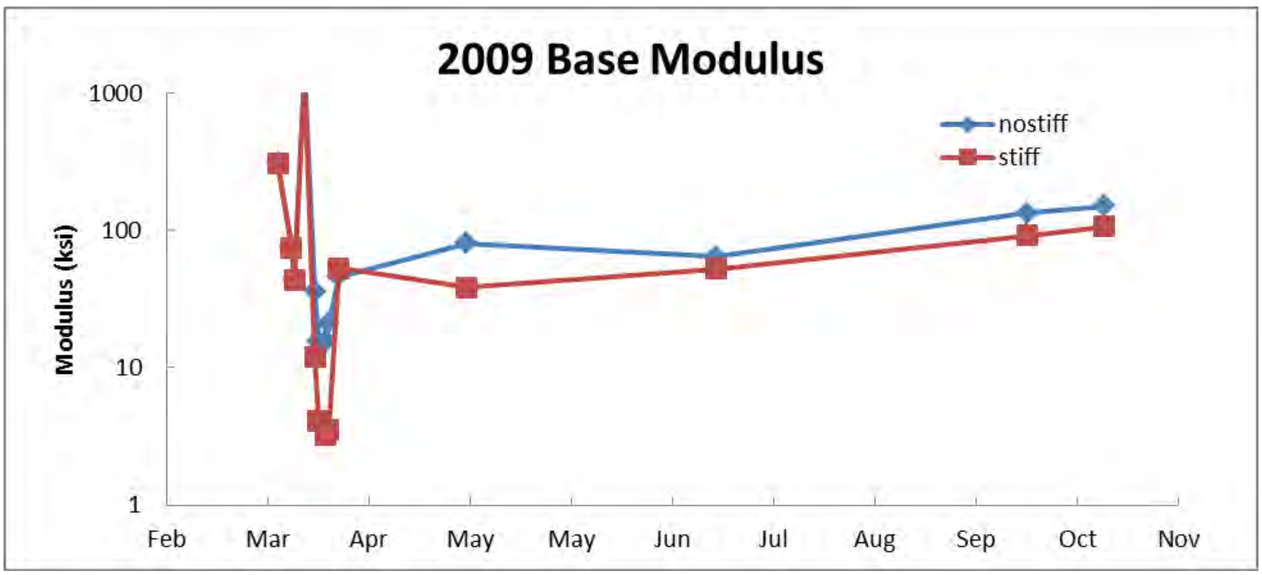

Figure 31: Young's Modulus Values for Goodhue CR 30 West in 2009

Observing the ratios between Goodhue CR 30 West and Class 5 roads, it can be seen that the base moduli for CR 30 West are higher than those of the Class 5 roads. It can be observed that the moduli calculated without a stiff layer are higher than those calculated with one during the late spring, summer, and fall months. However, there are some discrepancies as to which one is higher in the early spring months. The spring thawing effect causes several spikes all three years in both sets of data, which makes it difficult to reach a firm conclusion.

4.8 Olmsted County Road 13

When comparing the ratios between Olmsted CR 13 and Class 5 roads, it can be seen that the base moduli for CR 13 are much higher than those of the Class 5 roads.

Table 13: Young's Modulus Values in ksi (1000 lb/in.^{2}) for Olmsted County Road 13

		Spring		Summer		Fall	
2011			w/r Cell21		w/r Cell21		w/r Cell21
HMA	nostiff	2902.63	2.88	563.18	1.81	2058.16	1.35
	stiff	3038.08	3.35	642.29	2.31	2121.97	1.49
BASE	nostiff	188.14	12.74	274.23	13.47	373.84	20.32
	stiff	174.74	7.28	219.61	8.37	365.43	12.92
2010							
HMA	nostiff	3636.55	3.61	594.69	1.91	1740.05	1.14
	stiff	3783.48	4.17	799.37	2.87	2510.08	1.76
BASE	nostiff	138.77	9.40	182.66	8.97	228.95	12.45
	stiff	120.08	5.00	134.32	5.12	123.30	4.36
2009							
HMA	nostiff	3669.15	3.64	583.04	1.87	2480.74	1.63
	stiff	583.04	0.64	675.53	2.43	2743.04	1.93
BASE	nostiff	118.09	8.00	182.33	8.96	228.95	12.45
	stiff	66.91	2.79	81.42	3.10	123.30	4.36

