U.S. Department of Transportation
Federal Highway Administration

TEACR Engineering Assessment

Temperature and Precipitation Impacts on Cold Region Pavement:
State Route 6/State Route 15/State Route 16 in Maine

This is one of nine engineering case studies conducted under the Transportation Engineering Approaches to Climate Resiliency (TEACR) Project.[1] This case study focused on the vulnerability of cold region pavement to changes in temperature and precipitation.

Case Study Snapshot

Purpose: Evaluate potential impacts from projected change in temperature and precipitation to pavement performance.

Location: State Route 6/State Route 15/State Route 16, Piscataquis County, Maine

Approach: The pavement performance was estimated using mechanistic-empirical pavement performance prediction models and was evaluated against future projections of temperature and precipitation.

Key Findings: As various climate change scenarios indicate, there will be a steady increase in ambient temperature over the course of 21st century. These trends will result in a modest increase in pavement distresses. Shorter winters will require adjustments to seasonal load allowances and restrictions.

Key Lessons: Most adaptation strategies can be implemented as part of routine pavement rehabilitation. There is a need to monitor climate trends and re-evaluate future design related decisions using newly available climate information. Also, since the impacts are systemic, there is a need to evaluate the economic consequences of implementing adaption measures. The economic impacts of shorter winters on freight truck traffic should be evaluated.

Overview

The primary goal of this case study was to investigate the impacts of changing climate, including both temperature and precipitation, on the performance of pavement in cold regions. Environmental conditions have a significant effect on the performance of both flexible and rigid pavements.[2] Seasonal variations in climate factors affect both material properties and the integrity of pavement layers and subgrade[3] (e.g., rutting, cracking) and, consequently, have a profound influence on the structural performance, riding comfort, and safety of pavements.

Most pavement design approaches recognize the criticality of climate factors in materials selection, construction practices, and structural thickness determination, however, these design methodologies have

[1] For more information about the project, visit the project website at: https://www.fhwa.dot.gov/environment/climate_change/adaptation/ongoing_and_current_research/teacr/

[2] Pavements are generally classified into flexible and rigid pavements. A flexible pavement is constructed with asphalt concrete (commonly simply called "asphalt") as a surface course resting on a base course, typically made of granular materials, and a subbase placed over a prepared soil bed. A rigid pavement is constructed with Portland cement concrete placed either directly on a prepared soil bed or with a layer of granular or stabilized material between the pavement and the prepared soil bed.

[3] Subgrade is the prepared soil platform or foundation upon which the pavement is built. The soil layers can be left as is (un-compacted) or compacted using a roller to improve the profile.

traditionally relied on historical weather records to evaluate and incorporate climate related considerations in design decisions. Given the concern with climate change, there is a need to evaluate how future projected changes in temperature and precipitation will affect the performance of pavements.

The research team selected State Route 6/State Route 15/State Route 16 (SR-6/SR-15/SR-16) in Guilford, Piscataquis County, Maine, for this case study. SR-6/SR-15/SR-16 around Guilford is representative of many roads in the New England region in terms of the roadway type (a two-lane highway in a rural area) and its exposure to long, cold, and snowy winters.

With climate change, temperature and precipitation patterns are expected to change over time which will change the freezing and thawing patterns in pavement systems[4]. The objective of this case study is to investigate how considerable variations in precipitation and temperature due to climate change affect the performance of pavements located in colder climates, particularly pertaining to frost heaving and thaw weakening.

To analyze the effects of these environmental conditions this analysis projected the future climate conditions, analyzed the pavement performance under those conditions and developed options for adapting to the future conditions.

The results show that an increase in ambient temperatures over the remainder of the 21st century that will modestly increase the physical stresses on pavement. The adaptation options (Table 1) include relatively inexpensive measures consisting of increasing the layer thickness of the asphalt pavement over time as the ambient temperature and precipitation increase. Table 1 presents a summary of impacts expected due to climate change for a typical flexible pavement serving a two-lane rural highway in the New England region. The impacts presented in this table were assessed under the RCP 8.5 scenario for the period up to 2099.

[4] Pavement systems include both the pavement structure (i.e. all pavement layers placed during construction) and the roadbed soil.

Table 1: Summary of Climate Change Impacts and Adaptation Options.

Topic	Expected Impact by 2099	Recommended Adaptation Strategy	Other Considerations	Impacts of Implementing Adaptation Strategies
Load Related Fatigue Cracking	Fatigue cracking will increase by 34 percent	Strengthen by increasing pavement thickness	Use polymer modified asphalt binders; improve subsurface drainage; and consider subbase/subgrade stabilization	Higher construction costs
Subgrade Rutting	Subgrade rutting will increase by 40 percent	Strengthen by increasing pavement thickness	Improve subsurface drainage; and consider subbase/ subgrade stabilization	Higher construction costs
AC Rutting	AC rutting will increase by 42 percent	Use polymer modified asphalt binders starting 2060s	Optimize AC mix designs	Higher construction costs
Serviceability Loss due to Frost Heave only	Serviceability will improve by 10 percent	None	N.A.	N.A.
Winter Weight Premium	No opportunities for winter weight premiums by the early 2080s	Strengthen by increasing pavement thickness	Use polymer modified asphalt binders; improve subsurface drainage; and consider subbase/subgrade stabilization	Higher construction costs
Spring Load Restrictions	Early posting of load restrictions by at least 4 weeks	None	N.A.	N.A.

The remainder of this case study is organized around the Adaptation Decision-Making Assessment Process (ADAP), as shown in Figure 1, to illustrate how it was applied for this analysis.

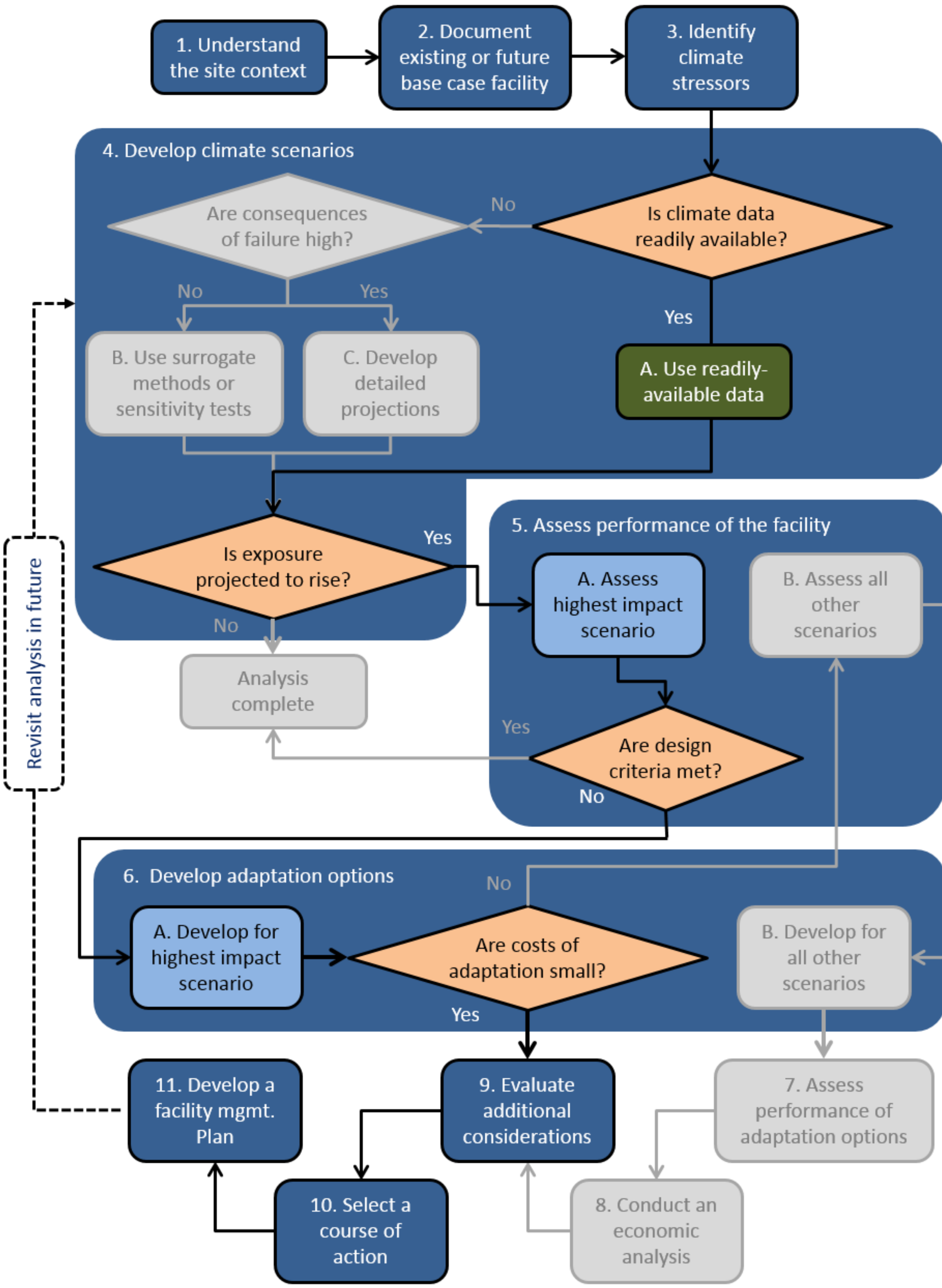

Figure 1: Adaptation Decision-Making Assessment Process (ADAP) Used for this Analysis (steps not completed are indicated in gray).

Contents

Background: A General Discussion of Cold Region Pavement Design

Before discussing the specifics of this case, however, it is important to understand the basics of cold region pavement design.

The Frost Heave Mechanism

Climate factors that most influence frost heave[5] are precipitation and temperature. The amount of precipitation and the temperatures projected to increase in the coming decades. Three factors must be present to cause frost heave: (1) frost-susceptible soils, (2) freezing temperatures, and (3) the availability of moisture in the subgrade. Figure 2 presents a schematic of the mechanism of frost heaving. Heave begins with the formation of frost in the pavement and soil subgrade, the formation of which depends on the severity and length of the freezing season. When water in soil pores freezes to ice, the original volume of water expands by approximately nine percent to exert an upward pressure on pavement layers.

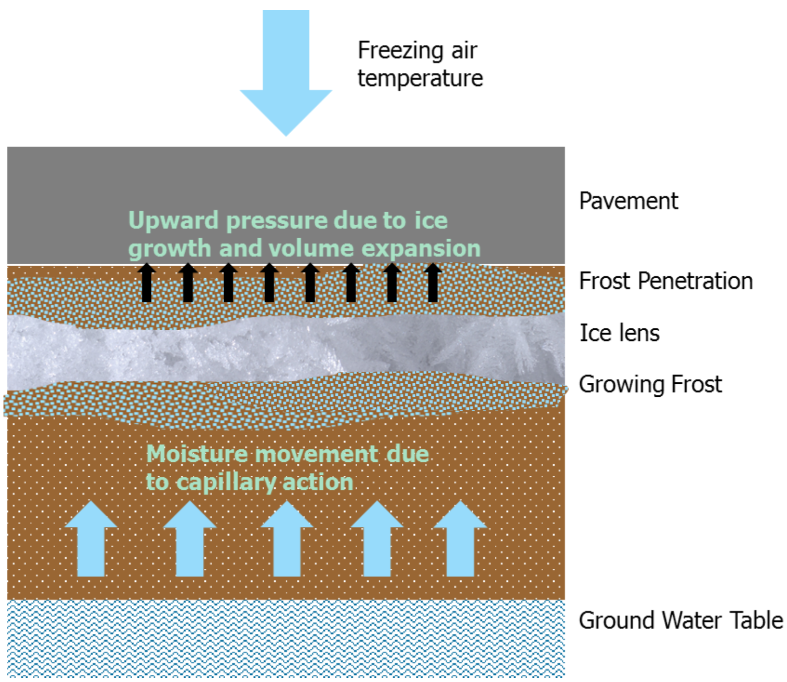

Figure 2: Mechanism of Frost Action.

Furthermore, the conversion to ice causes a dearth of liquid water in the frozen zone of the subgrade soil. The moisture deficiency in the soil increases the capillary action[6] and induces an upward movement of moisture through narrow soil pores toward the frozen zone from the underlying unfrozen soil. The frozen soil continues to expand with additional moisture supply, resulting in further expansion of ice lenses and more volume expansion in the subgrade. The

[5] Frost heave refers to an upward swelling of a portion of the pavement caused by the formation of ice crystals in a frost-susceptible subgrade or base course. In pavements, differential frost heave may create bumps along the roadway resulting in hazardous driving conditions.

[6] Capillary action (commonly observed in wicks) is the process whereby moisture is drawn up through an object.

moisture movement is more pronounced in soils containing silt as they have high capillarity and high permeability to attract more water. A shallow ground water table can provide a continuous supply of moisture to worsen the problem of ice growth in subgrade soils. Note that the moisture may also infiltrate into the subgrade from the roadway surface through cracks and as a result of poor drainage.

Overall, the heaving phenomenon results in an increased upward pressure on the pavement layers causing detrimental effects to pavements such as poor ride quality or loss in smoothness[7] and cracking due to distortions. Figure 3 presents a picture showing pavement damage due to frost heave. Not only does this damage to pavement often require expensive fixes to restore ride quality and ensure structural adequacy, the differential soil movements due to frost heave may create operational and safety issues and potential vehicle damage.

Figure 3: Pavement Damage due to Frost Heave.[8]

Roadways located in wet freeze climatic regions (see Figure 4),[9] such as in Maine, undergo one or more cycles of freezing and thawing during winter and early spring where the asphalt and soil subgrade layers harden under freezing conditions and soften during thaw (i.e. thaw weakening). Frost beneath the roadway surface can have detrimental effects on pavements, particularly when

[7] Smoothness is a measure that reflects irregularities in the pavement profile. Smoothness is often defined by a standard measure called the International Roughness Index (IRI). The higher the IRI is, the rougher the pavement surface is.

[8] FHWA, A Quarter Century of Geotechnical Research, Federal Highway Administration, Report No. FHWA-RD-98-139, June 1999. Accessible at: http://www.fhwa.dot.gov/publications/research/infrastructure/geotechnical/98139/04.cfm.

[9] The FHWA's Long Term Pavement Performance Program (LTPP) categorizes the geographical location of pavements into four climatic regions: wet freeze (e.g. New York and Minnesota), wet no-freeze (e.g. Florida and Georgia), dry freeze (e.g. the Dakotas and Idaho), and dry no-freeze (e.g. Arizona and New Mexico). The wet/dry regions are defined in terms of average annual precipitation (20 inches per year being the threshold) and freeze/no-freeze regions as a function of an average annual freezing index. The freezing index is defined as a cumulative number of degree-days when the air temperature is above 32° Fahrenheit.

frost-susceptible[10] soils are present. Frost susceptible soils are those soils that are particularly susceptible to volume expansion and segregation of ice lenses[11] when frozen which can cause uplift and damaging of pavement, a process known as frost heaving. The soil profile in much of the northern U.S., including Maine, is known for the presence of silt,[12] sandy silt, and loam,[13] soils that are highly susceptible to frost heave (see Figure 5).

The Guilford site in Maine has all the three factors to produce a frost heave action: (1) presence of frost-susceptible silt and sandy silt soils; (2) long, cold, snowy winters;[14] and (3) a shallow ground water table[15] to supply moisture in abundance.

Figure 4: Map of FHWA Long Term Pavement Performance Climate Zones.[16]

[10] According to the U.S. Army Corps of Engineers (USACE), all soils that contain more than three percent of particles smaller than 0.0008 inches by weight are considered frost susceptible. USACE uses four categories (F1 through F4) to indicate different degrees of frost susceptibility depending on the soil type (gravel, sand, silt, or clay) and percent weight of particles smaller than 0.0008 inches by weight.

[11] An ice lens or ice lenses are formed when moisture, diffused within soil or rock, accumulates in a localized zone.

[12] Silt is a sedimentary deposit consisting of very fine granular particles whose size is somewhere between sand and clay.

[13] Loam soil has more or less equal proportions of sand, clay, and silt. Depending on their relative proportions, loam soils can be further classified into sandy loam, silty loam, clay loam, loamy sand, silty clay loam, and sandy clay loam.

[14] Colder and longer winters cause the penetration of frost deeper into the pavement and subgrade.

[15] The SR-6/SR-15/SR-16 roadway in the Guilford area runs parallel and is within 1,000 feet of Piscataquis River. Since it is so close to the river, the ground water table is within six feet of the soil surface.

[16] Image Source: Schwartz et al, 2015.

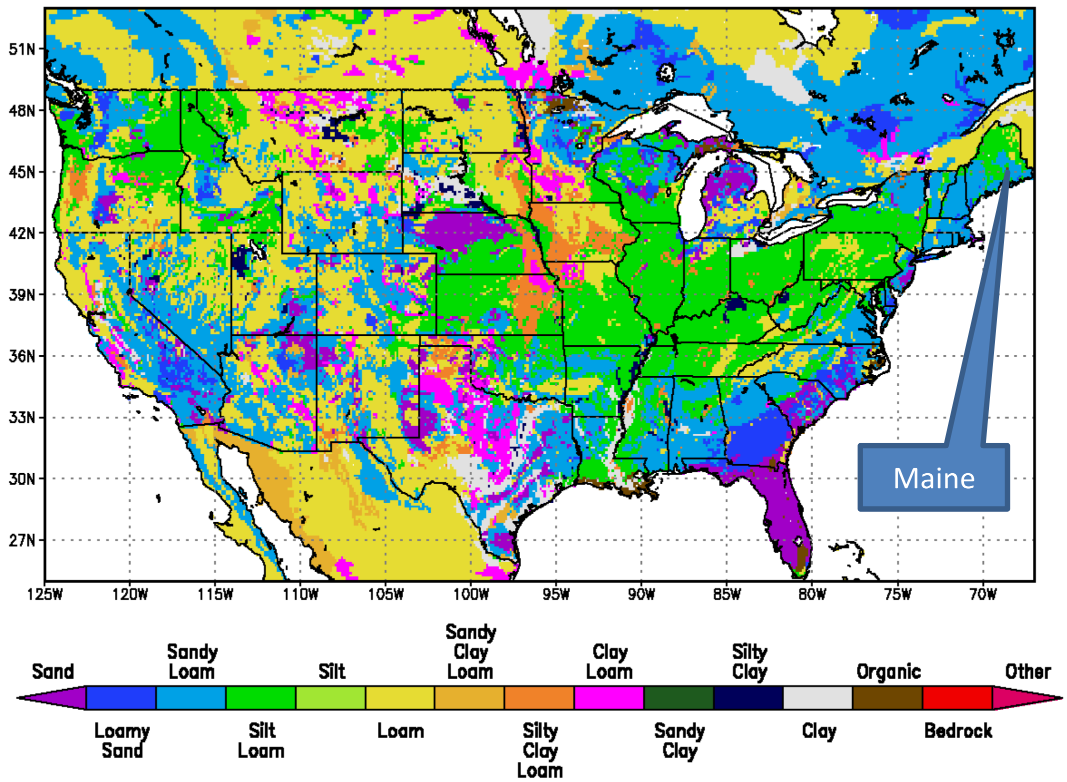

Figure 5: Predominant Surface Soil Classes of the Contiguous United States[17]. Silt and sandy silt soils are most susceptible to the formation of ice lenses.

Thaw Weakening

During winter, when soil water freezes to ice, the resilient modulus[18] of frozen soil could rise as high as 1 to 3 million pounds per square inch, which can be 20 to 120 times higher than the value of the modulus before freezing. During spring thaw, as the frost begins to disappear from the top-down, the subgrade is soaked with excess free water from melting of ice lenses. The soil subgrade rapidly loses its load bearing capacity (by about 50 to 60 percent of its resilient modulus under normal conditions) under wet conditions, thereby significantly weakening the pavement system. Allowing heavy vehicles on such weakened pavement will contribute to significant structural damage or even failure.

Figure 6 is a photograph that shows pavement damage caused by thaw weakening on a typical roadway in Maine. As the excess soil water is eventually drained from the pavement system, a period of recovery occurs over late spring and early summer where the subgrade slowly regains

[17] Image source: NASA NLDAS, accessed 2015.

[18] Resilient modulus is the standardized measurement of resistance of roadbed soil or other pavement material to being temporarily deformed (i.e. its stiffness or, more technically, a standardized modulus of elasticity) based on the recoverable strain under repeated loads. Among other factors, the resilient modulus proportionately decreases with increasing moisture. Typical values for natural soil may range from 5,000 to 40,000 pounds per square inch.

the lost load bearing capacity back to its normal levels. Figure 7 presents a schematic of typical seasonal variations in resilient modulus (i.e. load bearing capacity) of soil subgrade.