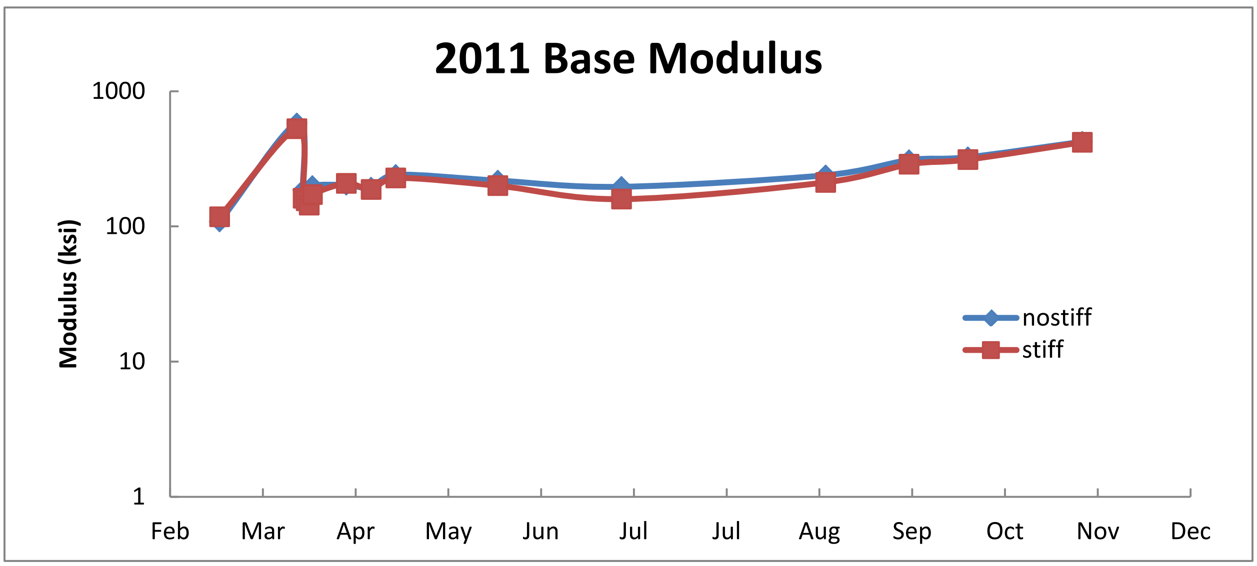

Figure 32: Young's Modulus Values for Olmsted CR13 in 2011

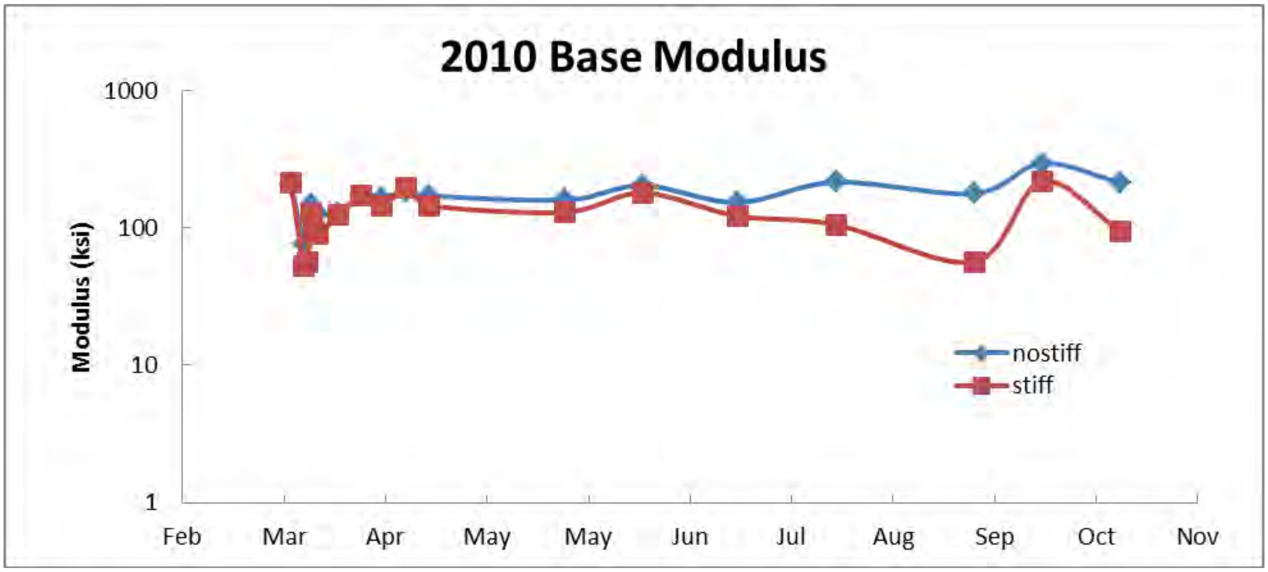

Figure 33: Young's Modulus Values for Olmsted CR13 in 2010

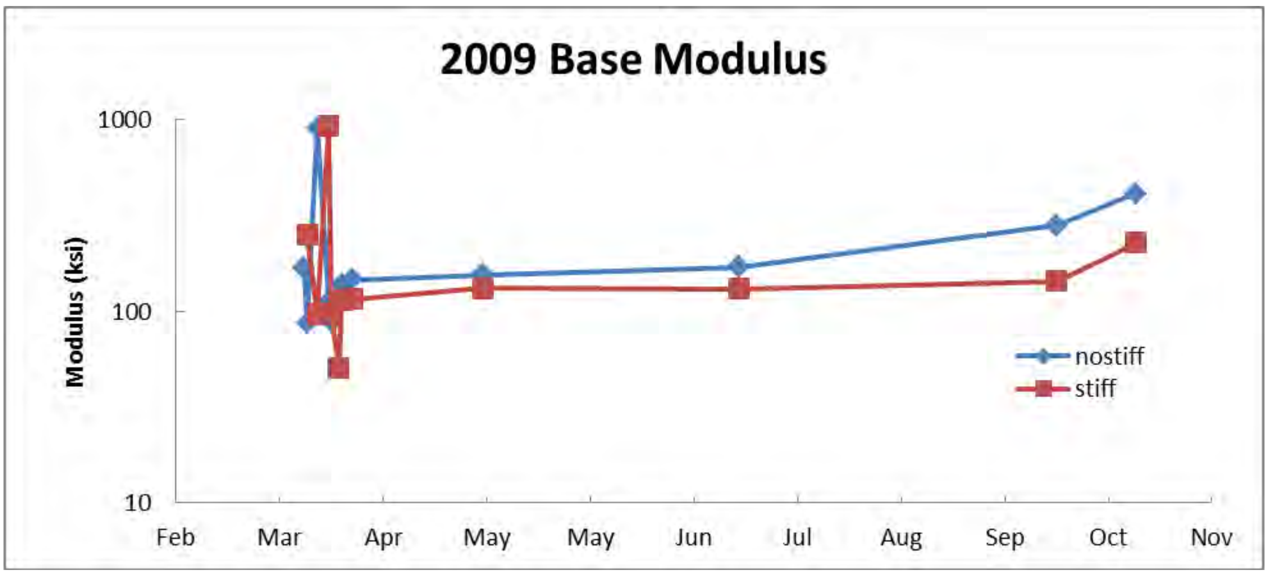

Figure 34: Young's Modulus Values for Olmsted CR 13 in 2009

For Olmsted CR 13, it can be seen that the moduli calculated without using a stiff layer are only slightly higher than those calculated with. However, the values have very small differences from one another- they are nearly identical across the span of all 3 years. The spring thawing effect also caused some large fluctuations in the data pattern in 2009. There appear to be multiple periods of freezing and thawing that year. On the other hand, there was hardly any variation between spring and summer values in 2011 and 2010.

4.9 Modulus Values Summary

The backcalculated modulus values indicate that these materials also have seasonal effects. Figures 35-37 show the comparison of backcalculated moduli of all base materials for 2009, 2010 and 2011, respectively.

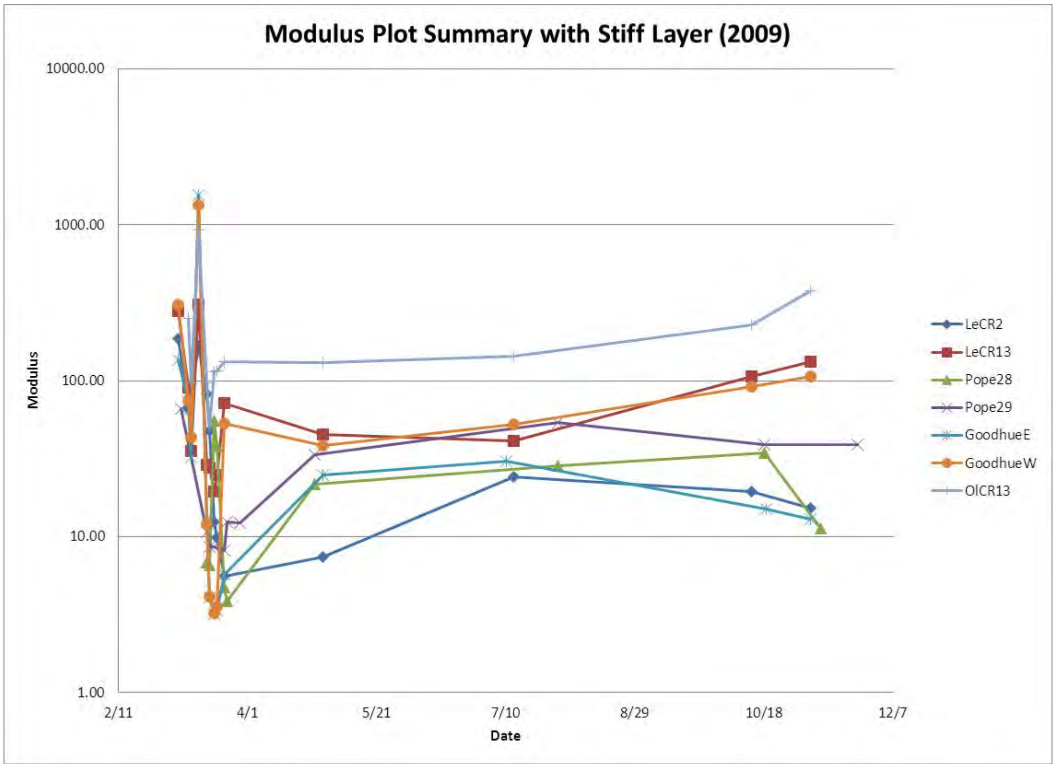

Figure 35: Modulus Plot Summary with Stiff Layer (2009)

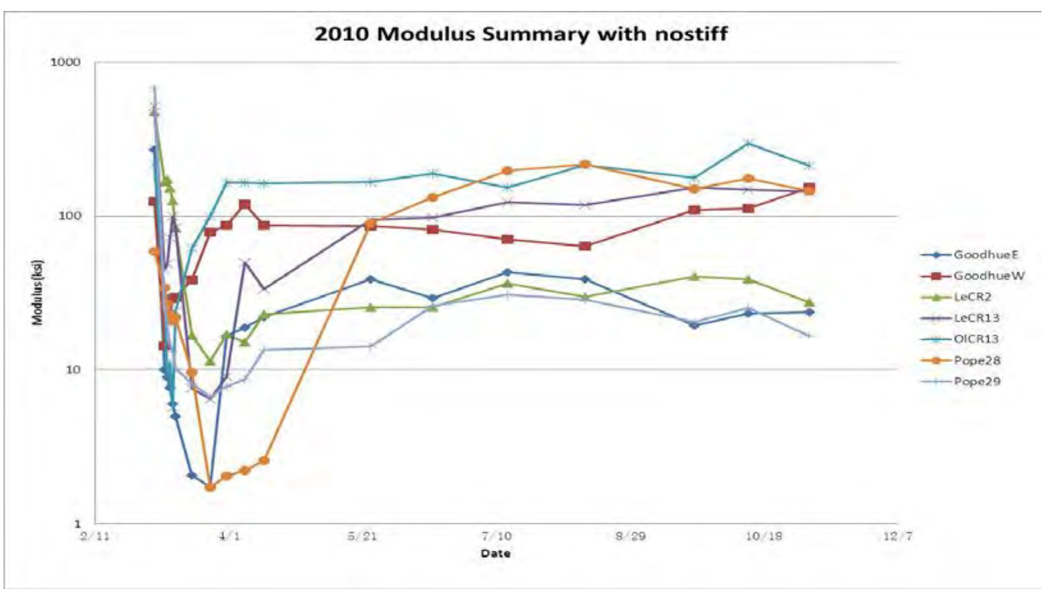

Figure 36: Modulus Plot Summary without Stiff Layer (2010)