Figure 6: Pavement Damage Caused by Thaw Weakening.[19]

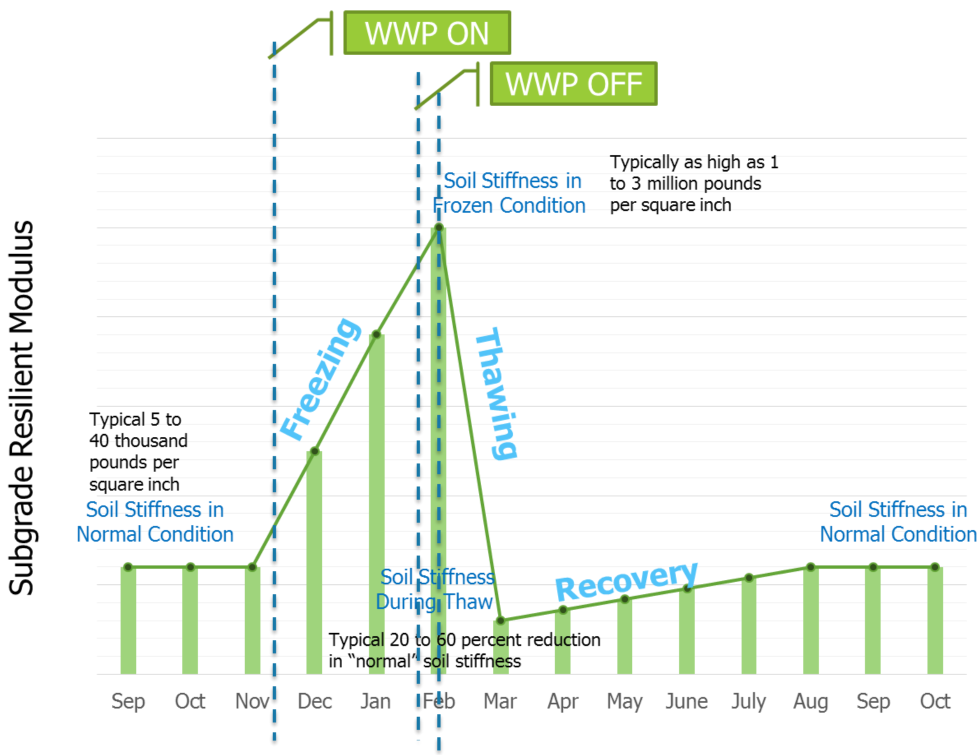

Figure 7: Schematic of Typical Seasonal Variations in Resilient Modulus of Subgrade. See below for a discussion of winter weight premium policy (WWP).

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[19] Image source: Maine Local Road News, MaineDOT Newsletter, Winter 2007/2008 accessible at: http://www.maine.gov/mdot/csd/mlrc/documents/pdf/Winter%2007-08%20Newsletter-MLRC%20.pdf.

Most highway agencies, especially in the northern United States (e.g. Minnesota, Washington, Maine, New York, Iowa) and Canada, are cognizant of this seasonal change in the overall load carrying capacity of pavements and their vulnerability to heavy loads during spring thaw. In response to this vulnerability, the highway agencies post spring load restriction (SLR) policies within their jurisdiction to regulate the axle load of trucks during the spring thaw period. Under SLR policies, a highway agency will typically reduce the maximum allowable weight by as much as 90 percent from their normal legal limits.[20] In most states, the restrictions[21] are posted in early March, often with a few days' notice, and may extend through late April or early May. The restrictions typically run for a span of eight weeks. The agencies' decisions to post SLR may be based on a combination of experience, climate monitoring, field observations and testing, and technical analyses.

Highway agencies may also have winter weight premium (WWP) policies that take advantage of the pavement hardening during winter to allow for a temporary increase in allowable axle load limits. A typical premium may involve a 10 percent increase. The Maine Department of Transportation (MaineDOT) does not currently allow winter weight increases on its roadways; however, since climate change may impact WWP and SLR policies in tandem, the technical aspects of WWP were investigated in this study. Figure 8 presents a schematic to demonstrate how WWP and SLR are typically posted in highway agencies based on seasonal change in pavement structural capacity.

[20] Federal legal limits are 20,000 pounds for single axles, 34,000 pounds for tandem axles (two sets of axles in combination), and 80,000 pounds for gross vehicle weights (total vehicle weight). Some states have adopted the federal legal limits in their entirety while other states have relaxed the federal limits in their jurisdictions either based on individual axle type or gross vehicle weight.

[21] Restrictions may be placed on all or specific roadways. There may be some exemptions for certain vehicles such as emergency vehicles, maintenance vehicles, and trucks carrying perishable goods.

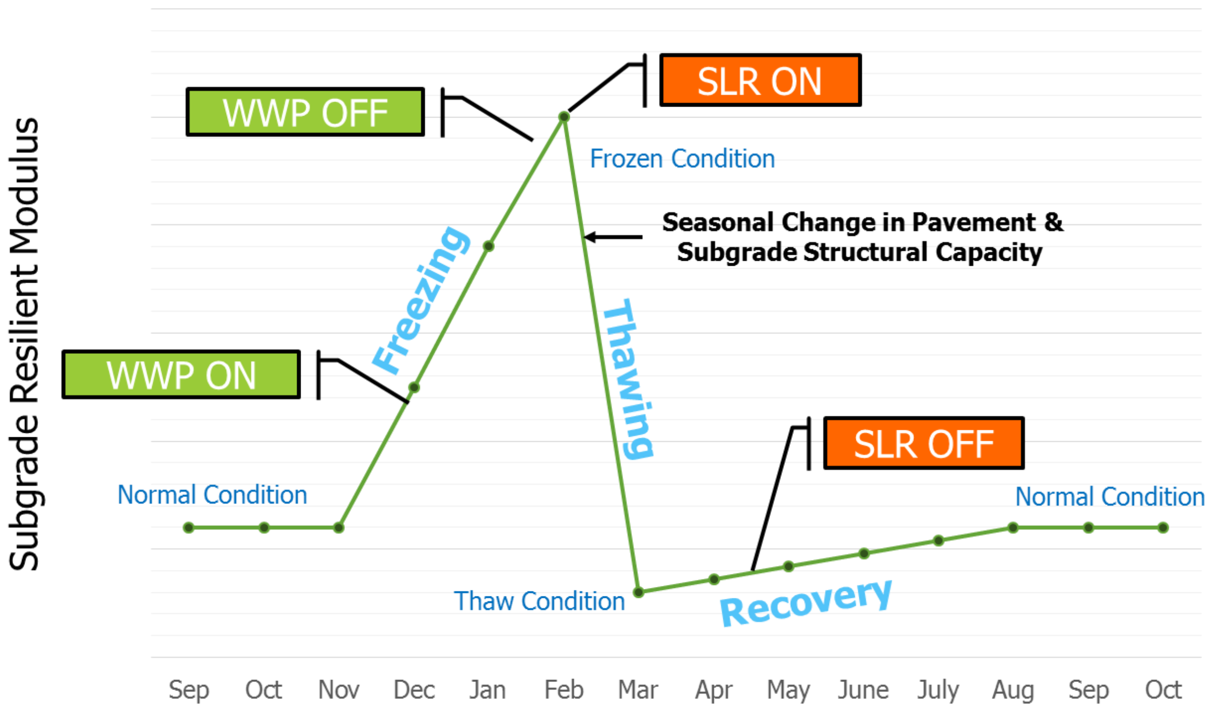

Figure 8: Schematic of Winter Weight Premium and Spring Load Restriction Policies.

As this discussion has made clear, it is necessary to understand how climate change will impact the overall performance of pavements from the perspective of frost heave potential as well as seasonal load restriction policies. In addition, other climate-related effects on pavement need to be considered in order to develop a holistic understanding of future pavement performance on SR-6/SR-15/SR-16 with climate change. The following is a complete list of items the research team has investigated:

- Impacts from temperature changes:
    - Recommended changes in the performance grade requirements of asphalt binder[22] and dynamic modulus[23] of asphalt concrete[24] mixtures due to changes in temperature

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[22]Asphalt binder is a viscous petroleum-based product that essentially acts as the glue that holds the asphalt together. Suitable asphalt binder grades are selected for use in paving asphalt mixtures based on the climate, (high and low temperature statistics), total traffic volume, and traffic speed of the roadway in which they are intended to serve.

[23] Dynamic modulus is a mechanical property that indicates the stiffness (or the ability to withstand deformation against the applied force) of a material. Dynamic modulus of asphalt concrete is a function of temperature and loading time. The same material of asphalt binder or asphalt concrete mixture will have higher stiffness under lower temperatures and longer loading time (i.e. lower traffic speeds) and lower stiffness under higher temperatures and shorter loading time (i.e. higher traffic speeds).

[24] Asphalt concrete (commonly referred to simply as asphalt), a key component of flexible pavement design, is pavement comprised of a mixture of asphalt, aggregate, and other admixtures as may be required.

    - Changes in asphalt pavement performance resulting from: (1) load-related fatigue cracking,[25] (2) subgrade rutting,[26] (3) asphalt concrete (AC) rutting,[27] and (4) serviceability[28] loss due to frost heave
  - Impacts from precipitation changes:
    - Variation in the Thornthwaite Moisture Index (TMI)[29] and its impact on soil moisture and soil support conditions
- Posting of seasonal load restrictions on highways, including the increases in allowable gross vehicle weights in winter as well as restrictions during spring thaw

Details of the Analysis

Step 1: Understand the Site Context

The roadway selected for the case study is a 4.1-mile long section of SR-6/SR-15/SR-16 near the town of Guilford in Piscataquis County, Maine (see maps in Figure 9 and 10). SR-6/SR-15/SR-16 runs parallel to the Piscataquis River in an east-west direction and is typically within 1,000 feet of the river's shoreline. Figure 11 shows a photograph of the existing roadway and the Piscataquis River. The remainder of this section discusses the transportation network and environmental context of the study area.

[25] Load-related fatigue cracking is defined as a series of interconnected cracks (characteristically with a "chicken wire/alligator" pattern) caused by fatigue failure under repeated traffic loading. The cracking initiates at the bottom of the asphalt concrete layer, due to strains caused by wheel loads, and propagates to the surface.

[26] A rut is a surface depression in the wheel paths caused by permanent deformation in any of the pavement layers or soil subgrade due to repeated traffic loading. Subgrade rutting is a form of depression on the pavement surface predominately caused by permanent deformation in soil subgrade. Significant rutting can lead to major vehicle safety issues and structural failures.

[27] Hot mix asphalt is another term for an asphalt roadway surface treatment. The hotness refers to the typical production and application process whereby a performance grade binder is mixed at high temperatures with aggregate and other admixtures and then laid down and compacted while hot. AC rutting occurs when problems with the mix design are the cause of the rutting as opposed to issues with the subgrade.

[28] Serviceability is the ability of the pavement to provide a safe and comfortable ride to users. Serviceability ranges from five (perfect) to zero (impassable).

[29] TMI is a measure that indicates the humidity or aridity of soil in a geographic region.

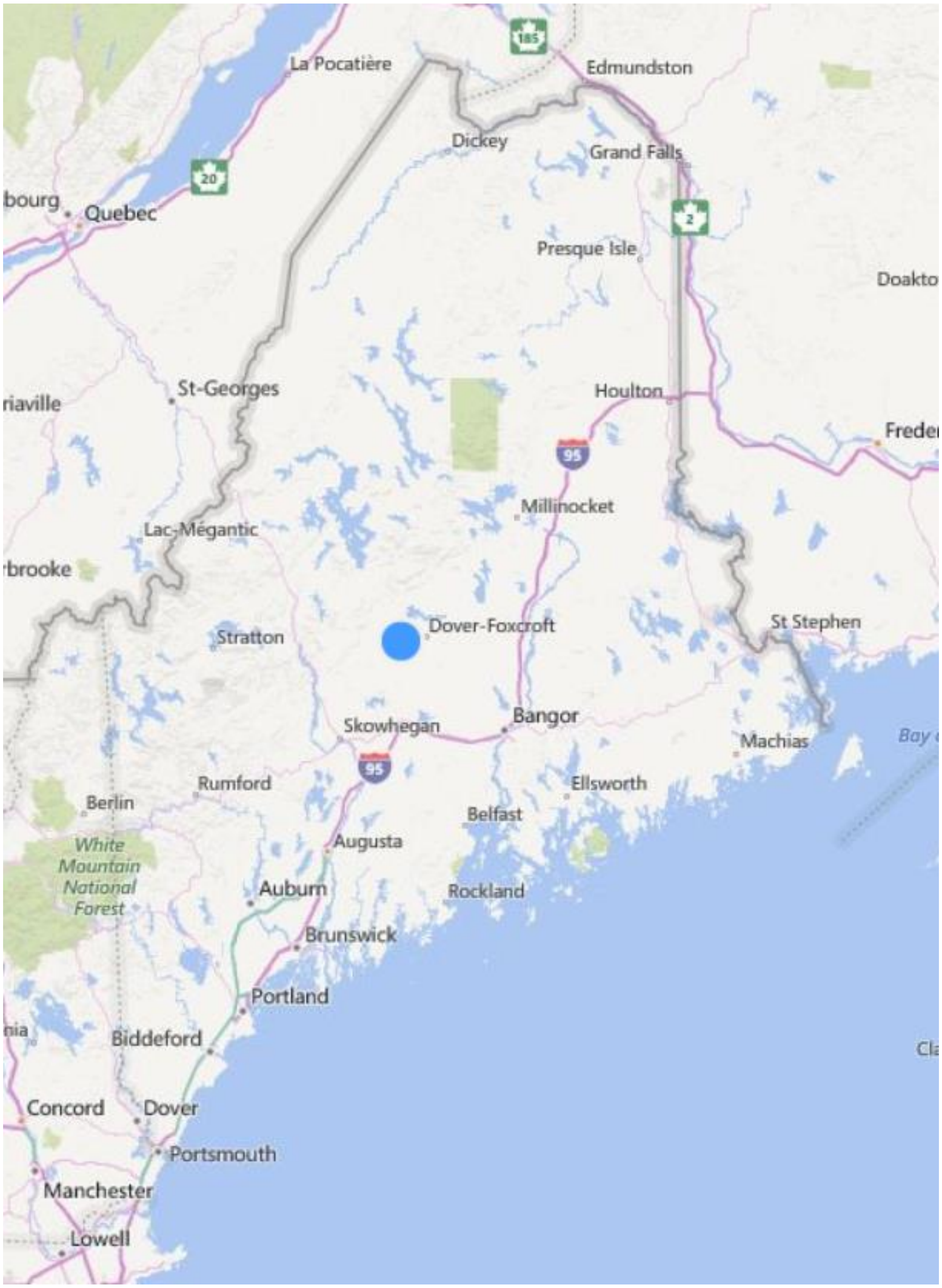

Figure 9: Location of Guilford (blue circle) in Maine.[30]

[30] Image source: Bing Maps (as modified).

Figure 10: Study Area Project Limits[31]

Figure 11: Photograph SR-6/SR-15/SR-16 alongside the Piscataquis River.[32]

Transportation Network Context

SR-6/SR-15/SR-16 in the Guilford area is a rural minor arterial highway that connects the towns of Guilford and Dover-Foxcroft. The SR-6 highway is the third-longest state highway in Maine and runs between the border with Quebec, Canada in the west and New Brunswick, Canada in the east. SR-15 runs concurrently with SR-6 between Dover-Foxcroft in the south and Jackman in the north. SR-16 runs concurrently with SR-6 from Abbot Village in the west to Lagrange in the east.

Per 2004 estimates, the SR-6/SR-15/SR-16 roadway section carries an average daily traffic of 5,530 vehicles per day with eight percent trucks at a posted speed limit of 55 miles per hour. The projected annual traffic growth rate through 2024 is 1.5 percent.

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[31] Image source: DeLorme Topo (as modified).

[32] Image source: Google.

Environmental Context

The environmental context of the site is important for understanding the implications of climate change on pavement performance. This subsection focuses on describing the soil types in the study area and their frost susceptibility.

Soil Types

MaineDOT conducted geotechnical investigations at the Guilford site in 2002 to examine and measure soil parameters for roadway design and construction. The geotechnical investigations involved soil borings[33] at multiple locations along the roadway alignment as well as follow-up laboratory testing.[34]

The geotechnical investigations determined that the predominant soil type at the Guildford site is sandy silt with an intermittent presence of silt soils. The depth to bedrock is approximately 10 feet from the ground surface, while the ground water table was present at an average depth of six feet from the surface.

Soil Frost Susceptibility

The U.S. Army Corps of Engineers (USACE) has established a frost design soil classification system to categorize soils based on their degree of frost susceptibility and thaw weakening. Per the USACE criteria, soils are classified into four groups depending on their particle sizes and plasticity indices[35] (see

Table 2). The degree of frost susceptibility generally increases from F1 to F4, with F1 and F2 soils expected to be relatively stable under frozen conditions and F3 and F4 soils expected to be more prone to frost heave and thaw weakening.

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[33] Soil borings typically involve drilling into the ground to characterize subsurface geological condition. The purpose of soil borings is to collect soil samples at various depths of the vertical soil profile for (1) material identification and characterization in the field or laboratory, (2) to determine ground water table depth, and (3) to ascertain bedrock depth.

[34] MaineDOT, 2002.

[35] Plasticity is a material property that indicates the degree to which a material undergoes irreversible damage in response to an applied force. Plastic materials undergo permanent deformation even after the force is withdrawn. The plasticity index indicates the range of water content over which the soil remains plastic.

Table 2: USACE Frost Susceptibility Criteria

Group	Description
F1	Gravelly soils containing between 3 and 10 percent finer than 0.0008 inches by weight
F2	(a) Gravelly soils containing between 10 and 20 percent finer than 0.0008 inches by weight (b) Sands containing between 3 and 15 percent finer than 0.0008 inches by weight
F3	(a) Gravelly soils containing more than 20 percent finer than 0.0008 inches by weight (b) Sands, except very fine silty sands, containing more than 15 percent finer than 0.0008 inches by weight (c) Clays with plasticity indexes of more than 12
F4	(a) All silts (b) Very fine silty sands containing more than 15 percent finer than 0.0008 inches by weight (c) clays with plasticity indexes of less than 12 (d) Varved[36] clays and other fine-grained banded sediments

Per the geotechnical investigation undertaken in 2002, the frost susceptibility ratings of soils prevalent at the Guilford site are predominantly F3 (moderately susceptible) or F4 (highly susceptible) as indicated in Table 3.[34] According to the 1993 American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures, the average frost heave potential of sandy silt soils is 0.28 inches per day.[37] For silts, the frost heave potential is somewhat greater ranging from 0.59 to 0.79 inches per day.[38] Frost heave can continue in an additive manner for multiple days if moisture continues to be present.

[36] Varved clays are multi-layer sedimentary deposits of silt and clay materials.
[37] AASHTO, Guide for Design of Pavement Structures, 1993.
[38] Note that this information is used in estimating the loss in pavement serviceability due to frost heave.

Table 3: USACE Frost Susceptibility Rating of Soils Along the Roadway Alignment at the Guilford Site. See map on the next page for station locations.

Station Number	Upper Soil Layer		Lower Soil Layer	
	Soil Type	Frost Susceptibility Rating	Soil Type	Frost Susceptibility Rating
0+736	Gravel	F1	N.A.	N.A.
0+860	Sandy Silt	F3	Silt	F4
1+660	Sandy Silt	F4	N.A.	N.A.
2+700	Sandy Silt	F3	Sandy Silt	F4
3+400	Sandy Silt	F3	N.A.	N.A.
3+650	Sandy Silt	F3	Silt	F4
4+400	Sandy Silt	F2	Sandy Silt	F3
4+500	Sandy Silt	F3	Sandy Silt	F3
4+850	Sandy Silt	F2	Sandy Silt	F4
5+250	Sandy Silt	F3	Silt	F4
5+350	Sandy Silt	F3	N.A.	N.A.
5+504	Silt	F4	Silt	F4
6+106	Sandy Silt	F2	Sandy Silt	F3
6+250	Sandy Silt	F3	Sandy Silt	F3
6+650	Sandy Silt	F3	Sandy Silt	F4
7+200	Sandy Silt	F3	Sandy Silt	F3

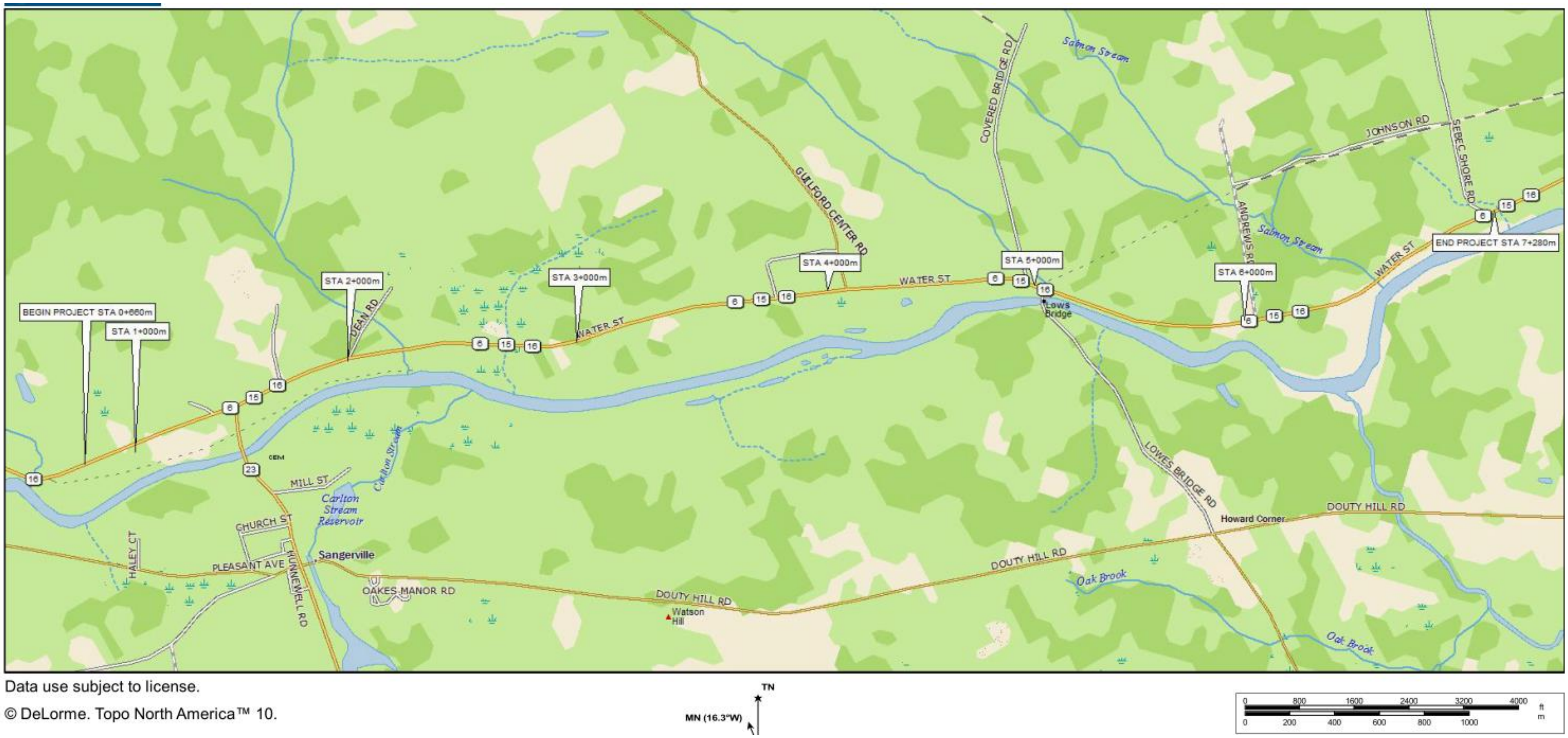

Figure 12: Route Map Showing Station Locations. Stations are in the metric notation used in the MaineDOT, Geotechnical Design Report for Proposed Rehabilitation of Route 6/15/16 and the Salmon Stream Crossing.[39]

[39] Source: Adapted from DeLorme Maps.