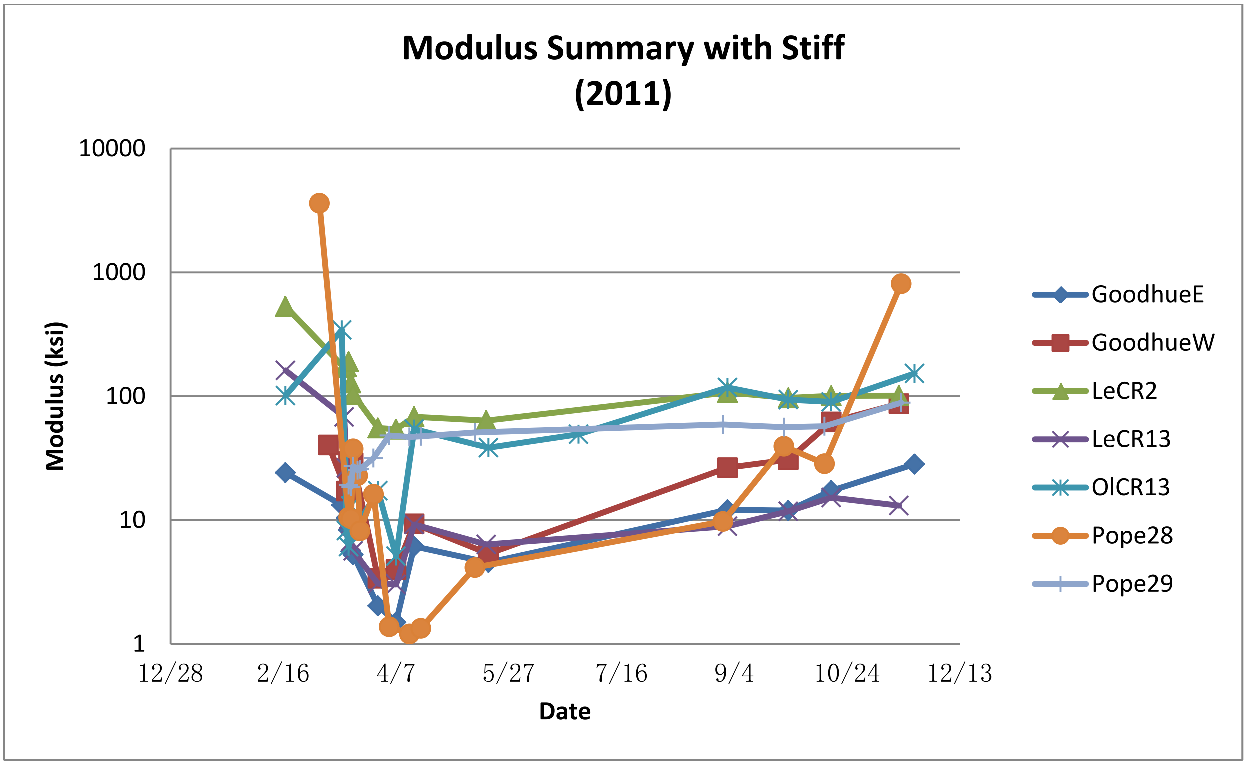

Figure 37: Modulus Plot Summary with Stiff Layer (2011)

In general, it can be seen that non-stabilized FDR (Pope 29 and Goodhue E) has lower strength than other materials and also has weaker strength in the spring time. This illustrates that SFDR is typically less sensitive to spring thaw than FDR materials.

Figure 38 shows modulus ratios between backcalculated modulus of the base materials and modulus of Class 5 material. GE of Class 5 is 1, so, the ratio is a measure of the GE of the material.

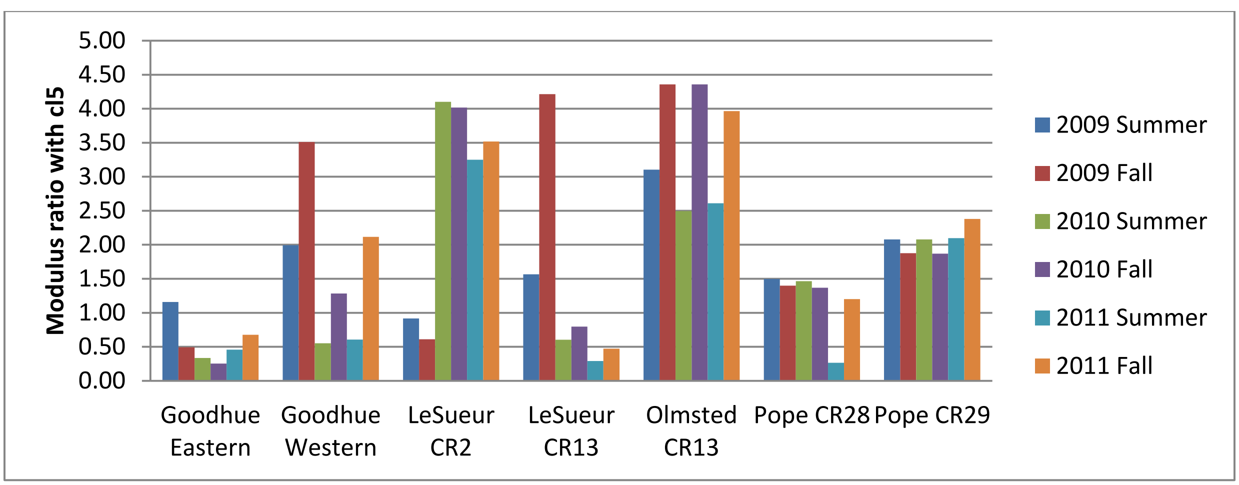

Figure 38: Modulus Ratios

Figure 38 shows that some ratios are very high, which indicates that the method used to estimate GE may not be appropriate.