Step 2: Document Base Case Facility

The existing pavement within the study segment was last rehabilitated in 2004. The rehabilitated pavement on the SR-6/SR-15/SR-16 roadway is a conventional asphalt concrete that includes an eight-inch AC with PG 64-28 binder[40] placed on a 21-inch MaineDOT Type D gravel subbase over the underlying soil. The cost of AC rehabilitation for the study roadway segment was $6,635,780 in 2004 dollars.[41] Figure 13 presents the schematic of the existing pavement cross-section.

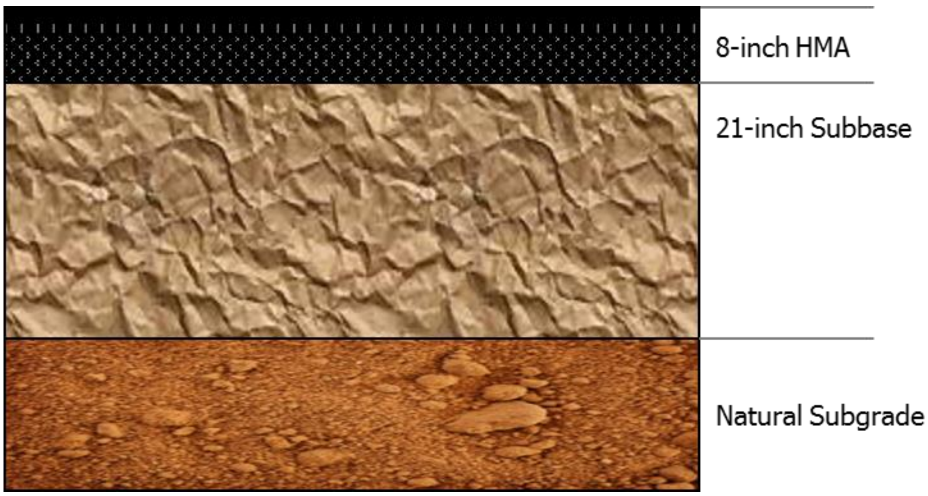

Figure 13: Existing Flexible Pavement Design Structure.[42]

The pavement was structurally designed to carry a traffic loading of 4.4 million flexible equivalent single axle load (ESAL)[43] repetitions, based on a daily ESAL of 604, over a 20 year design period (2004 to 2024).

For research purposes, MaineDOT installed sensors along SR-6/SR-15/SR-16 near Guilford to measure climate (air and pavement temperatures), traffic (vehicle types and axle weights), and engineering responses (stresses and strains at different depths of the pavement). In addition, non-destructive testing has been conducted periodically to measure material properties and responses of the in-place pavement structure.[44] This testing estimated that the in-place resilient modulus of the soil subgrade and gravel subbase were 10,500 and 12,000 pounds per square inch, respectively.

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[40] The asphalt binder specification uses the designation PG XX-YY, where XX and -YY are the high and low pavement design temperatures (in degrees Celsius), respectively. MaineDOT typically uses PG 64-28 binder for roadway types and traffic volumes similar to SR-6/SR-15/SR-16.

[41] Based on the bid price for the MaineDOT project number: STP-9200(100)X.

[42] Source: WSP | Parsons Brinckerhoff.

[43] ESAL is a measure used to develop a summary statistic of loading from a mixed stream of traffic (i.e., light duty and heavy duty vehicles). Loading from the mixed stream is expressed as the equivalent number of repetitions of one 18,000 pound (18 kilo pound or kip) standard axle load with dual tires.

[44] Mallick et al, 2006.

The remainder of this section discusses the estimation of the allowable traffic of the in-place pavement--an important consideration for subsequent climate related analyses. First, a description of how this is calculated without consideration of frost heave is presented. The following sub-section then adds in the consideration of frost heave to show its implications.

Allowable Traffic under Base Case Design with No Frost Heave Consideration

The research team conducted an analysis to determine the performance of the existing roadway in terms of allowable traffic ESALs that the base case pavement can withstand before reaching the minimum acceptable condition. The 1993 AASHTO Guide for Design of Pavement Structures, the MaineDOT adopted approach for flexible pavement design, was used for this analysis. [45] The Guide uses the concept of serviceability rating to indicate how well travelers are being served by the road. Though it is inherently a subjective index, the serviceability rating bring in the traveler's perspective of performance. The serviceability indices range from five (i.e. perfectly smooth) to zero (essentially impassable), and can be determined objectively using measures of pavement structural deterioration and surface deformation, such as pavement roughness, faulting, cracking, patching and rut depth.

The recommended serviceability index for a typical new pavement, referred to as initial serviceability[46] (p_0), is 4.5.[47] For a rural two-lane highway like SR-6/SR-15/SR-16, the recommended serviceability index at which the pavement would require renewal (i.e. terminal serviceability,[48] p_t) is 2.5.[49] The pavement serviceability is expected to decline from the initial to terminal value due to repeated applications of axle loads, degradation of pavement and subgrade materials, and environmental factors, including frost-heave.

From the pavement design perspective, a pavement structure is designed for the loss in serviceability (ΔPSI); in other words, the difference between the initial serviceability index (p_0) and the terminal serviceability index (p_t):

$$\Delta PSI = p_0 - p_t = 4.5 - 2.5 = 2.0$$

The assumed loss in serviceability, ΔPSI of 2.0, will be considered adequate to cover the loss in performance due to traffic and pavement damage over the design life of a pavement structure.

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[45] MaineDOT Highway Design Manual, 2007.

[46] Initial serviceability is the highest practicable index after new pavement construction, reconstruction, rehabilitation, or resurfacing.

[47] Constructing a pavement with a serviceability index of "perfect" 5.0 is not practicable in real-world construction.

[48] Terminal serviceability is the lowest serviceability index that will be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary and generally varies with the importance or functional classification of the pavement.

[49] At a terminal serviceability of 2.5 the pavement would have significant surface distresses (e.g. cracking but not potholes) and be barely manageable for high-speed traffic.

Per the AASHTO 1993 method, the number of 18-kilopound (kip) single-axle load applications (W_{18}) the pavement can withstand before reaching a final serviceability index value (time to final serviceability index = t) is calculated using the following formula:

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

Where,

- ΔPSI is the serviceability loss (considering no frost heave) = 2.0
- Normal deviate (Z_R) = -1.282 at 90 percent design reliability[50]
- Subgrade resilient modulus = 10,500 pounds per square inch
- Effective roadbed soil resilient modulus (M_R)[51] = 8,000 pounds per square inch
- Overall design standard deviation (S_o)[52] = 0.45
- Structural Number[53] (SN) for the existing pavement cross-section which is calculated as follows:

$$SN = a_1 * D_1 * m_1 + a_2 * D_2 * m_2$$
$$SN = 0.44*8.0*1.0 + 0.09*21.0*1.0 = 5.41$$

Where,

- Structural coefficient of AC (a_1)= 0.44
- AC thickness (D_1)= 8 inches
- Structural coefficient of subbase (a_2) = 0.19
- Subbase thickness (D_2)= 21 inches
- Drainage coefficients[54] (m_1 and m_2) = 1[55]

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[50]Design reliability is the probability that a given pavement design will last for the anticipated design life.

[51] Effective roadbed soil resilient modulus is an equivalent modulus that would result in the same damage if seasonal modulus values were actually used. More details on the computation can be found in the 1993 AASHTO Guide for Design of Pavement Structures.

[52] Standard deviation is used to account for variability in design, construction and traffic inputs.

[53] Structural Number (SN) is an index used in structural design of flexible pavements that indicates the combined effects of thicknesses of various layers above the subgrade soil, their material properties, and drainage quality.

[54] A drainage coefficient of 1.0 assumes that the quality of drainage provided during construction will be in fair to good condition over its lifetime and the pavement system is exposed to saturation for at least three months in a year.

[55] The gravel materials typically available in Maine have high fines content and low permeability. The use of this material in subbase, as MaineDOT currently uses, may result in poor drainability of water that percolates through the pavement layers. In pavement designs, MaineDOT uses a drainage coefficient of 1.0 to account for the quality of drainage and its effect of pavement performance. However, it is unknown whether the MaineDOT's recommended drainage coefficient of 1.0 adequately considers the combined effects of poor drainage

Based on the above calculations, the existing pavement section, as constructed, has the capacity of carrying 25.4 million 18-kip ESALs before the pavement reaches the terminal serviceability condition; however, since the soil profile in this region is prone to frost heave, the actual value may be different. The impacts of additional serviceability loss due to frost heave are discussed in the following section.

Allowable Traffic under Base Case Design after Considerations for Frost Heave

The serviceability loss assumed in the above analysis did not consider expected reduction in pavement performance due to environmental factors, such as frost heave and swelling. The additional serviceability loss that the pavement will incur over its design life due to frost heave should be compensated for in the design using a higher serviceability loss value, as follows:

$$\Delta PSI = p_0 - p_t - p_{FH}$$

Where, p_{FH} is the additional serviceability loss due to frost heave.

The steps involved in determining the change in serviceability loss due to frost heave are as follows.[56] All of the variables are also summarized in Table 4.

1. **Determine the frost heave rate (Ø)** specified in the frost heave model of the AASHTO 1993 Guide in millimeters per day. The predominant soil types at the Guilford site are sandy silt and silt, whose frost heave rates are 0.28 inches per day and 0.59 to 0.79 inches per day, respectively. A value of 0.79 inches per day was used for silt soils.
2. **Estimate the probability of frost heave (P_F).** While the recommended range of probability is 25 to 75 percent, there is no guidance available in the 1993 AASHTO Guide, MaineDOT Highway Design Manual, or the literature generally, on how to arrive at the preferred probability. Therefore, the selection of the probability value is at the discretion of the designer based on his/her confidence to control frost heave. By taking both the prevalence of frost susceptible soils and mitigation measures (e.g. provision of subsurface drainage and a thicker layer of granular material to protect the frost susceptible materials) into consideration, an input value of 33 percent was deemed appropriate for the given location. While a probability value of 50 percent indicates an equal chance that the frost heave will or will not occur, a lower probability of 33 percent indicates a greater likelihood that frost heave occurrence will be controlled.

characteristics of the subbase materials and the amount of time the pavement structure is exposed to moisture levels approaching saturation.

[56] AASHTO, Guide for Design of Pavement Structures, 1993.

3. **Estimate the depth of frost penetration[57] (FD)** using the design annual freezing index[58] of 2,114 degree Fahrenheit[59] (see Step 4 for information on how this value was selected) and the graph shown in Figure 14. The estimated depth of frost penetration is 5.25 feet or 63 inches.

4. **Determine the maximum serviceability loss due to frost heave (ΔPSI_{MAX})** based on the quality of drainage. For fair quality of drainage, the 1993 AASHTO Guide for Design of Pavement Structures recommends a value of 0.3. Given this, ΔPSI_{MAX} = 0.3* FD = 1.57

5. **Determine the change in serviceability loss due to frost heave (p_{HF})** using the following formula:

$$p_{FH} = \frac{P_F}{100} * \Delta PSI_{MAX} * 1 - e^{-0.02*20\ years*\emptyset}$$

The range of expected changes in serviceability loss due to frost heave are presented in Table 4 for both sandy silt and silt soils. Since sandy silt soils are the most predominant soil type in the Guilford area, the value of 0.49 was selected to incorporate the additional serviceability loss in further design analysis.

The adjusted loss in serviceability for use in the design is calculated as follows:

$$\Delta PSI = p_0 - p_t - p_{FH} = 4.5 - 2.5 - 0.49 = 1.51$$

The value of ΔPSI indicates how much serviceability loss can be caused by traffic and "normal" material degradation, design, and construction factors before the pavement reaches the PSI of 2.5. The ΔPSI changes from 2.0 to 1.51 (approximately a 25 percent reduction) due to frost heave. This change indicates that only 75 percent of the total serviceability loss (1.51 of 2.0) can be caused by traffic and other normal factors. Assuming that there is no change in typical levels of design errors, material and construction workmanship defects, a decrease in ΔPSI indicates that the pavement can carry lesser traffic over its performance period, and vice-versa.

[57] The method adopted by the 1993 AASHTO Guide for Design of Pavement Structures to estimate the depth of frost penetration is recommended for determining the change in serviceability due to the empiricism behind the method; however, later in the analyses, a different method will be discussed. The key inputs in the AASHTO 1993 method are the freezing index of the location and the soil types.

[58] The annual freezing index is the accumulation of daily freezing indices for all days of a year. The daily freezing index is the difference between 32° Fahrenheit and daily mean temperature. The daily index becomes zero when the daily mean temperature exceeds 32° Fahrenheit.

[59] Degree days are the accumulated total of the positive or negative differences between daily temperatures and a base temperature (in the case of the freezing index, 32° Fahrenheit).

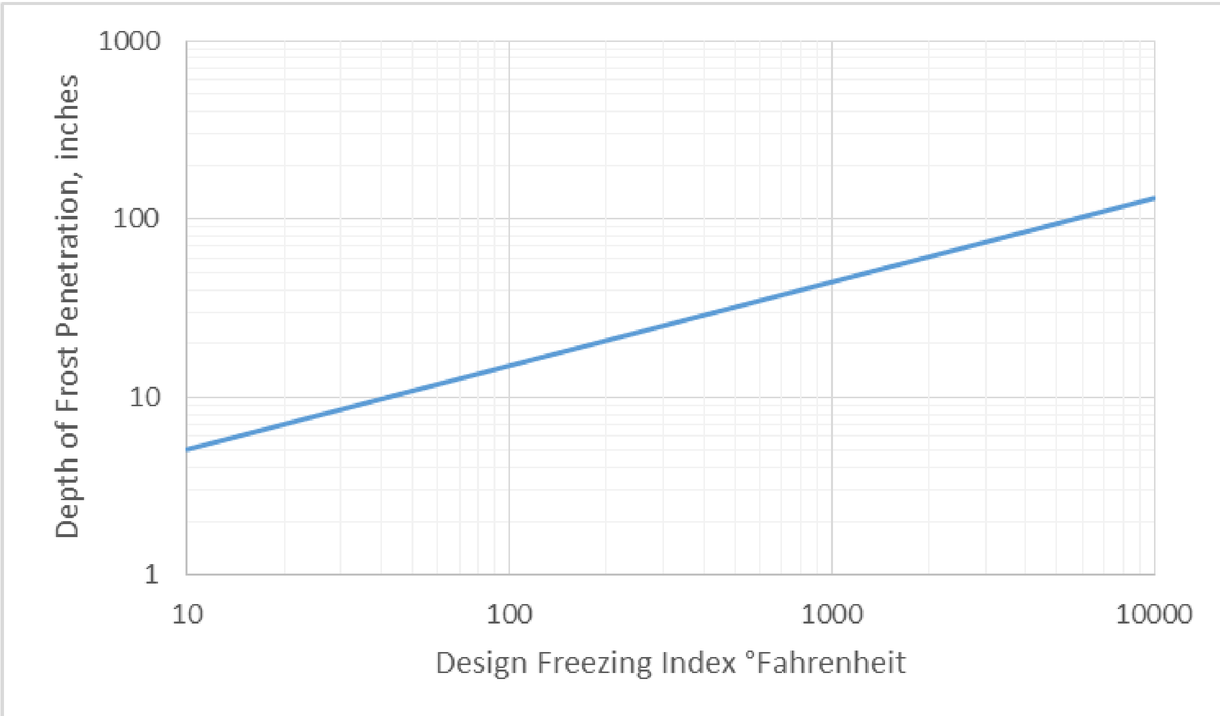

Figure 14: Estimation of Frost Penetration using the 1993 AASHTO Guide for Serviceability Loss Estimates.[60]

Table 4: Change in Serviceability Loss Due to Frost Heave for the Base Case Design.

Design Period (years)	Frost Heave Probability	Design Freezing Index	ΔPSI_{max}	Sandy Silt Soils		Silt Soils		Design ΔPSI
				Φ (inch/day)	ΔPSI_{FH}	Φ (inch/day)	ΔPSI_{FH}	
Base case design	33	2114	1.57	0.28	0.49	0.79	0.52	1.51

Before the adjustments for serviceability loss due to frost heave potential, the number of 18-kip single-axle load applications that can be allowed over the pavement before failure[61] was calculated to be 25.4 million but after accounting for serviceability loss due to frost heave potential, the allowable number of 18-kip load vehicles before failure is reduced to 14 million. In other words, the existing pavement section was built to serve 14 million ESALs of traffic while maintaining acceptable condition under potential frost heave conditions during the 20-year performance period.

[60] Image source: AASHTO Guide for Design of Pavement Structures, 1993.

[61] Recall that the AASHTO 1993 pavement design method defines the terminal serviceability index as the failure criterion, which is the minimum acceptable condition before resurfacing, restoration or reconstruction is necessary.

Step 3: Identify Climate Stressors

Climate Stressors

- Temperature
- Precipitation

Most pavement materials are sensitive to changes in climate factors, in particular temperature and precipitation. For example, the stiffness of asphalt concrete is fundamentally dependent on temperature while the soil subgrade stiffness is defined by the moisture content of the soil. Changes in temperature and precipitation also induce changes in other climate-related variables, such as the depth of frost penetration, freeze-thaw cycles, ground water table levels, etc. Together, these factors affect the fundamental properties of pavement materials and subgrade, control their structural responses under or in the absence of traffic loading, and contribute to distress accumulation and smoothness deterioration of pavements.

Table 5 presents a list of common distresses for flexible pavements that are known to be affected by temperature, moisture, and related environmental factors. While some of these distresses listed in the table can be addressed in the design process, many of these distresses are caused by deficiencies in materials, construction workmanship, and maintenance and are not covered in this case study--this analysis focuses on those climate-related distresses that are the most influenced by pavement design (indicated by asterisks in the table).

Table 5: Flexible Pavement Distresses Affected by Temperature and Precipitation.

Flexible Pavement Distress	Temperature	Precipitation (Moisture)
Fatigue Cracking*	✓	✓
Rutting*	✓	✓
Non-Wheel Path Longitudinal Cracking	✓	
Transverse Cracking	✓	
Smoothness*	✓	✓
Reflective Cracking[62]	✓	✓
Settlement/Grade Depression	✓	✓
Swell/Upheaval*	✓	✓
Raveling/Weathering[63]	✓	✓
Pot Hole	✓	✓
Stripping[64]	✓	✓
Block Cracking[65]	✓	
Edge Cracking		✓
Pumping[66]	✓	✓
Slippage Cracks	✓	✓
Corrugation[67] and Shoving[68]	✓	✓
Bleeding[69]	✓	
Delamination[70]		✓

*Distresses covered in this case study

In order to determine the impact of climate change on the pavement distress, this study focused on the projected changes in the following specific climate stressors:

- Temperature
  - Mean temperature
  - Annual average maximum temperature
  - Annual average minimum temperature
  - Degree days greater than 50° Fahrenheit
  - Low pavement temperature
  - Annual freezing index

[62] Reflective cracks are cracks that reflect directly over the underlying cracks or joints.
[63] Raveling is wearing away of the pavement surface caused by the dislodging of aggregate particles and loss of asphalt binder.
[64] Stripping is the loss of bond between aggregates and asphalt binder typically caused by the interaction of moisture with some minerals in aggregates.
[65] Block cracking is a pattern of cracks that divides the pavement into approximately rectangular pieces.
[66] Pumping is erosion of fine materials from support layers, accompanied with water seepage, through cracks or joints.
[67] Corrugation is the ripples of distortion on pavement surface.
[68] Shoving is a localized distortion on pavement surface that typically occurs at traffic stops.
[69] Bleeding is the excess asphalt binder occurring on the pavement surface.
[70] Delamination is loss of bonding between two pavement layers.

- Number of freezing days[71]

- Annual precipitation

These stressors can also be used to derive climate-related variables such as the depth of frost penetration and TMI. Any change in these climate stressors could have positive and/or negative impacts on the structural and functional performance of pavements.

Step 4: Develop Climate Scenarios

Historically, the Guilford area experiences a humid continental climate of cold snowy winters and warm summers with an annual average mean temperature of 41.5° Fahrenheit and average annual total precipitation of 43.8 inches.[72] Significant levels of precipitation are observed throughout the year with average monthly precipitation ranging between 2.8 and 4.4 inches. This section presents a description of the climate change scenarios used in this case study, followed by discussion of the projected climate change variables.

> **Climate Data Overview**
>
> **Level of Detail:** Developed detailed projections of future temperature and precipitation.
>
> **Data Source:** Global Historical Climatology Network (GHCN), U.S. Bureau of Reclamation (2013) which provides peer-reviewed statistically downscaled data of the World Climate Research Programme's Coupled Model Intercomparison Project 5 **Scenarios:** Representative Concentration Pathways (RCPs) 4.5, 6.0 and 8.5.

Climate Change Scenarios

The research team used three climate change scenario based on plausible trajectories of future global greenhouse gas emissions (GHGs), referred to as representative concentration pathways (RCPs) (van Vuuren et al, 2011) . The RCPs describe how global society may evolve in its use of fossil fuels, technology, population growth, etc. and the resulting GHG concentration levels in the atmosphere. The three scenarios used include:

- **RCP 4.5** with a radiative forcing[73] of 4.5 watts per square meter indicating a low to moderate increase in the total greenhouse gas concentration levels in the atmosphere.
- **RCP 6.0** with a radiative forcing of six watts per square meter indicating a moderate increase in the total GHG concentration levels in the atmosphere

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[71] The number of days with mean temperatures less than 32° Fahrenheit (i.e. a non-zero daily freezing index).

[72] Source: The Weather Channel Monthly, Average/Record Temperatures and Average Precipitation. Accessible at: http://www.weather.com/weather/monthly/l/04443:4:US.

[73] Radiative forcing causes a change in the energy balance leading to a net warming or cooling of climate. For example, a change in the concentration of carbon dioxide or the output of the sun can cause a radiative forcing (IPCC 2014 WGIII).

- **RCP 8.5** with a radiative forcing of 8.5 watts per square meter indicating a high or unabated increase in the total GHG concentration levels in the atmosphere

Figure 15 presents the equivalent carbon dioxide[74] concentrations and radiative forcing trajectories of different RCPs. As shown in the graphs, RCP 4.5 annual GHG emissions rise quickly but then stabilize with time. After about 2060, RCP 6.0 GHG emissions exceed RCP 4.5 GHG emissions and then stabilize. Finally RCP 8.5 GHG emissions rise steadily at a greater rate compared to the other RCPs and do not stabilize at the end of the century.

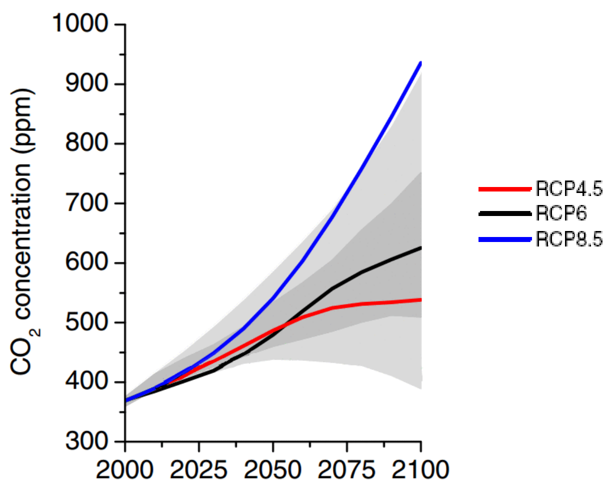

Figure 15: Equivalent Carbon Dioxide (CO_2) Emission and Radiative Forcing Trajectories of Different Representative Concentration Pathways.[75]

It is important to note that (1) the global radiative forcing does not reflect local changes in precipitation (i.e. more global radiative forcing does not necessarily consistently indicate more or less precipitation for a given area) and (2) these figures are at the global scale and there can be variations from these relationships at the local scale (e.g. if one RCP scenario suggests a location will have a higher level of particulate matter that could contribute to cloudiness and/or rainfall, it could reduce the potential warming for that area even though that scenario may be warmer than others globally).

Climate Change Projections

This section discusses the:

- General temperature and precipitation projections for the study area,

[74] Carbon dioxide equivalent is a measure used to compare the emissions from various greenhouse gases based upon their global warming potential.
[75]Source: IPCC Data Distribution Center. Scenario Process for AR5. http://sedac.ipcc-data.org/ddc/ar5_scenario_process/RCPs.html. The original chart included RCP 2.6, but the research team considered this RCP too unrealistic to include in this analysis.

- Calculation of the annual freezing index and length of freezing season,
- Derivation of the maximum depth of frost penetration,
- Calculation of TMI, and
- Design high and low temperatures for use in specifying an asphalt binder.

General Temperature and Precipitation Projections

For the future climate projections, the research team used publically available statistically downscaled[76] data provided by the U.S. Bureau of Reclamation (USBR).[77] The USBR's website provides downscaled data from the World Climate Research Programme's (WCRP) Coupled Model Intercomparison Project 5 (CMIP5) that was used to inform the Intergovernmental Panel on Climate Change (IPCC) Fifth Assessment reports. These simulations, originally available at a spatial resolution around one degree (approximately 4,760 square miles),[78] have been statistically downscaled to 1/8 degree resolution grid cells (approximately 55.6 square miles) for the United States by USBR. Daily values from 1950 to 2099 were downloaded for minimum temperature, maximum temperature, and precipitation from 11 global climate models, using four USBR grid cells[79] and the three RCPs (see Table 6 below).

Table 6: Summary of Global Climate Models and Scenarios Used.

Global Climate Models	Scenarios
▪ **bcc-csm1-1** ▪ **ccsm4** ▪ **gfdl-esm2g** ▪ **gfdl-esm2m** ▪ **ipsl-cm5a-lr** ▪ **ipsl-cm5a-mr** ▪ **miroc-esm** ▪ **miroc-esm-chem** ▪ **miroc5** ▪ **mri-cgcm3** ▪ **noresm1-m**	▪ RCP4.5 ▪ RCP6.0 ▪ RCP8.5

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[76] Statistical downscaling is a technique for taking projected climate data at a coarser resolution and developing projections at much finer spatial resolution so that local conditions are represented. This technique is done by considering the relationship between local historical climate observations and hindcast GCM results.
[77] Source: Downscaled CMIP 3 and CMIP 5 Climate and Hydrology Projections, USBR, 2013. http://gdo-dcp.ucllnl.org/downscaled_cmip_projections/dcpInterface.html.
[78] The one degree value is approximate because each climate model's spatial resolution varies and may be smaller or larger than one degree.
[79] Four adjacent grid cells were chosen based on the center of the study location with a latitude of 32.974108° North and a longitude of 97.276382° West.

For each climate model and RCP scenario, the following approach was coded in Excel to arrive at annual mean temperature and precipitation:[80]

- Calculated the daily average temperature from the minimum daily temperature and maximum daily temperature for every day from 1950 to 2099.
- Averaged the daily average temperature across four grid cells for every day from 1950 to 2099, and averaged the daily average precipitation across four grid cells for every day from 1950 to 2099.
- Calculated the monthly average temperature and total monthly precipitation for each year.

The research team then averaged the monthly average temperature and total monthly precipitation across the models to obtain the ensemble averages (i.e. average across all climate models for each RCP).

Table 7 presents a twenty-year summary of ensemble averages of annual mean and extremes of maximum and minimum temperatures for the study site. Consistent with the largest increase in future emissions of greenhouse gases, the RCP 8.5 scenario indicates a significant increase in mean temperature, approximately 11° Fahrenheit, over the next 85 years. On the other hand, both the RCP 4.5 and RCP 6.0 scenarios indicate a more moderate increase in mean temperature, approximately 6° to 7° Fahrenheit, over the same period.

[80] Unlike other TEACR case studies, this analysis did not use the FHWA's Coupled Model Intercomparison Project (CMIP) Climate Data Processing Tool since only daily and monthly averages of temperature and precipitation were needed.

Table 7: Twenty-Year Summary of Temperature for the Guilford Area under Different RCP Scenarios (in °F).

Year	RCP 4.5			RCP 6.0			RCP 8.5		
	Highest Tmax	Tmean	Lowest Tmin	Highest Tmax	Tmean	Lowest Tmin	Highest Tmax	Tmean	Lowest Tmin
1950-69	83.3	41.0	-4.6	83.2	40.9	-7.4	83.3	41.0	-7.8
1960-79	83.7	40.9	-4.8	83.6	40.8	-7.4	83.7	40.9	-7.8
1970-89	83.7	41.2	-4	83.6	41.1	-7.7	83.7	41.2	-7.8
1980-99	84.5	41.8	-3.9	84.3	41.7	-7.7	84.5	41.8	-7.8
1990-09	84.5	42.4	-4.2	84.3	42.4	-7.5	84.8	42.4	-7.3
2000-19	85.5	43.1	-2.1	86.1	42.9	-4.0	86.1	43.2	-4.2
2010-29	86.3	43.7	-1.8	86.1	43.4	-3.4	86.2	43.9	-3.3
2020-39	87.6	44.4	0.7	85.7	43.9	-4.8	87.7	44.8	-1.4
2030-49	88.0	45.1	1.1	86.4	44.5	-4.8	88.3	45.8	-1.4
2040-59	88.9	45.8	1.2	87.2	45.2	-2.0	89.2	46.9	-1.4
2050-69	88.9	46.3	2.5	88.1	45.7	-1.7	90.4	48.1	3.1
2060-79	90.0	46.6	4.3	88.4	46.4	-3.6	91.7	49.3	4.6
2070-89	90.0	46.8	4.1	89.1	47.3	-3.6	93.1	50.6	4.6
2080-99	89.0	47.0	3.5	91.0	48.0	2.6	94.6	51.8	8.5

Figures 16 through 18 present the annual mean temperature trends projected under RCP 4.5, RCP 6.0 and RCP 8.5, respectively. The blue-colored lines on the figures indicate the overall trend of the ensemble averages of annual mean temperature while the upper and lower bounds of the ribbon indicates the overall range in the projections across the 11 different global climate models. Irrespective of the significant variability among projections made by different climate models, an overall warming trend is projected to be observed over the next 85 years.

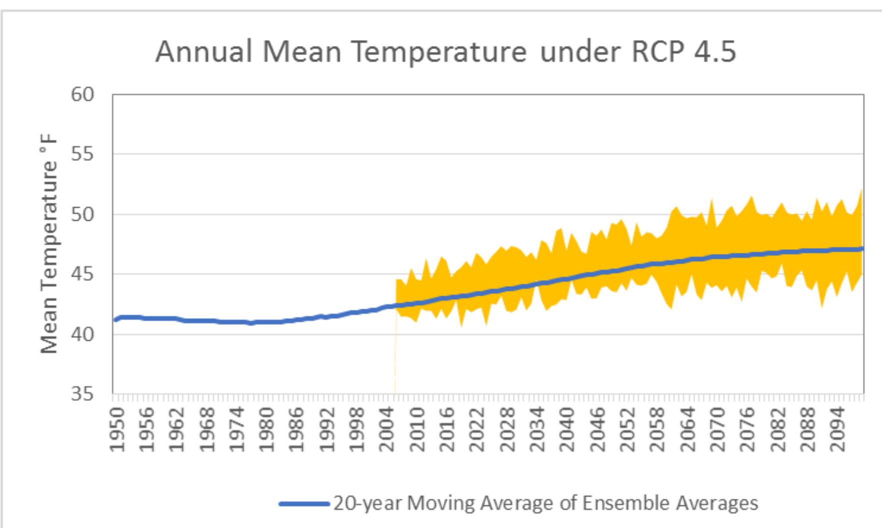

Figure 16: Annual Mean Temperature Projections for the Guilford Area under RCP 4.5.

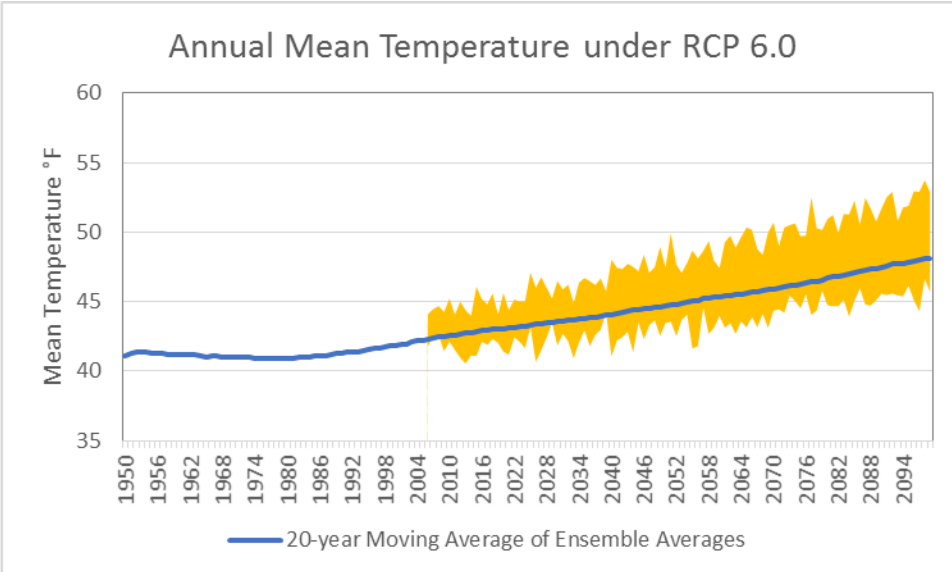

Figure 17: Annual Mean Temperature Projections for the Guilford Area under RCP 6.0.

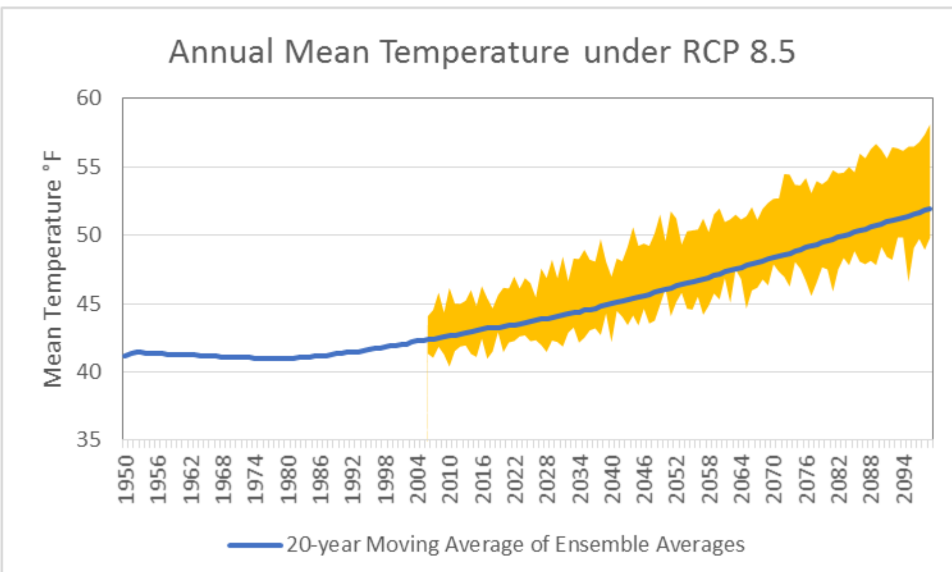

Figure 18: Annual Mean Temperature Projections for the Guilford Area under RCP 8.5.

Table 8 presents a twenty-year summary of ensemble averages of annual total precipitation under different scenarios. Figure 19, Figure 20, and Figure 21 present the annual total precipitation graphically under RCP 4.5, RCP 6.0, and RCP 8.5, respectively. As with the temperature graphics, the blue-colored lines on the figures indicate the overall trend of the ensemble averages of annual mean precipitation while the upper and lower bounds of the ribbon indicate the overall range in the projections across the 11 different global climate models.

Table 8: Twenty-Year Averages of Annual Total Precipitation for the Guilford Area under Different RCP Scenarios.

Year	Mean Annual Precipitation (Inches)		
	RCP 4.5	RCP 6.0	RCP 8.5
1950-69	43.1	43.2	43.1
1960-79	43.2	43.3	43.2
1970-89	43.4	43.5	43.4
1980-99	44.1	44.2	44.1
1990-09	44.7	44.8	44.8
2000-19	45.1	45.2	45.1
2010-29	45.2	45.6	45.5
2020-39	46.1	46.2	46.2
2030-49	46.4	46.7	46.8
2040-59	46.3	47.5	47.5
2050-69	46.7	47.0	47.6
2060-79	47.3	46.8	48.2
2070-89	47.6	47.7	48.6
2080-99	48.1	48.0	49.1

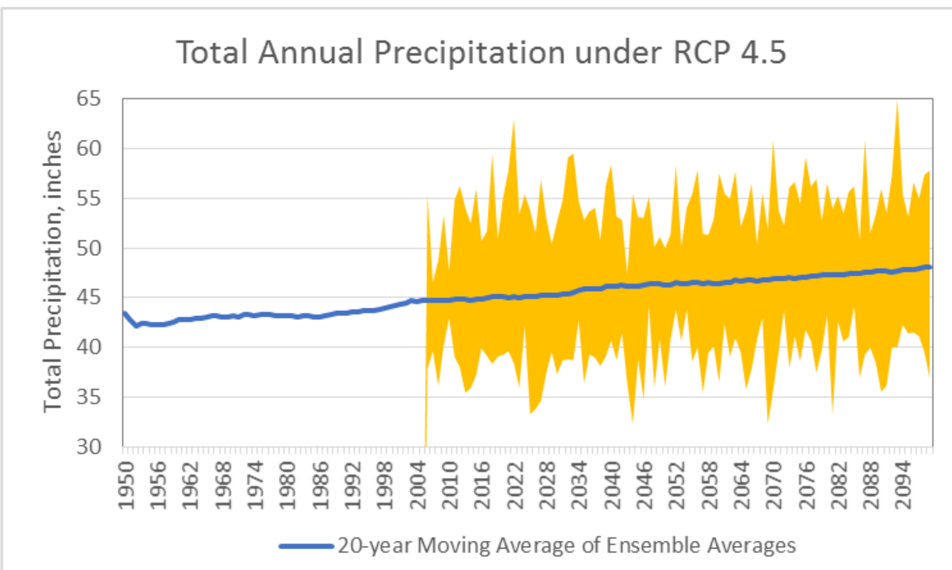

Figure 19: Annual Total Precipitation Projections for the Guilford Area under RCP 4.5.

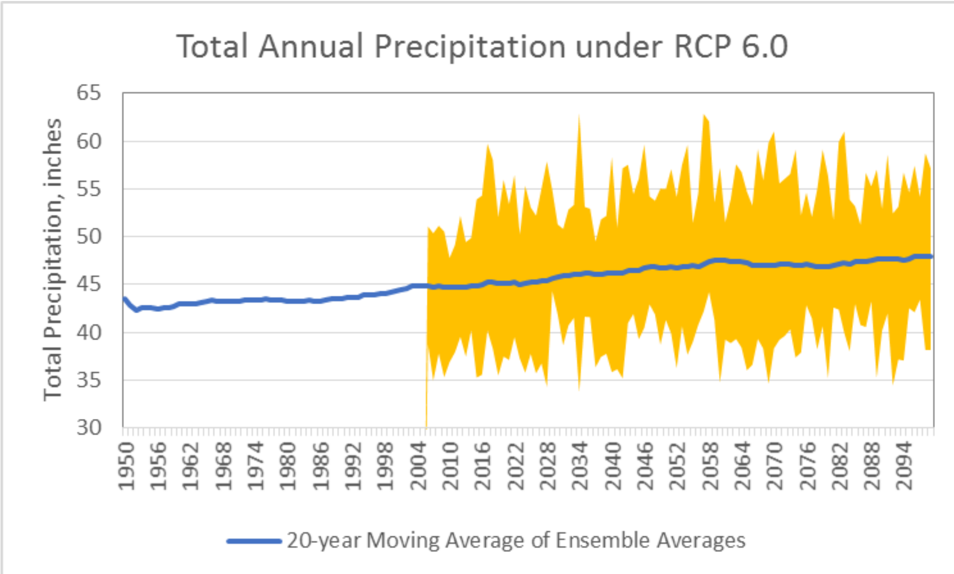

Figure 20: Annual Total Precipitation Projections for the Guilford Area under RCP 6.0.