4.10 Method Based on Hogg Model

As with any field study, discrepancies and inconsistencies are present. Nonetheless, it is apparent that the stabilizer is exerting an impact on the base, but it cannot be quantified. Many other factors are different for each county, including the year of construction, AADT, asphalt concrete, and stabilized depth, which contribute to the effectiveness of the stabilizer.

To provide a summary, the Hogg Model method is used to calculate the Granular Equivalencies (GE) from the FWD deflection data. The Hogg model is based on a hypothetical two-layer system consisting of a relatively thin plate on an elastic foundation. The method in effect simplifies the typical multilayered elastic system with an equivalent two-layer stiff-layer-on-elastic foundation model. Depending on the choice of values along the deflection basin used to calculate subgrade stiffness, the tendency exists to either over- or underestimate the subgrade modulus. The Hogg model uses the deflection at the center of the load and one of the offset deflections. Hogg showed that the offset distance where the deflection is approximately one-half of that under the center of the load plate was effective at removing estimation bias. The calculations consider variations in pavement thickness and the ratio of pavement stiffness to subgrade stiffness, since the distance to where the deflection is one-half of the deflection under the load plate is controlled by these factors.

The method also takes temperature, season, time of the day, and thicknesses of the layers into account. The Effective Granular Equivalencies (EGE) can be obtained using this method, which is the sum of the GEs for all layers; the seasonal factors are applicable for Jun-Oct. The effective depth (H_p) is assumed as 2/3 of the distance where 50% percent of the maximum deflection occurs. This value is interpolated from the locations of two sensors that are closest to 50% of the maximum deflection. Since the GE of the asphalt layer is assumed to be 2.25 and that of the subgrade layer is assumed to be Class 5 material, which has GE = 1.0, the GE of the base material can be calculated with the following equation:

$$GE_{base} = \frac{EGE - 2.25 \times Asphalt\ Thickness - 1.0 \times H_p}{Base\ Thickness} \tag{24}$$

The equation is adjusted accordingly if the effective depth does not reach the subgrade layer.

Figures 39-41 are a summary of the GE values of each road site test section from 2009-2011. It can be seen that, similar to the modulus values, spring thawing affects GE values as well. Higher GE values can be seen in early spring and late fall when temperatures are still low. As soon as temperatures rise, the GE values decrease.

In general, modulus for the base is higher when a stiff layer is assumed to be present for most counties, except LeSueur 13, for which a non-stabilized aggregate depth was assumed to be 3 in., and it could vary from 1 – 3 in. The values of the modulus for each county are generally consistent throughout the three year period, varying from 10 – 100 ksi. In terms of modulus comparison between counties, the ranking obtained from the calculation without a stiff layer appears reasonable. For example, for counties that have a stabilized base such as the Pope CR

28 and Goodhue Western section, the base modulus appears to be higher than their non-stabilized counterparts.

The calculated GE values appear to be quite high, which might be associated with the method used to obtain the effective depth. If the GE values are ranked for each year, similar results can be obtained. Comparing counties that are stabilized and those that are not, it is observed that the stabilized base exhibits a higher GE value than that of the non-stabilized.