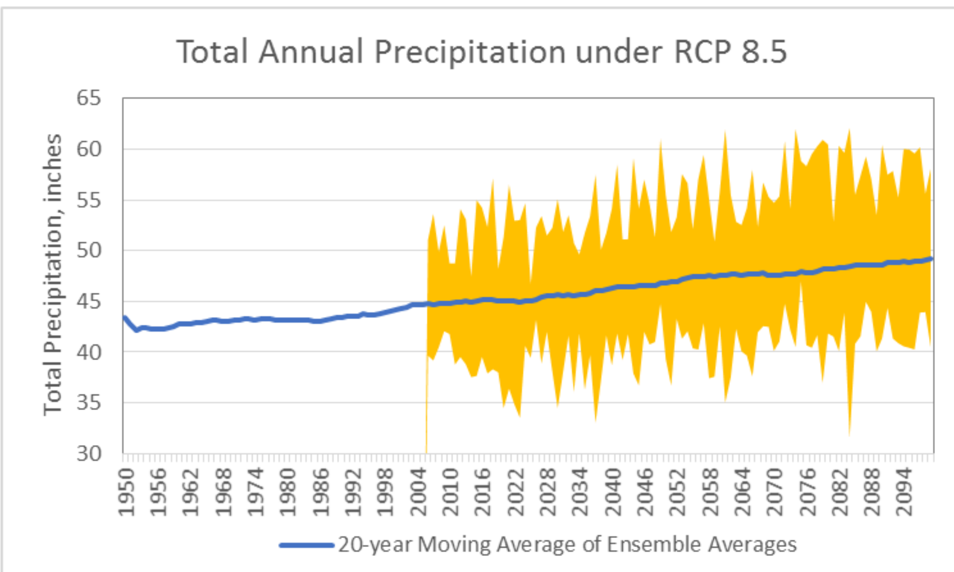

Figure 21: Annual Total Precipitation Projections for the Guilford Area under RCP 8.5.

The figures indicate that the annual total precipitation is expected to increase by 4 to 6 inches over the course of the 21st century; however, significant variability in the projections of individual global climate models indicate a lack of consensus whether there will actually be a wetting trend in the future.

As mentioned earlier, Guilford receives almost equal amounts of precipitation throughout the year with June and December being the wettest months and July and January receiving the lowest precipitation. As shown in Table 9 through Table 11, climate change is not projected to have an impact on the proportion of precipitation received in each month.

Table 9: Monthly Distribution of Annual Total Precipitation for the Guilford Area under RCP 4.5.

Month	Percent of Annual Total Precipitation by Each Month				
	1980-1999	2020-2039	2040-2059	2060-2079	2080-2099
January	7.6%	8.0%	8.1%	7.9%	8.0%
February	6.9%	7.1%	7.1%	7.4%	7.7%
March	8.1%	7.8%	8.2%	8.2%	8.2%
April	7.3%	7.0%	7.0%	7.4%	7.4%
May	8.3%	8.1%	8.3%	8.0%	8.0%
June	7.8%	8.4%	8.2%	8.3%	8.4%
July	8.8%	8.5%	8.4%	8.5%	8.1%
August	8.2%	8.1%	8.4%	7.9%	7.8%
September	8.4%	8.5%	7.9%	7.9%	7.6%
October	9.7%	9.2%	9.1%	8.8%	8.8%
November	9.4%	9.7%	9.7%	9.7%	10.1%
December	9.6%	9.3%	9.6%	10.1%	9.8%

Table 10: Monthly Distribution of Annual Total Precipitation for the Guilford Area under RCP 6.0.

Month	Percent of Annual Total Precipitation by Each Month				
	1980-1999	2020-2039	2040-2059	2060-2079	2080-2099
January	7.6%	8.0%	8.1%	7.9%	8.0%
February	6.9%	7.1%	7.1%	7.4%	7.7%
March	8.1%	7.8%	8.2%	8.2%	8.2%
April	7.3%	7.0%	7.0%	7.4%	7.4%
May	8.3%	8.1%	8.3%	8.0%	8.0%
June	7.8%	8.4%	8.2%	8.3%	8.4%
July	8.8%	8.5%	8.4%	8.5%	8.1%
August	8.2%	8.1%	8.4%	7.9%	7.8%
September	8.4%	8.5%	7.9%	7.9%	7.6%
October	9.7%	9.2%	9.1%	8.8%	8.8%
November	9.4%	9.7%	9.7%	9.7%	10.1%
December	9.6%	9.3%	9.6%	10.1%	9.8%

Table 11: Monthly Distribution of Annual Total Precipitation for the Guilford Area under RCP 8.5.

Month	Percent of Annual Total Precipitation by Each Month				
	1980-1999	2020-2039	2040-2059	2060-2079	2080-2099
January	7.6%	8.0%	8.1%	7.9%	8.0%
February	6.9%	7.1%	7.1%	7.4%	7.7%
March	8.1%	7.8%	8.2%	8.2%	8.2%
April	7.3%	7.0%	7.0%	7.4%	7.4%
May	8.3%	8.1%	8.3%	8.0%	8.0%
June	7.8%	8.4%	8.2%	8.3%	8.4%
July	8.8%	8.5%	8.4%	8.5%	8.1%
August	8.2%	8.1%	8.4%	7.9%	7.8%
September	8.4%	8.5%	7.9%	7.9%	7.6%
October	9.7%	9.2%	9.1%	8.8%	8.8%
November	9.4%	9.7%	9.7%	9.7%	10.1%
December	9.6%	9.3%	9.6%	10.1%	9.8%

Annual Freezing Index and Length of Freezing Season

As discussed in Step 2, the annual freezing indices are necessary to estimate the depth of frost penetration at a given location and serviceability loss due to frost heave. The freezing index is an indicator of the intensity and duration of winter experienced at a given location. According to MaineDOT's Highway Design Manual, the historical annual freezing index at the Guilford site is approximately 2,114 degree Fahrenheit days (see Figure 22). The star on the figure shows the location of Guilford.

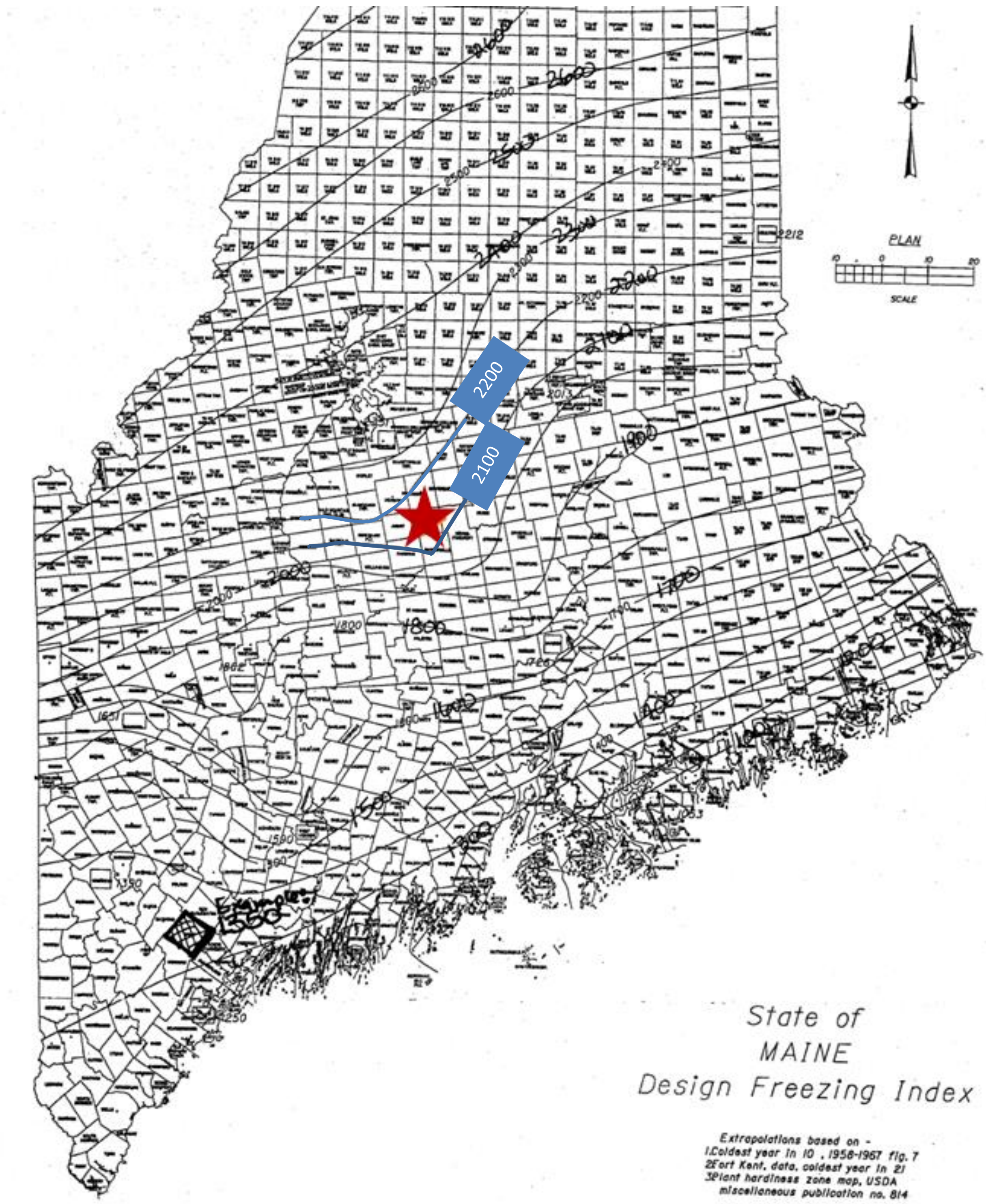

Figure 22: Historical Annual Freezing Index Values for Maine.[81]

[81] Image source: MaineDOT, 2007 (as modified).

Figure 23 presents the temporal change in the annual freezing index under different RCP scenarios. With climate change, a steady decrease in the annual freezing index is expected, albeit at different rates under different scenarios. This means that winters will be decreasing in their intensity. Under the RCP 4.5 and 6.0 scenarios, the annual freezing index will decrease by approximately 10 percent to 1,900 degree Fahrenheit days at the end of the century. Under the RCP 8.5 scenario, the annual freezing index will decrease by approximately 30 percent to 1,500 degree Fahrenheit days by end of the 21st century.

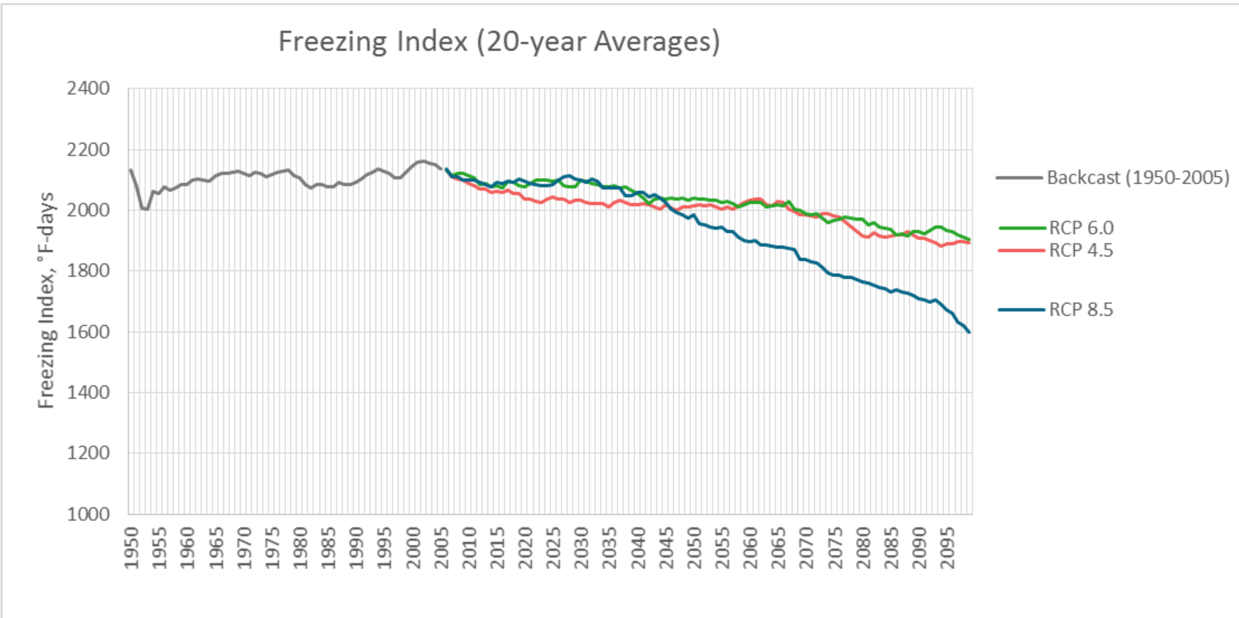

Figure 23: Annual Freezing Index for the Guilford Area under Different Climate Scenarios.

Figure 24 presents the temporal change in the average length of the freezing season (i.e. the number of days per year where the daily mean temperature is below freezing) under different RCPs. Historically, the average number of freezing days in a year was 110 (± 13 days). With climate change, towards the end of the century the annual average number of freezing days are projected to be approximately 80 (± seven days), 78 (± eight days), or 60 (± 10 days) under RCP 4.5, RCP 6.0, and RCP 8.5, respectively. Consistent with the overall temperature projections, both the annual freezing index and the number of freezing days in a year indicate significant warming trends and shorter freezing seasons in the future.

To allow for an investigation of the impacts of shorter freezing seasons on WWP and SLR policies, the temporal trends in the average number of freezing days in each month were investigated. Historically, the freezing season at the Guilford site begins in mid-November and continues through early to mid-March, including the full months of December, January, and February. However, as summarized in Figures 25 and 26, the freezing season at the Guilford site is expected to shorten gradually but steadily over time (note that RCP 4.5 and RCP 6.0 showed similar patterns and are grouped together on one figure).

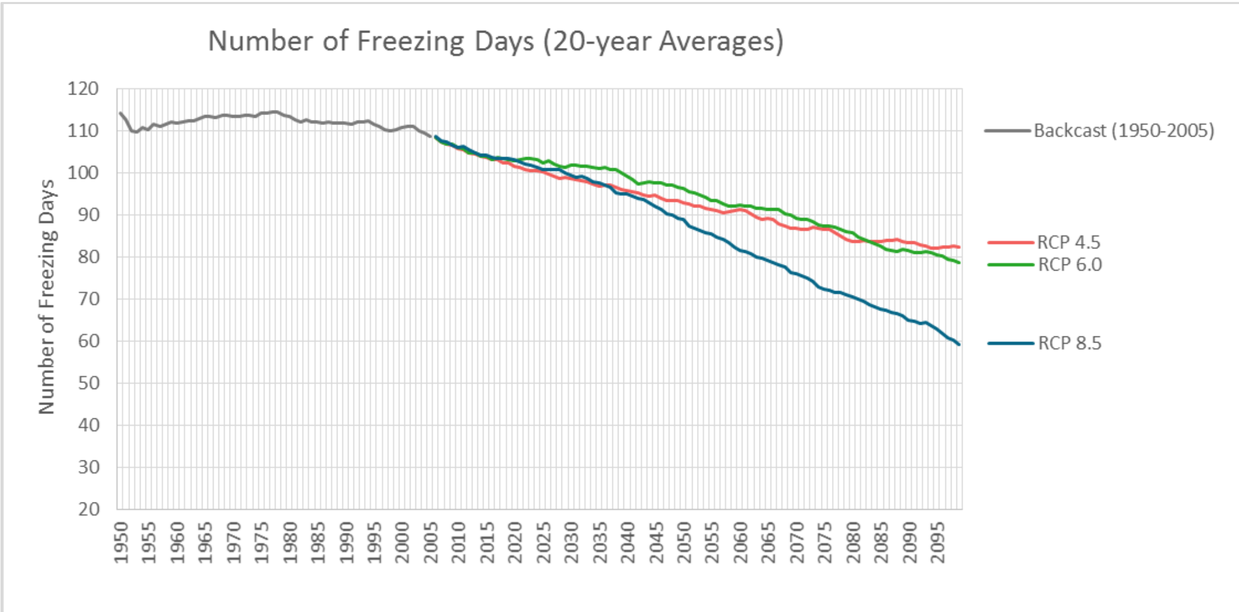

Figure 24: Annual Number of Freezing Days for the Guilford Area under Different Scenarios.

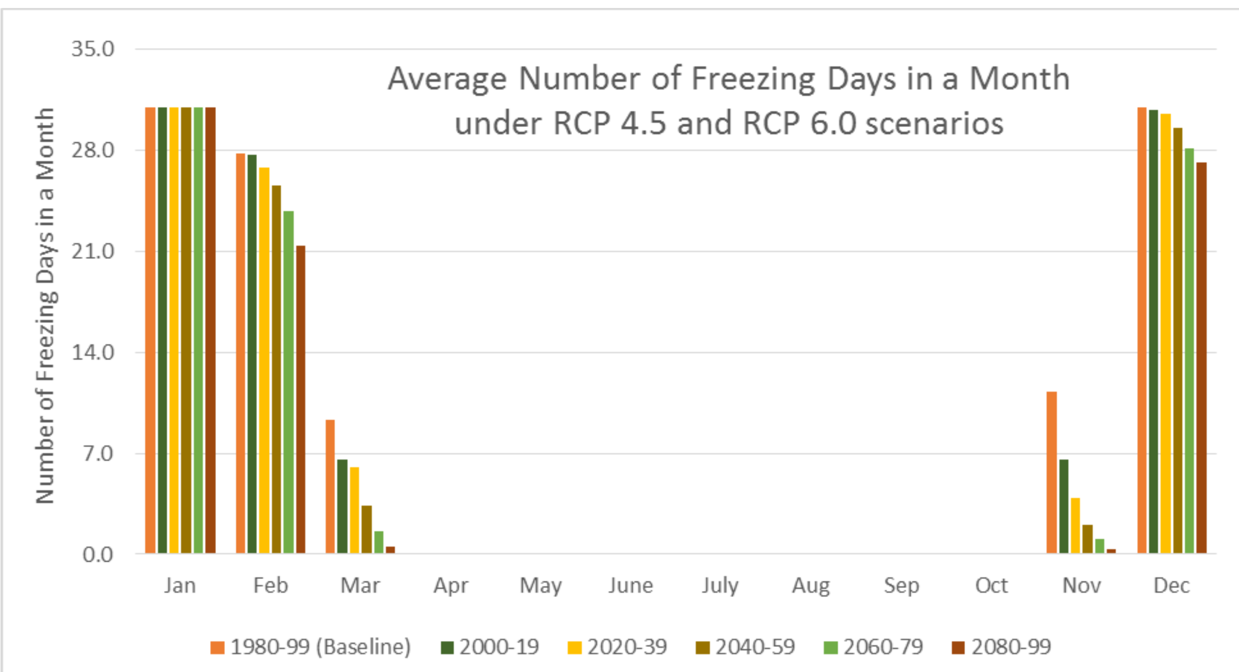

Figure 25: Average Number of Freezing Days per Month for the Guilford Area under RCP 4.5 and RCP 6.0.

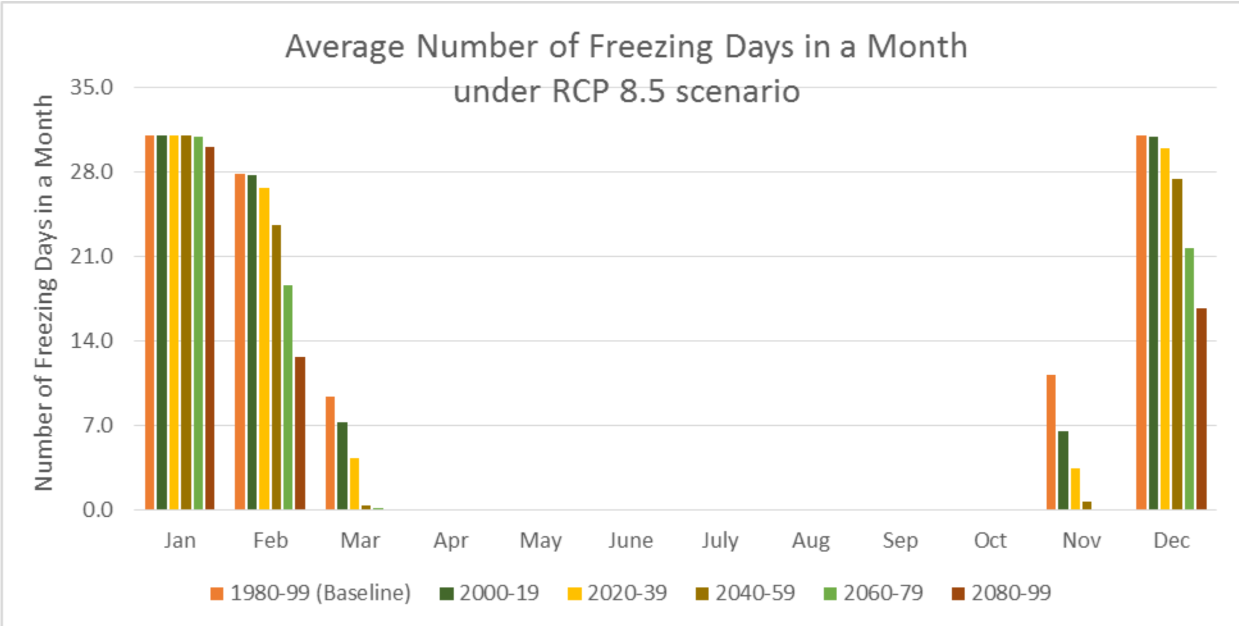

Figure 26: Average Number of Freezing Days per Month for the Guilford Area under RCP 8.5.

During the last three decades of the 21st century, the freezing season is projected to begin in early December and continue only through late-February under the RCP 4.5 and 6.0 scenarios. Under the RCP 8.5 scenario, the freezing season will be even shorter, beginning in mid-December and ending by mid-February.

Maximum Depth of Frost Penetration

The maximum depth of frost penetration was estimated empirically using a model developed by the University of Waterloo in Canada.[82] In 2006, MaineDOT conducted a research study "Mechanistic Approach to Determine Spring Load Restrictions in Maine" to investigate the applicability of various analytical methods using field data to improve the agency's procedure of determining SLRs. Under this study, MaineDOT installed frost gauges at eight test sites around central Maine and collected frost penetration data during the winter of 2006 – 2007. One of the key observations of this study was that the frost depths calculated using the University of Waterloo model aligned well with frost depths measured at the test sites.[83]

The following formula was used to calculate frost depth using the University of Waterloo model:

$$FD = 2.18\sqrt{(CFI)}$$

[82] There are many empirical models available in the literature to estimate the depth of frost penetration using soil physical and thermal properties and freezing indices. For more information, refer to Rajaei and Baladi, 2015.
[83] Marquis, 2008.

Where,

- FD = Frost depth below the pavement's surface in inches
- CFI = Cumulative freezing index (in degree Celsius days)

The CFI is an indicator of the intensity and duration of the winter season and is calculated using the following formulae:

$$CFI = \sum_{i=1}^{n} Daily\_Freezing\_Index$$

$$Daily\_Freezing\_Index = \; Max\left(0, \left(0°\mathrm{C} - \frac{(T_{max} + T_{min})}{2}\right)\right)$$

Where,

- T_{max} = Maximum daily air temperature (in Celsius),
- T_{min} = Minimum daily air temperature (in Celsius)

Similarly, the cumulative thawing index (CTI), another indicator to capture the degree of warming occurring during the freezing season, can be calculated using the following formula:

$$CTI = \sum_{i=1}^{n} Daily\_Thawing\_Index$$

$$Daily\_Thawing\_Index = \; Min\left(0, \left(\frac{(T_{max} + T_{min})}{2} - T_{ref}°\mathrm{C}\right)\right)$$

Where,

- T_{max} = Maximum daily air temperature (in Celsius),
- T_{min} = Minimum daily air temperature (in Celsius)
- T_{ref} = Reference air temperature (in Celsius) as shown in Table 12

Figure 27 presents the temporal trends of annual maximum frost depths estimated using the University of Waterloo model under different scenarios. Consistent with the temperature projections, the frost penetration into the pavement system will be shallower over the course of the 21st century. Based on the computed CFI values, the annual maximum frost depths calculated using the historical climate data averaged around 62 inches, while the annual maximum frost depths will be 44, 39, and 26 inches at the end of the century under the RCP 4.5, RCP 6.0, and RCP 8.5 scenarios, respectively.

Table 12: Reference Air Temperatures for use in Daily Freezing Index Calculation.

Date	Reference Air Temperature (Tref)	
	°Fahrenheit	°Celsius
January 1 – January 31	32	0
February 1 – February 7	29.3	-1.5
February 8 – February 14	28.4	-2
February 15 – February 21	27.5	-2.5
February 22 – February 29	26.6	-3
March 1 – March 7	25.7	-3.5
March 8 – March 14	24.8	-4
March 15 – March 21	23.9	-4.5
March 22 – March 28	23	-5
March 29 – April 4	22.1	-5.5
April 5 – April 11	21.2	-6
April 12 – April 18	20.3	-6.5
April 19 – April 25	19.4	-7
April 26 – May 2	18.5	-7.5
May 3 – May 9	17.6	-8
May 10 – May 16	16.7	-8.5
May 17 – May 23	15.8	-9
May 24 – May 30	14.9	-9.5
June 1 – December 31	32	0

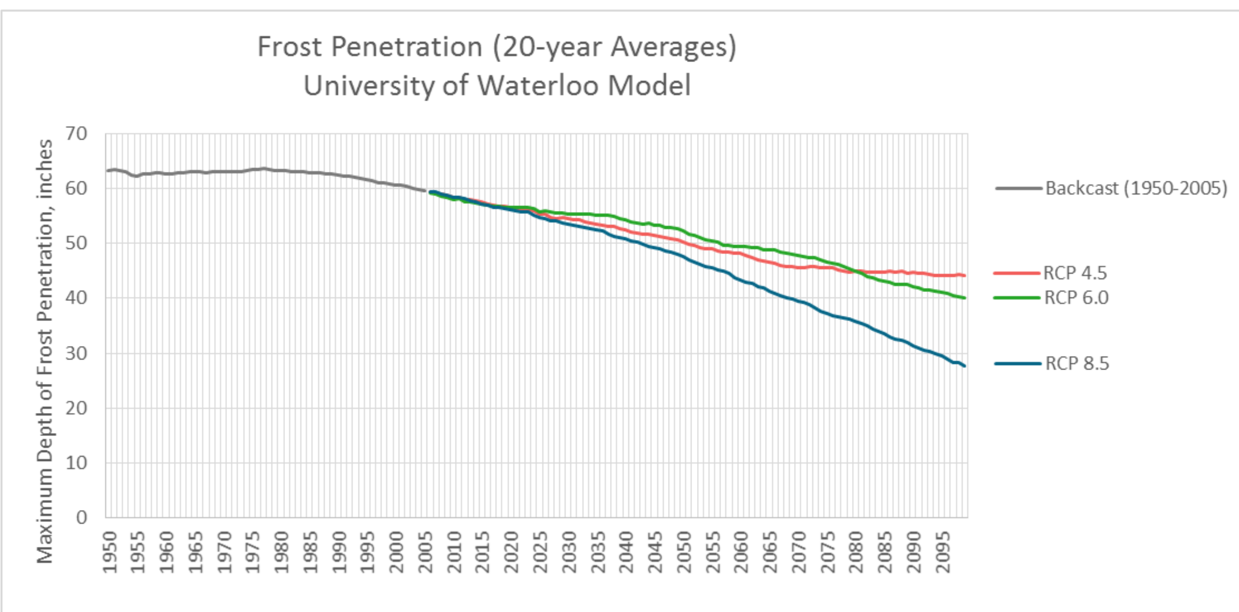

Figure 27: Estimated Maximum Depth of Frost Penetration for the Guilford Area under Various Scenarios.

Thornthwaite Moisture Index

TMI is a dimensionless measure that indicates the humidity or aridity of soils in a geographic region. TMI is modeled as a function of temperature, precipitation, and the resulting potential evapotranspiration[84] rate. Positive values of TMI indicate a humid climate with water surplus (i.e. the amount of water received through annual precipitation is higher than the amount of water lost through evapotranspiration) while negative values of TMI indicate an arid climate with water deficit (i.e. the amount of water that can be lost through evapotranspiration is higher than the amount of water received through annual precipitation). A TMI value of zero indicates that the annual precipitation is enough to meet the water demand.[85]

In pavement design, TMI is used as the key parameter to estimate moisture availability in soil. Specifically, TMI is used in the estimation of soil moisture at equilibrium state. Note that the analytical models adopted in the Mechanistic Empirical Pavement Design Guide (MEPDG)[86] uses a 12-month rolling summation of precipitation in TMI calculations in lieu of monthly precipitation-based TMI for pavement analysis[87], which takes any new wet or dry events into consideration.

TMI is a function of precipitation and potential evapotranspiration. For this case study, the TMI formula developed for NCHRP Project 1-40D Technical Assistance to NCHRP and NCHRP Project 1-40A: Versions 0.9 and 1.0 of the M-E Pavement Design Software by Arizona State University (ASU) was adopted.[87] The formula is:

$$TMI = 75(P/PE - 1) + 10$$

Where,

- P is the annual total precipitation in centimeters

PE is potential annual evapotranspiration in centimeters, which is a function of mean monthly temperature and the heat index. Figure 28 presents the summary of temporal change in annual TMI values. The average historical TMI value (i.e. 1950-2005) is approximately 85 for the Guilford site, indicating a humid climate with surplus moisture. As indicated in Figure 28, the climate data shows a slight but steady decrease in TMI over the next scores of years, albeit at different rates for different RCPs. The RCP 8.5 scenario indicates a significant decrease in TMI to an average of 62 in the 2090s while the RCP 4.5 and RCP 6.0 scenarios indicate a more modest decrease to an

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[84] Potential evapotranspiration is the amount of water that will be lost from the land surface through evaporation and transpiration (loss of water through leaves of vegetation) if sufficient water is available.
[85] Thornthwaite, 1948.
[86] The MEPDG is the state-of-the-art methodology for pavement design in the U.S. and Canada. The MEPDG is a culmination of various national and state research performed during the last few decades, including National Cooperation Highway Research Program (NCHRP) Projects 1-37A, 1-40B, 1-40D and FHWA’s Long Term Pavement Performance (LTPP) projects.
[87] Witczak et al, 2006.

average TMI value of 82 and 77, respectively, during the same timeframe. Under all the RCPs, there will still be surplus moisture but it will be less than current amounts.

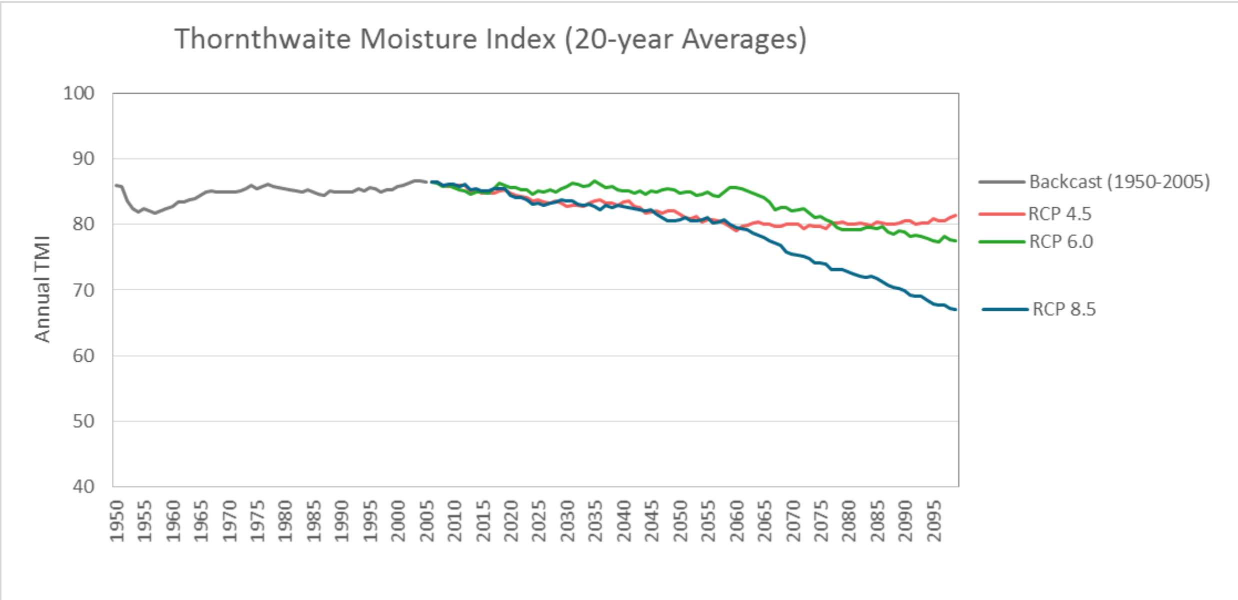

Figure 28: Annual TMI Projections for the Guilford Area under Different RCP Scenarios.

Design High and Low Temperatures for Asphalt Binder Requirements

Local climate plays an influencing role in the selection of asphalt binder grade for any roadway project. Appropriate consideration of the maximum and minimum air temperature statistics in asphalt binder grade selection is necessary to ensure adequate resistance against pavement rutting and thermal cracking[88], respectively. The annual total degree days greater than 50° Fahrenheit (10 °Celsius), which is a measure of how much and how long a pavement will experience warmer or hotter conditions, is used in assessing the high temperature grade requirements for asphalt binders. The design pavement low temperature, which is a measure of how cold (i.e. lowest minimum temperature) a pavement can experience, is used in identifying the low temperature grade requirements.

Both the average annual degree-days of daily temperatures greater than 50° Fahrenheit and design pavement low temperatures were calculated for the three RCP scenarios. The annual total degree days exceeding 10 °Celsius (DD, in Celsius), used in the determination of high temperature requirements of asphalt binders, was calculated using the following formula:[89]

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[88] Thermal cracking is a type of cracking that occurs across the asphalt pavement when thermal stresses caused by low temperatures exceed the strength of the asphalt binder. The propensity to thermal cracking increases in response to cold ambient temperature. This type of cracking is more common in northern U.S. and Canada.
[89] Mohseni, 2005.

$$DD = \sum_{i=1}^{365} max\big((T_{max} - 10), 0\big)$$

Where, T_{max} is the daily maximum temperature (in Celsius).

The design pavement low temperature (T_{pav}, in Celsius) for asphalt concrete used in the determination of low temperature requirements of asphalt binders is calculated using the following formula:[89]

$$T_{pav} = -1.56 + 0.72 * T_{air} - 0.004 * Lat^2 + 6.26 \log_{10}(H + 25) - z(4.4 + 0.52 * \sigma_{air}^2)^{0.5}$$

Where,

- T_{air} = Lowest minimum air temperature during the design period (in Celsius)
- Lat = Latitude of the site
- H = Depth from pavement surface (in millimeters)
- σ_{air}= Standard deviation of the mean low air temperature (in Celsius)
- z = Standard normal distribution value based on reliability level (at 98% reliability, z = 2.054)

Figures 29 and 30 present the projections of annual degree days and design pavement low temperatures, respectively. Reflecting the trends of future temperature projections, both the annual average degree days and design pavement low temperatures will increase steadily under different scenarios over time.

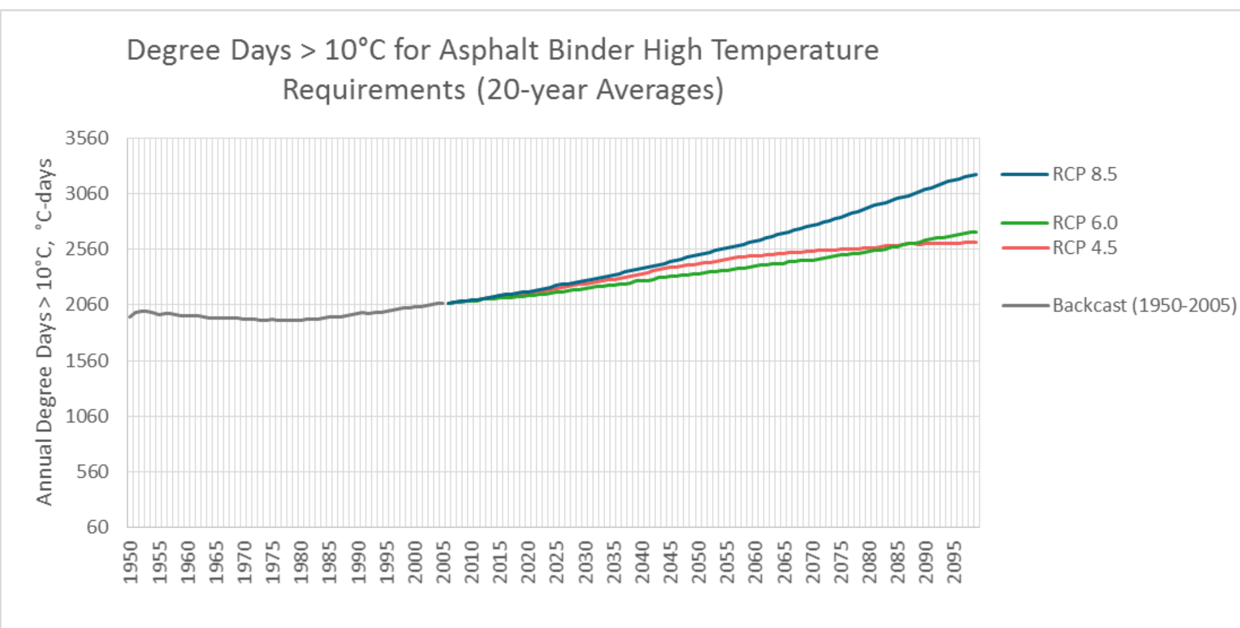

Figure 29: Projections of Annual Degree Days Greater than 50° Fahrenheit for the Guilford Area under Different RCP Scenarios.

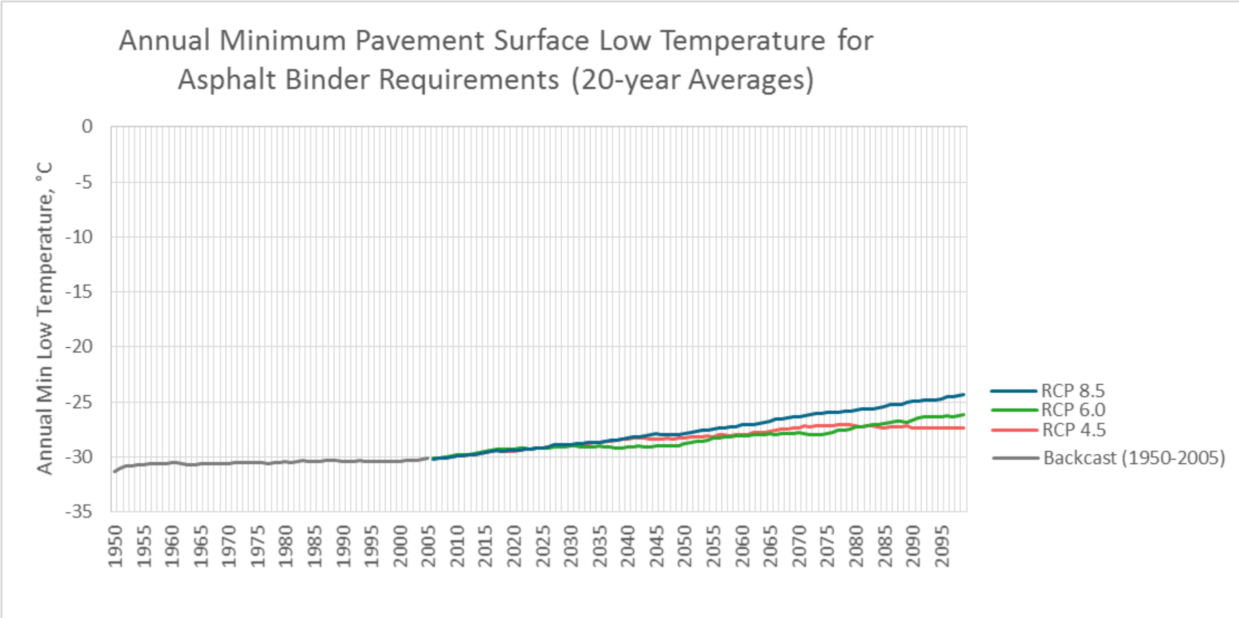

Figure 30: Design Pavement Surface Low Temperature Projections for the Guilford Area under Different RCP Scenarios.

Summary

The temperature and precipitation projections indicate the possibility of increasing exposure to warm temperatures and wet conditions (the climate stressors of interest). Thus, following ADAP, the analysis proceeded to Step 5.

Step 5: Assess Asset Performance

Among all the climate change scenarios, RCP 8.5 is projected to result in the highest increase in air temperature and precipitation and, hence, is expected to have the highest impact on pavement performance. Thus, as specified in the ADAP methodology, the project team first analyzed the existing flexible pavement system to evaluate its adequacy under the climatic conditions projected for RCP 8.5.

To have a holistic view on pavement performance, it is necessary to understand how pavements respond to changing climatic conditions at both the material and structural levels. As previously discussed, increasing air temperatures can potentially cause adverse effects on the performance of flexible pavements. This includes a reduction in the stiffness of asphalt concrete which can weaken its resistance against repeated traffic loading (i.e. long-term fatigue performance) and result in more deformation under heavier and repeated wheel loads (i.e. rutting depressions).

Beyond pavement performance, the shorter freezing seasons projected with climate change will likely have impacts on MaineDOT's seasonal load allowance and restriction policies. Therefore, future temperature patterns were also evaluated to determine how the WWP and SLR might change.

The remainder of this step is organized around the following analyses that were undertaken to evaluate the implications of RCP 8.5 (the "worst case scenario") on seasonal load restrictions and pavement performance:

- Seasonal Load Restrictions
  - Winter Weight Premiums
  - Spring Load Restrictions
- Analysis of the resilience of existing flexible pavement design
  - Subgrade support conditions
  - Asphalt binder requirements
  - Dynamic modulus (stiffness) of hot mix asphalt
  - Load-related fatigue cracking
  - Subgrade rutting
  - Asphalt concrete rutting
  - Serviceability loss due to frost heave

Seasonal Load Restrictions

In this section, the research team analyzed the impacts of temperature projections under RCP 8.5 on future seasonal load restriction policies. To help set seasonal load restriction policies, the analysis focused on when the freezing season would begin, the duration of the freezing, and when thawing would begin. The investigations of observed temperature patterns revealed that, typically, every year in Guilford, at the onset of the freezing season (late fall and early winter), there are one or more short freeze-no freeze cycles (i.e. a few days of freezing [days with an average temperature below 32° Fahrenheit] followed by a few days of no-freeze weather) before the more continuous long freezing season sets in. Similarly, at the end of the long freezing season (late winter and early spring), there are typically one or more short freeze-thaw[90] cycles before thawing persists. This trend is illustrated schematically in Figure 31 with an example of weather data collected in the winter of 1994-1995.