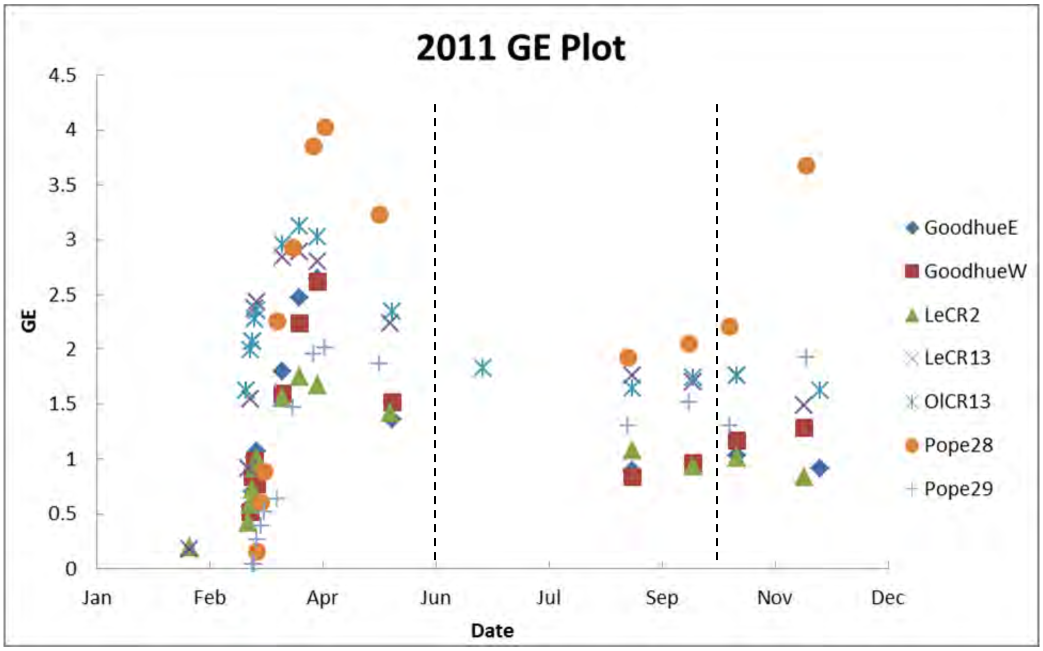

Figure 39 2011 GE Summary Plot

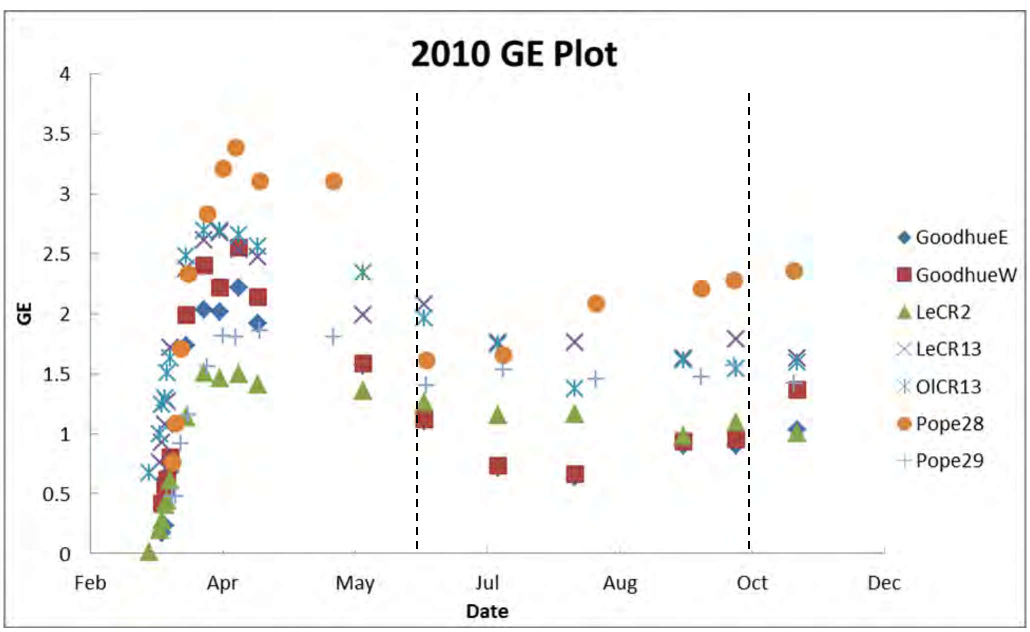

Figure 40: 2010 GE Summary Plot

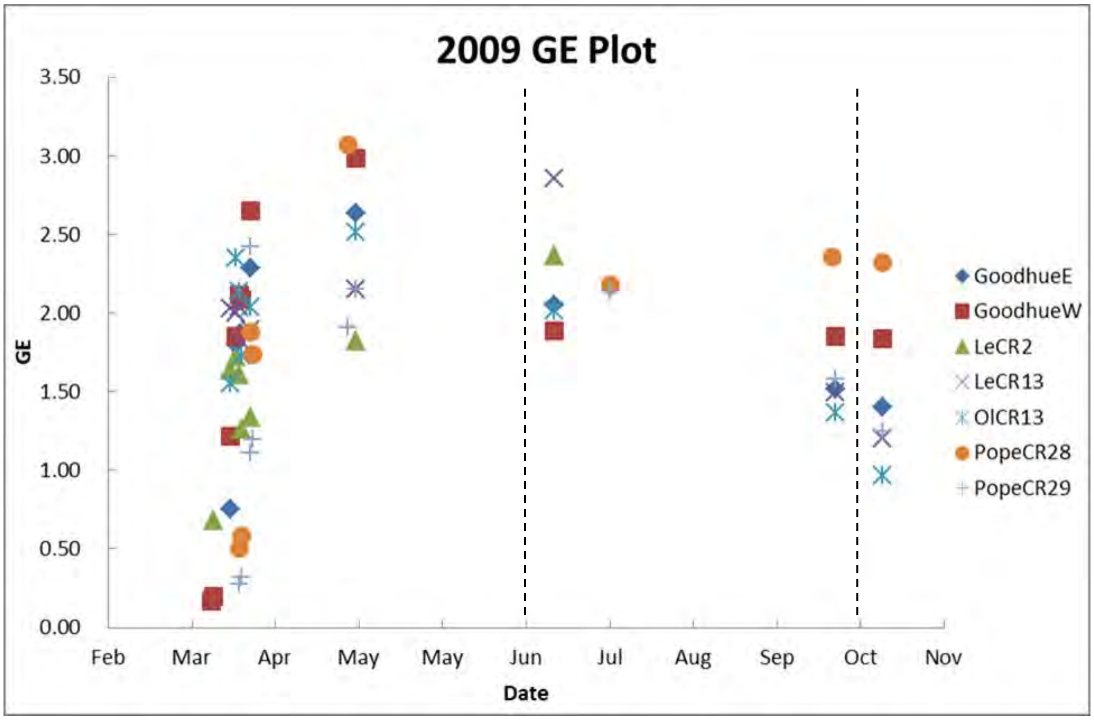

Figure 41: 2009 GE Summary Plot

Comparing the GE ranking and the modulus ranking between counties, the results are similar, but not completely consistent. For example, the modulus for Olmsted CR 13 is the highest for all three years, while the GE value for Olmsted CR 13 is only ranked around the middle. One possible reason for these inconsistencies is the method used to calculate GE values. Apart from the fact that there might be some problems with the calculation of the effective depth, another important parameter used to calculate the GE for the base, the effective GE, is greatly dependent on the seasonal adjustment factor (SAF), which is also dependent upon the sub-grade soil type, where plastic, semi-plastic, and non-plastic make a difference. Such information for each county is incomplete. Only LeSueur County Roads are specifically given as plastic sub-grade soil; Pope County Roads are class 4 (assumed to be non-plastic) and Goodhue and Olmsted CR 13 are given as N/A (assumed to be non-plastic as well). The back-calculation routine might also contribute to the discrepancies, as the results depend significantly on the initial input.

Chapter 5. Summary and Recommendations

Full-depth reclamation (FDR) is a recycling technique where the existing asphalt pavement and a predetermined portion of the underlying granular material are blended to produce an improved base course. FDR is an attractive alternative in road rehabilitation: resources are conserved, and material and transportation costs are reduced as recycling eliminates the need for purchasing and hauling new materials and disposing of old materials. An additive is sometimes used, and this process is referred to as stabilized full-depth reclamation (SFDR). Previous research has demonstrated that the strength of a traditional aggregate base (such as Class 5) normally shows a weakening during springtime thaw. It is part of the reason that spring load restrictions have been applied on some local pavements during each year's spring thaw period. However, not much research has been conducted on seasonal effects of SFDR base.