[90] A distinction has been made between "no freeze" and "thaw". No Freeze refers to the conditions that exist before freezing occurs, while thaw refers to the post-freezing warm conditions that ice and snow melt. Thawing is considered to set in when the daily mean air temperature exceeds the reference temperature (i.e., DTI >0).

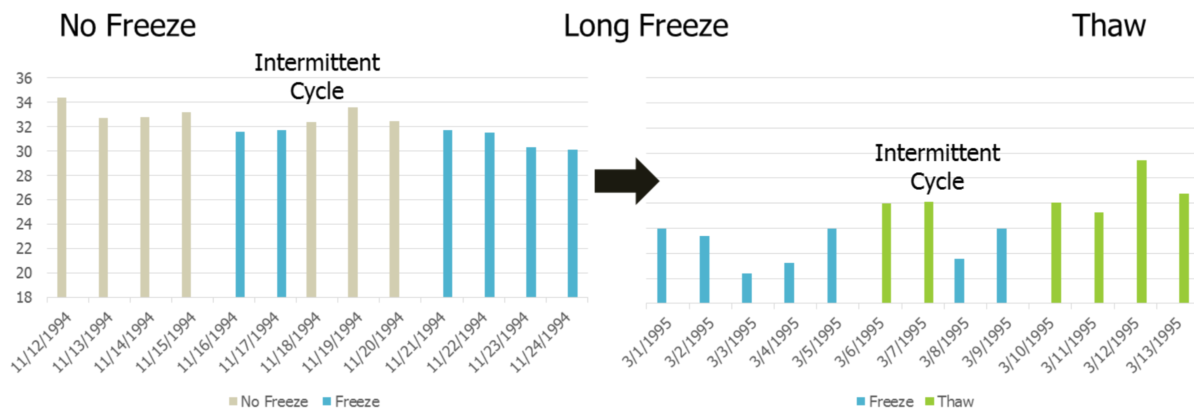

Figure 31: Example of Intermittent and Long Freezing in a Season.

Table 13 summarizes the mean projected dates of long freezing for every two decades and Figure 32 shows this information graphically. The values presented in the parenthesis indicate the range of projected dates for the two-decade period. The overall trends indicate that the freezing season will get shortened steadily over the course of the 21st century. As time progresses, the onset of the freezing season will be delayed into December while the season will end as early as February. The freezing season start and end dates will both shift at an approximate rate of one week per two decades.

Table 13: Summary of Freezing Season Projections under RCP 8.5.

Years	First Day of Freezing	Long Freeze Starts	Long Freeze Ends	Last Day of Freezing
1980-99	11/15 (11/11—11/21)	11/23 (11/14—11/30)	3/6 (2/20—3/14)	3/14 (3/3—3/21)
2000-19	11/21 (11/14—11/30)	11/27 (11/18—12/5)	3/2 (2/21—3/16)	3/12 (3/4—3/19)
2020-39	11/24 (11/13—12/2)	12/2 (11/24—12/10)	2/25 (2/15—3/4)	3/10 (3/3—3/19)
2040-59	12/1 (11/22—12/10)	12/6 (11/28—12/21)	2/21 (2/15—2/28)	3/1 (2/20—3/11)
2060-79	12/7 (12/1—12/14)	12/14 (12/7—12/26)	2/16 (2/3—2/21)	2/24 (2/20—3/6)
2080-99	12/10 (12/2—12/18)	12/21 (12/9—12/31)	2/8 (2/2—2/17)	2/17 (2/12—2/20)

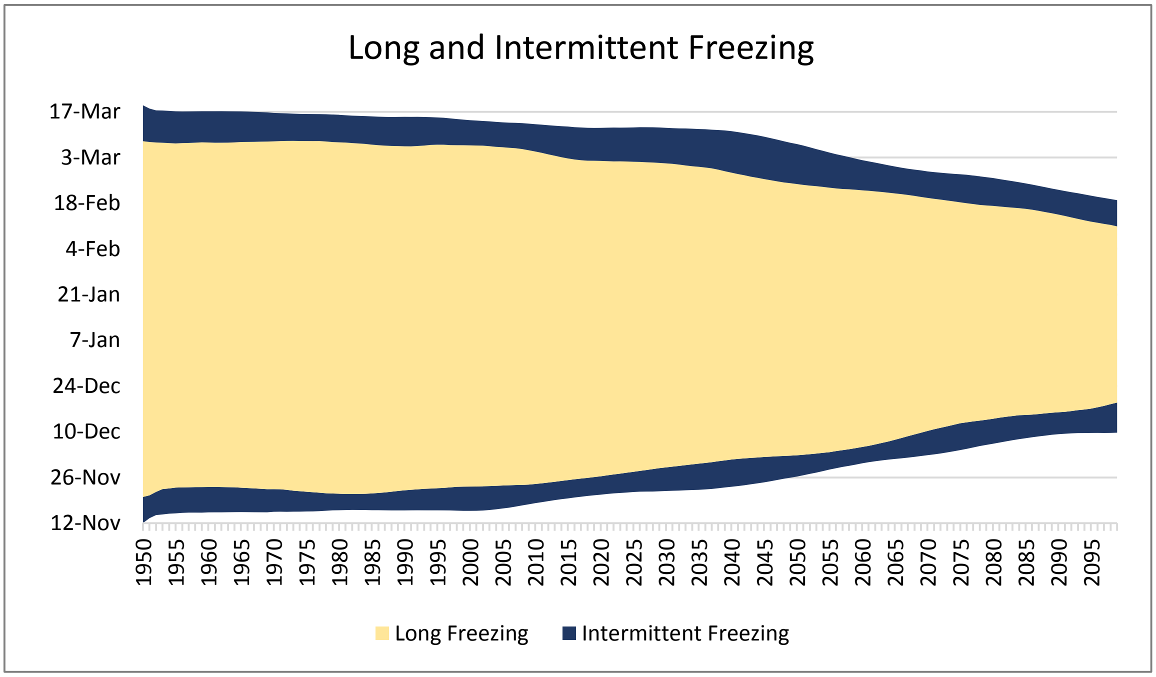

Figure 32: Summary of Long Freezing and Intermittent Freezing Season Projections under RCP 8.5.

Posting of Winter Weight Premiums

Timing is a critical factor for posting and lifting WWPs. Most highway agencies take advantage of the hardened pavement conditions to temporarily relax the posted legal limit of axle weights for a given highway by a specific limit or percent (e.g. 10 percent). As discussed earlier, the pavement system is typically hardened in winter (the pavement layer made with asphalt concrete stiffens proportionately with colder temperatures while the soil subgrade when frozen stiffens by about 20 to 120 times in comparison with non-frozen conditions). Since the load carrying capacity of the pavement is increased under frozen conditions, relaxing axle weight limits within a certain threshold will not cause undue damage to the pavement system; however, the allowable additional weights should be commensurate with the capacity of the pavement system as well as the intensity and duration of winter conditions.

The timing and duration of WWPs will depend entirely on prevailing climatic and field conditions. Delaying the posting of WWPs will deprive the trucking industry of truck weight allowances while any failure to remove the posting on time will result in additional damage to the pavement system and substantial financial implications to the highway agency.

To time the posting and lifting of WWPs, different highway agencies have developed their own methodologies including a combination of sensor-based field observations, mechanistic, and empirical analytical solutions. For example, Washington State, Minnesota, Wisconsin, Michigan and Ontario have developed their own solutions. Based on the comparisons using frost gauge measurements, MaineDOT's research study "Mechanistic Approach to Determine Spring Load

Restrictions in Maine" reported observations that the frost depths measured in the field reasonably matched with those computed using the Minnesota's empirical approach. Therefore, MaineDOT recommended using the Minnesota approach for this case study when predicting the depths of frost penetration.

Per the Minnesota approach, WWPs can be allowed when the cumulative freezing index exceeds 280 degree Fahrenheit days.[91] It is surmised that the when the cumulative freezing index reaches this threshold, there will be enough frost penetration in the pavement system (approximately equal to 2.5 feet) to accommodate weight increases.

While the decision to remove the WWPs should be solely based on on-site weather monitoring, it is relatively safer to remove the posting when the first thaw occurs (i.e. when the long freezing season ends or when the daily freezing index first becomes zero). Typically, the removal of WWPs precedes the starting date of SLRs. In case of any occurrence of intermittent freeze-thaw cycles during the long freezing seasons, the decision to remove WWPs should be based on prevailing weather and field conditions, such as the duration of thaw, precipitation amounts, etc.

Figure 33 presents the temporal trends of projected start and end dates for the WWPs as well as the estimated number of days (i.e. period between the WWP posting date and first thaw) using ensemble averages of temperature projections under the RCP 8.5 scenario. Historically, climate conditions allow the MaineDOT to post WWPs for a 10-week period starting the third to last week of December. However, with climate change, the projections show that the posting of WWPs will have to be delayed until the last week of January starting in the late 2060s and early 2070s and will only last for a period of two to three weeks. Beginning in the 2080s, the opportunities for posting WWPs might be low (i.e. less than two weeks) or completely lost.

[91] Mechanistic Approach to Determine Spring Load Restrictions in Maine, Ovik et al, 2000.

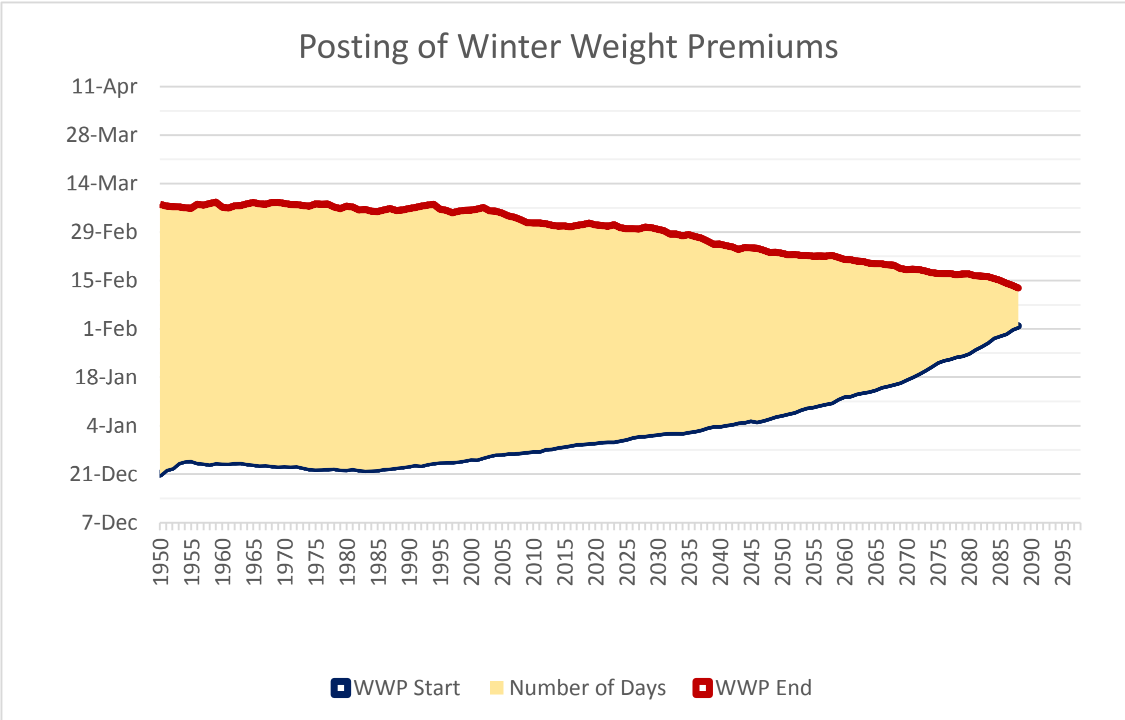

Figure 33: Projected Start and End Dates for WWPs under the RCP 8.5 Scenario using Ensemble Averages.

Considering the high variability in temperature projections among the 11 different global climate models, the starting dates for WWPs were computed for each model. Figure 34 presents the corresponding variability in the WWP start dates with the dark line representing the start dates projected by the ensemble averages and the yellow area representing the range across all the models. The light red area indicates that there may not be adequate frost or cold to enable the posting of WWPs for a duration of at least two weeks.

Most agencies have traditionally allowed a 10 to 25 percent increase in the maximum annual allowable gross vehicle weight (typically 80,000 pounds) during WWPs. However, with shorter freezing periods and decreasing frost penetration depths, the payload weight increases that agencies have traditionally allowed will also likely decrease over time. Thus, WWPs will need to be both smaller and applied over a shorter time period under the RCP 8.5 scenario.

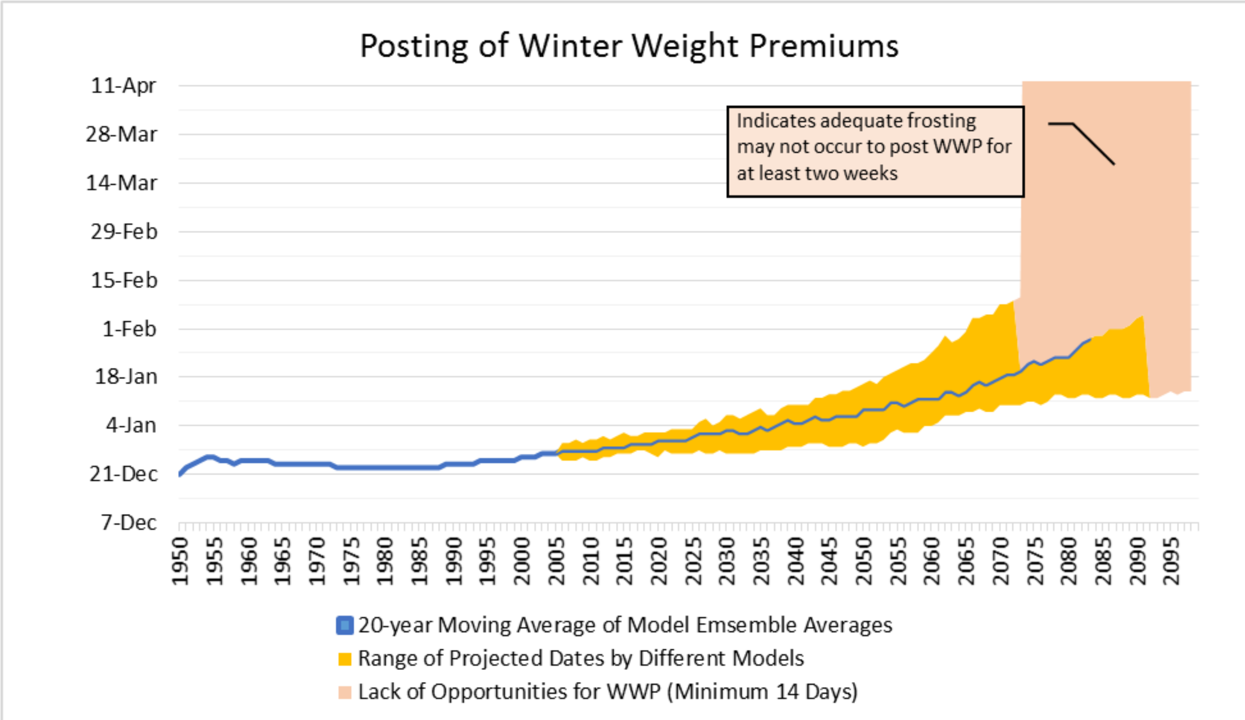

Figure 34: Projected Start Dates for WWPs by Different Global Climate Models under the RCP 8.5 Scenario.

Posting of Spring Load Restrictions

Highway agencies post SLRs to protect the pavement system against significant damage during thaw weakening. Typically, in early spring, when thaw occurs from the top-down, excess water melting from subsurface sources (e.g. ice lenses and frosting) and surface infiltration (ice and snow on the ground) is released into the pavement system. While immediate thawing occurs on the top surface, the lower layers of frozen soil melt only as the thaw progresses with time. When the thawing process is under progress, the excess water in upper thawed areas is trapped above the impenetrable underlying layers of frozen soil thus affecting its drainage.

The excess water in the upper soil layers drastically reduces the load carrying capacity of the pavement system. The MEPDG suggests that the resilient modulus (i.e. load carrying capacity) of the soil subgrade will reduce by 25, 37, 60, and 48 percent of its normal (unfrozen) modulus for gravel, sand, silt and clay soils, respectively.[92] Therefore, a timely posting of SLRs that would limit the truck weights applied on the weakened pavement will help control pavement damage.

Per Minnesota's approach, SLRs should be in-place when the cumulative thawing index reaches 25 degree Fahrenheit days.[93] Figure 35 presents the temporal trends of projected spring load restriction start dates. The figure also presents the variability in estimated start dates due to

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[92] AASHTO, Mechanistic-Empirical Pavement Design Guide, 2007.
[93] Mechanistic Approach to Determine Spring Load Restrictions in Maine, Ovik et al, 2000.

differences in the temperature projections of the various global climate models. The blue line represents the start dates projected by the ensemble average of the models and the yellow area represents the range of projected start dates across the models.

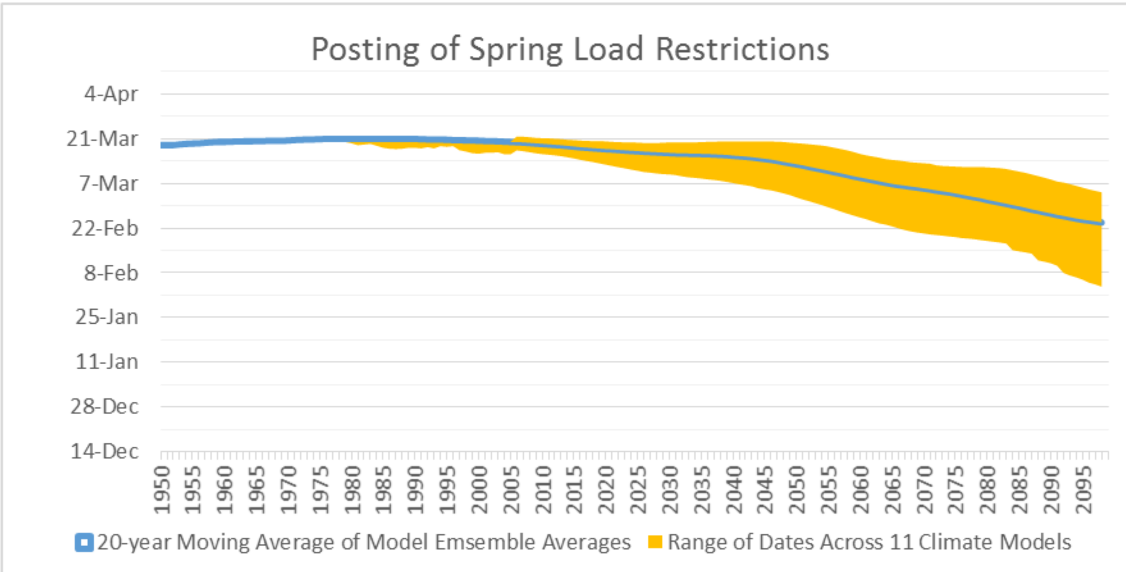

Figure 35: Projected Start Dates for Spring Load Restrictions by Different Global Climate Models under the RCP 8.5 Scenario.

As the historical temperature trends suggests, the SLRs could currently be in-place starting the second to third week of March; however, as expected, the temperature projections due to climate change indicate the need for earlier posting of the restrictions. Towards the end of the 21st century, the estimated start dates for posting SLRs using ensemble averages indicates they should occur in late February while some global climate models indicate they should occur as early as late-January.

The time period required to maintain SLRs should be commensurate with the intensity and duration of the freezing season. While MaineDOT currently uses a fixed period of eight weeks based on experience with the historical climate,[94] surrogate measures, such as the depth of frost penetration or duration of thaw, provide better insights on when to remove SLRs, particularly when field data is unavailable. Per Minnesota's approach the duration for thaw is a function of the maximum CFI (see Step 4) and maximum frost depth (FD) and is determined using the following formula:

$$D = 0.10 + 0.10 * CFI - 0.485 * FD - 307.086 * \left(\frac{FD}{CFI}\right)$$

[94] Most highway agencies use a fixed period of SLR that may range from eight to 12 weeks.

Figure 36 presents the temporal trends of the estimated duration for thawing using Minnesota's formula. Per the estimations made using historical temperature data, typically 75 to 90 days are required for the subsurface frost to go out of the pavement system. However, considering the warming trends and lower depths of frost penetration in the future, the subgrade might retain the frost conditions for no longer than a month by the 2050s and even shorter by the 2070s.