Currently, MnDOT pavement design recommends granular equivalency, GE = 1.0 for non-stabilized FDR material, which is equivalent to Class 5 material. For SFDR, there was no guideline for the GE value at the time this project was initiated (2009). Some local engineers believe that GE of FDR material should be greater than 1.0 (Class 5), especially for SFDR.

The objective of this project was to (1) estimate GE values of both non-stabilized and stabilized full-depth reclamation materials used for pavement base layer, and (2) assess spring thaw effects on stiffness of both stabilized and non-stabilized FDR. Falling Weight Deflectometer (FWD) tests were performed on seven selected sections on county roads in Minnesota. FWD tests were performed over a three year period. During spring thaw of each year, FWD was conducted daily in the first week of thawing in an attempt to capture spring thaw weakening of the base. After the spring thaw period, FWD was conducted monthly to study base recovery and stiffness changes through the seasons.

It is known that the GE factor is an empirical number, which is used by MnDOT to describe stiffness of asphalt and base materials. There is no well-defined method to determine GE either through mathematical computation or laboratory test. In this work, three different approaches were used in an attempt to estimate GE factor from FWD deflections. The first method is the AASHTO method, the second one is backcalculation using EVERCALC, and the third one is a MnDOT method developed by Erland Lukanen. It was found that the third method provides reasonable GE values.

Based on the data collected for this project, the average GE of SFDR is estimated to be 1.5. Certainly, the value varies from project to project as construction and material varies from project to project. It appears that all the materials tested showed seasonal effects on stiffness. In general, the stiffness is lower in spring than that in summer and fall, typical behavior for unbound base materials. Most of the SFDR materials tested in this project, but not all, showed improved seasonal stiffness.

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