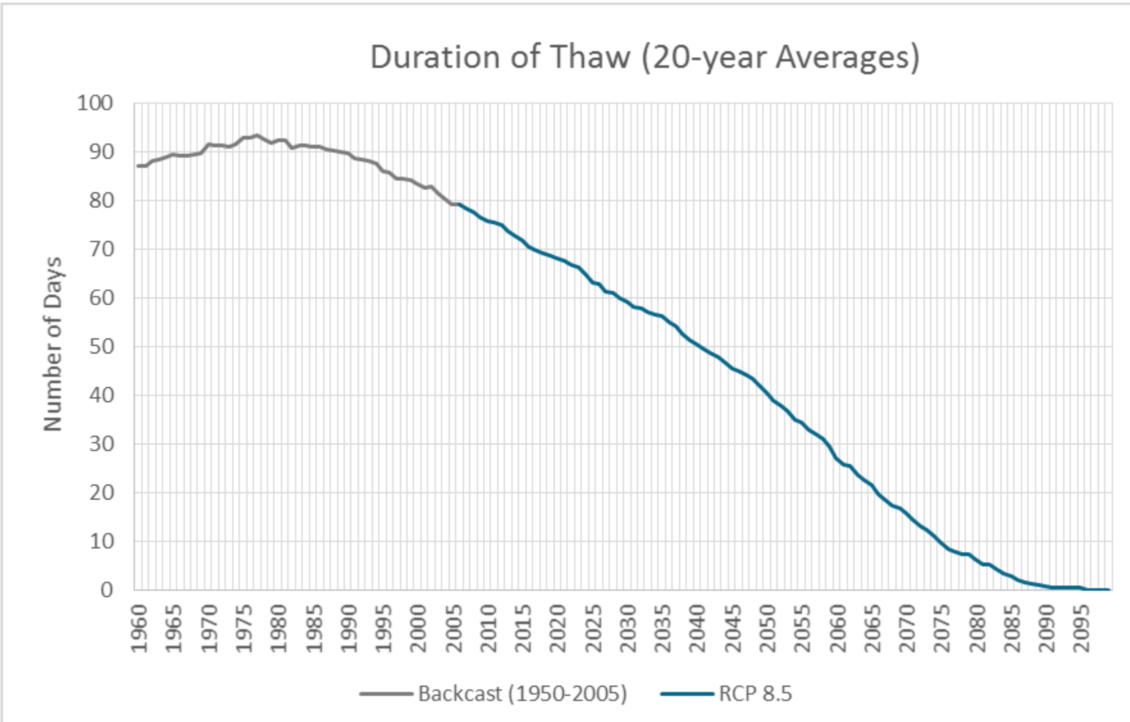

Figure 36: Estimated Number of Days for Thawing under the RCP 8.5 Scenario.

Even when the subsurface frost is completely out of the pavement system, the subgrade soils will still remain in the saturated condition. Despite the shorter freezing seasons and lower depths of frost penetration, precipitation levels are projected to remain the same or increase somewhat. The in-place saturated soil will need the time to drain out all the excess water from snow or rain and return back to normal conditions. This means that despite shorter thawing periods, it is reasonable to surmise that the weakened state of the pavement could exist for about the same amount of time.

Furthermore, the time required for subgrade soils to recover from the thawed condition to normal condition are generally governed by soil characteristics. The MEPDG suggests that the average recovery periods of subgrade soils are 90, 120, and 150 days for sands/gravels, silts, and clays, respectively.[95] For the sandy silt and silt soils prevalent at the Guilford site, the fixed eight-week SLR period appears reasonable, as the soils will recover back to at least 80 percent of their

[95] AASHTO, Mechanistic-Empirical Pavement Design Guide, 2007.

normal support conditions during that time. Thus, the time period that the subgrade requires to recover back to normal conditions is expected to remain unchanged in the future. Note that the recovery period can be shortened, if the excess water can be drained faster using an adequate and well-maintained pavement subsurface drainage system.

Analyses for the Flexible Pavement Design

The following section discusses the adequacy of the as-built flexible pavement system relative to the changing climate conditions projected under the RCP 8.5 scenario. The as-built pavement system was analyzed to predict changes in key structural distresses (fatigue cracking, subgrade, and AC rutting) as well as serviceability loss due to frost heave, for five different 20-year design periods: [96]

- Base period (1980-1999)
- 2020-2039
- 2040-2059
- 2060-2079
- 2080-2099

As discussed in Step 4, each of the design periods have different climate patterns and thus will have different associated impacts on the AC dynamic modulus and subgrade support conditions. The traffic levels were assumed to remain the same as current design period for all five design periods for the following reasons: (i) no future traffic projections were available beyond 2024; and (ii) adding traffic forecasts may introduce confounding effects in evaluating the impacts of climate change on pavement performance. While it is probable that traffic volumes would continue to increase between 2024 and 2099 resulting in increased impacts, the effects of future traffic growth on pavement performance could not be assessed in this study.

Long Term Impacts on Subgrade Support Conditions

The impacts of future temperature and precipitation trends, as captured by the TMI (the measure of humidity or aridity of soil), on the subgrade support conditions were investigated. Note that the TMI is a commonly used measure to evaluate the climate induced variations on the in-situ resilient modulus[97] of subgrade soils. As indicated by the TMI projections in Figure 28, the combined effects of future warming and slightly wetter trends at the Guilford site, will still result in humid conditions but with less surplus moisture over time. This reduction in surplus moisture

[96] A design period is the time period for which a pavement structure is being designed to keep structural distresses under a given threshold. Flexible pavements are typically designed to withstand traffic volumes over a 20-year design period.

[97] In-situ resilient modulus indicates the load bearing capacity of the soil at its present state.

due to warming conditions may bring an improvement in subgrade support conditions because the soil is more stable without the excess water.

The research team used the TMI-Matric Suction[98] models, developed by ASU for the National Cooperative Highway Research Program (NCHRP) Project 1-40D and adopted in the MEPDG, to evaluate the impacts of TMI change on subgrade resilient modulus.[99] Supported by the evidence gathered in field testing and numerical modeling, the TMI-Matric Suction models are based on the assumption that the subgrade beneath a pavement eventually attains a relatively stable moisture state (i.e. equilibrium state) under normal conditions, irrespective of whether the in-situ moisture is lower or higher than the optimum moisture,[100] over a few weeks or months for fine-grained soils. The research team used the TMI-Matric Suction model in Figure 37 to estimate the subgrade moisture at equilibrium and the corresponding resilient modulus. The model requires TMI and soil properties as inputs for computing soil stiffness at equilibrium.

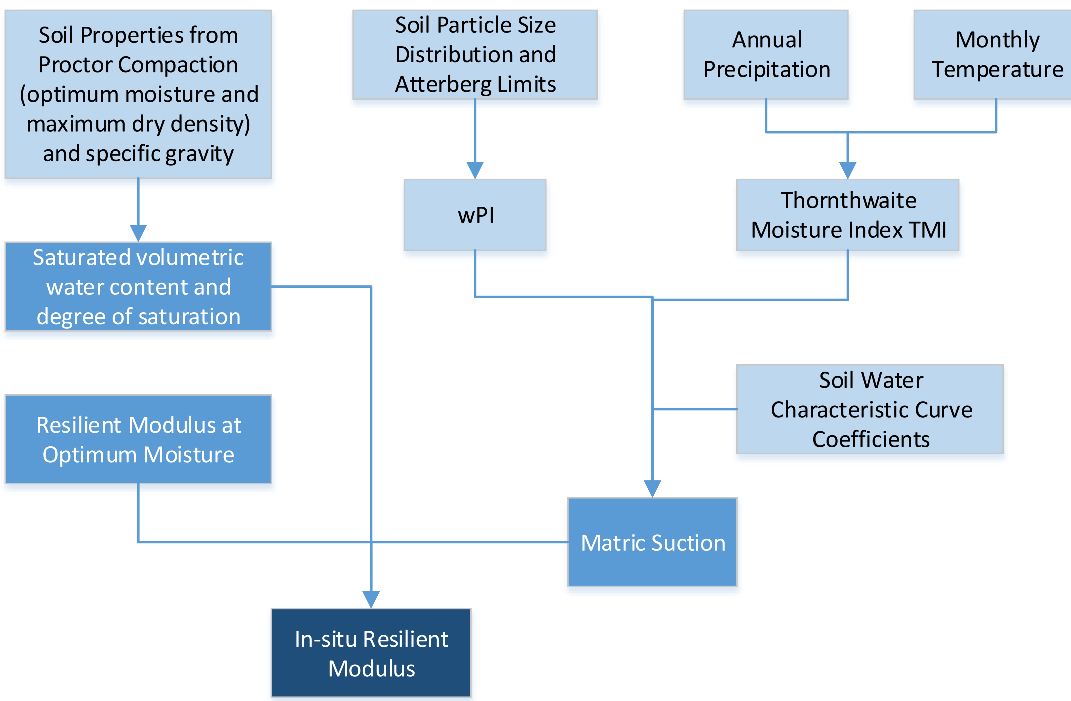

Figure 37: Schematic of ASU TMI-Matric Suction Model

The research team used the ASU TMI-Matric Suction model and projected TMI values (see Figure 28), to estimate the resilient modulus of subgrade soils for both the baseline climate (for

[98] Matric suction is the pressure exerted by soils in drier state on the surrounding soils in wetter state to equalize the differences in moisture content between two soil masses.
[99] Witczak et al, 2006.
[100] The optimum moisture of soil is the moisture content at which the maximum density can be achieved at a given compaction effort which translates to the maximum load bearing capacity or resilient modulus of soil. The moisture content at equilibrium can be above or below the optimum moisture content.

reference) and the RCP 8.5 scenario. As expected, an increase in the soil resilient modulus was found; however, it is projected to be very small (less than one percent in all design periods). The anticipated decrease in TMI from a historical value of 85 to 62 in the 2090s still leaves the soil condition in a humid state, and hence, would not have any noticeable impact on the equilibrium conditions of the soil resilient modulus in the longer term.

Impacts on Asphalt Binder Performance Grade

The research team conducted an analysis using the Long Term Pavement Performance Program Bind (LTPPBind) 3.1 software tool to evaluate how future temperature projections would influence the selection of asphalt binder grade. The LTPPBind 3.1 uses a combination of empirical models to estimate high and low pavement temperature needs for a project location.[101] While the selection of the base asphalt binder grade is largely dependent on local climate, additional adjustments are often made to performance grade to account for higher reliability, additional damage potential under slower traffic and higher traffic volume, and the depth from pavement surface where the AC layer will be placed.

The process to determine the recommended high and low temperature grade of an asphalt binder with consideration of climate change involved the following steps. The output of each of these steps is documented in Table 14.

1. Calculate the average annual degree days above 50° Fahrenheit for each 20-year period as done in Step 4 (see Figure 29).
2. Calculate the base PG high temperature (PGd) at 50 percent reliability using average annual degree days (DD) for a threshold rut depth[102] (RD) of 0.5 inches[103]: $PGd = 48.2 + 14\ DD - 0.96\ DD^2 - 2\ RD$
3. Given the statistical variations in climate factors, adjust the base PGd for higher factor of safety at 98 percent reliability (PGrel) using the following formula: $PGrel = PGd\ (1 + z*CVPG/100)$
   Where, z is standard normal variable corresponding to a desired reliability (e.g. 2.055 for 98% reliability) and CVPG is the yearly coefficient of variation of pavement base high temperature, in percent, estimated using the formula: $CVPG = 0.000034\ (latitude-20)^2 * (RD)^2$
4. Typically, at this point, one would then adjust the PG high temperature for pavement depth and traffic. Assuming that a two-inch surface layer will be used for flexible

[101] Mohseni, 2005.
[102] Note that the LTPP 3.1 algorithm uses a mechanistic-empirical model to ensure adequate resistance against rutting damage to estimate high temperature needs.
[103] Note that the asphalt binder is typically designed to a tolerable rut-depth (wheel depression) of 0.5 inches over 20 years. It is widely acceptable to consider 0.5 inches as the threshold for tolerable rutting.

pavements, the LTPPBind 3.1 recommends a negative temperature adjustment of 5.2° Fahrenheit for a design depth of one inch below the pavement surface.[104] For an assumed traffic range of three to 10 million ESALs[105] over the design period, the LTPPBind 3.1 recommends a positive temperature adjustment of 11.7° Fahrenheit to the PG 64-28 currently being used, in accordance with the recommendations of the LTPPBind 3.1 software.

5. Calculate design pavement low temperature as done in Step 4 (see Figure 30)

Table 14 presents the high and low temperature performance grade requirements for asphalt binder under RCP 8.5. As the table indicates, the PG 64-28 binder, currently used by MaineDOT on SR-6/SR-15/SR-16, will be adequate to withstand the increasing maximum and minimum temperatures.[106]

Table 14: Asphalt Binder High and Low Temperature Requirements under RCP 8.5.

Year	Average Annual Degree Days	PG Damage (PGd) °C (°F)	PG with Reliability (PGrel) °C (°F)	PG with Traffic and Depth Adjustment °C (°F)	Design Pavement Low Temperature °C (°F)	Recommended PG Grade
1980-99	2,003.9	47.5 (117.5)	49.4 (120.9)	54.3 (129.7)	-32.0 (-25.6)	PG 58-34
2000-19	2,132.9	48.9 (120.0)	50.8 (123.4)	55.7 (132.3)	-31.2 (-24.2)	PG 58-34
2020-39	2,316.6	50.7 (123.3)	52.7 (126.9)	56.9 (134.4)	-29.8 (-21.6)	PG 58-34
2040-59	2,552.9	53.0 (127.4)	55.1 (131.2)	59.3 (138.7)	-29.4 (-20.9)	PG 64-34
2060-79	2,831.2	55.6 (132.1)	57.7 (135.9)	61.3 (142.3)	-27.3 (-17.1)	PG 64-28
2080-99	3,149.8	58.2 (136.8)	60.5 (140.9)	63.7 (146.7)	-26.2 (-15.2)	PG 64-28

[104] The design depth of one inch is the midpoint of the two-inch AC surface layer.
[105] High temperature performance grade of asphalt binder needs to be adjusted to account for additional damage potential under slow traffic and for traffic volume exceeding three million cumulative 18 kip equivalent single axle load applications.
[106] The selection of asphalt binder is an engineering design decision that often involves considerations of reliability, market, and other local factors. Those factors are not captured in this analysis.

Impacts on Hot Mix Asphalt Dynamic Modulus

Changes in temperature will have an impact on the dynamic modulus[107] (|E*|) of AC layers. Among other factors, both temperature and asphalt binder grade are considered to have profound effects on AC's resistance against damage under repeated loading. It is necessary to evaluate how the stiffness of AC, used in the in-place pavement surface layer at the Guilford site, will change over time under RCP 8.5. The research team used the AC |E*|prediction model,[108] adopted in the MEPDG to estimate the AC dynamic modulus values. [109]

Figure 38 presents a temporal distribution of AC dynamic modulus estimated for a typical MaineDOT bituminous mixture[110] using the future projections of annual mean temperatures under RCP 8.5 scenario with the currently used asphalt binder grade of PG 64-28. As the figure indicates, a steady decrease in AC stiffness is expected over time. With the projected increase in temperature, the estimated reduction in AC stiffness is approximately 14 percent by the 2070s and 18 percent by the 2090s. The estimated reduction in AC stiffness can be primarily attributed to the projected trends in future air temperature. Any reduction in AC stiffness indicates the softening of AC layers, which will contribute to increases in pavement distresses, such as rutting and fatigue cracking.

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[107] Dynamic modulus is a mechanical property that indicates the stiffness (or the ability to withstand deformation against the applied force) of a material. Dynamic modulus of asphalt concrete is a function of temperature and loading time. The same material of asphalt binder or asphalt concrete mixture will have higher stiffness under lower temperatures and longer loading time (i.e. lower traffic speeds) and lower stiffness under higher temperatures and shorter loading time (i.e. higher traffic speeds).

[108] The AC |E*| prediction model uses pavement temperature, performance grade of asphalt binder, and other properties in dynamic modulus estimations.

[109] Witczak et al, 2006.

[110] The bituminous surface mixture used during the pavement reconstruction of SR-6/SR-15/SR-16 was assumed to be typical for all surface mixtures used in roadways of the same functional class and similar traffic characteristics in Maine. The properties of the bituminous mixture, as reported in Mallick et al, 2006, were used in the prediction of AC dynamic modulus.

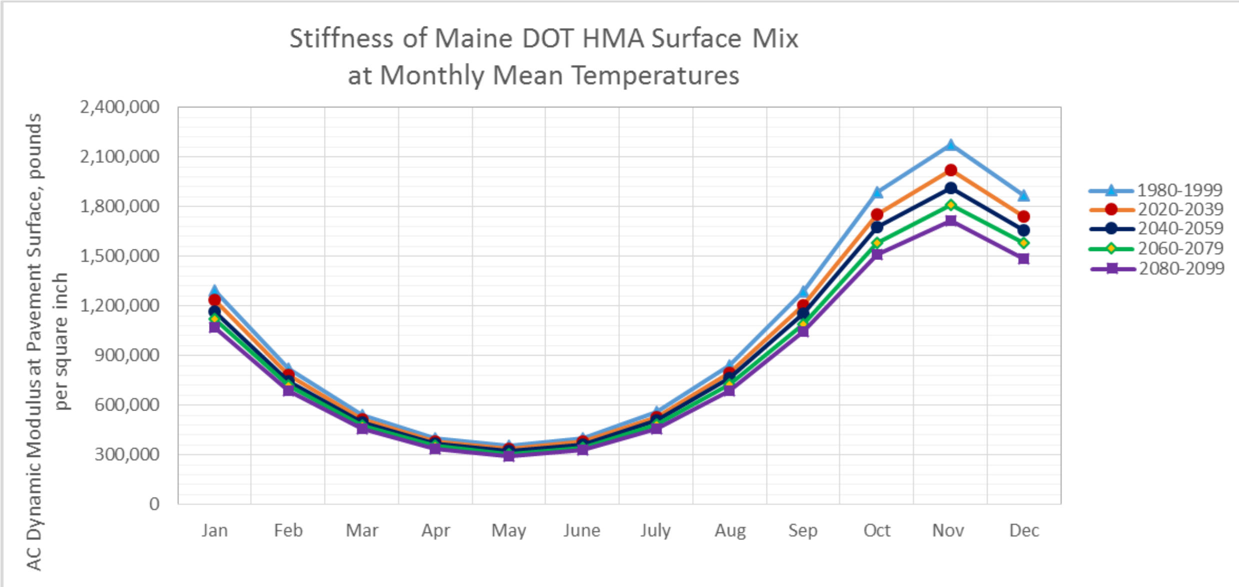

Figure 38: Stiffness of a Typical Bituminous Mix using Annual Mean Temperature Projections under RCP 8.5.

Structural Distresses

In this section, three key structural distresses affecting flexible pavement were analyzed: load-related "bottom-up" fatigue cracking, subgrade rutting, and AC rutting.

Note that the performance prediction models used herein were not locally calibrated for Maine conditions (e.g., MaineDOT's asphalt concrete mixture types and construction practices); and therefore, the absolute values of predicted performance are not presented; instead, only the percent changes are reported for the purpose of comparing various climate change scenarios.

Load-Related "Bottom-Up" Fatigue Cracking

Load-related cracking is a series of interconnected cracks in an "alligator" pattern caused by the fatigue failure of AC under repeated traffic loading. This form of cracking initiates at the bottom of the AC layers due to repeated bending under traffic loading, the cracking eventually propagates to manifest on the surface of the pavement structure, as more fatigue damage is accumulated under repeated traffic loading (see Figure 39).

Figure 39: Example of Fatigue Cracking in a Flexible Pavement.[111]

The research team evaluated the fatigue cracking potential of the existing pavement under RCP 8.5 using the methodology proposed by the Asphalt Institute.[112] The Asphalt Institute's fatigue cracking model postulates the following fatigue equation for a standard bituminous mix with a design asphalt binder volume of 11 percent and an air void volume of five percent:

$$N_f = 0.0796 * (\varepsilon_t)^{-3.291} * |E^*|^{-0.854}$$

Where,

- N_f is allowable number of axle load applications that would result in fatigue cracking of 20 percent of the total area,
- ε_t is the horizontal strain at the bottom of the AC layer[113], and
- $|E^*|$ is the AC dynamic modulus.

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[111] Source: Flexural Fatigue. Accessible at: http://www.pavementinteractive.org/article/flexural-fatigue/

[112] Huang, 2003.

[113] The horizontal strains can be calculated using any standard layered elastic analysis software available in the public domain. In engineering analysis, a flexible pavement structure is commonly modeled as a multi-layered system and analyzed using theories of elasticity. The theory of elasticity is one of the domains of engineering sciences that deals with studying the behavior of solid materials under loading. The solutions derived using the theory of elasticity are implemented in layered elastic analysis software to compute engineering parameters (stresses, strains, and deflections) at any point in a pavement structure as responses to application of traffic loads.

For the existing pavement, the estimated allowable number of load applications is 8.2 million ESALs.

The research team calculated the cumulative index for fatigue damage (DI_{FC}) as the ratio of actual number of axle load applications within a specific time period (n) over the allowable number of load applications (N_f). If multiple time periods or loading groups (i.e. by axle type, truck type, or axle weight) are used in the damage analysis, the damage index is computed as a summation of incremental damage indices over time for each period or loading subgroup:

$$DI_{FC} = \Sigma\, n/N_f$$

A damage index of one indicates that there is a 50 percent probability that fatigue damage (manifested as the interconnected cracks on the pavement surface) will occur. The damage index will be less than one when the cumulative traffic load applications that the pavement is designed to carry has not exceeded its structural capacity. Note that the Asphalt Institute does not include reliability considerations in damage estimations.

Table 15 presents a summary of percent change in fatigue performance of the existing flexible pavement system for the 20-year design periods. Compared to current conditions and estimated fatigue damage, the pavement structure will undergo more damage in the future, as high as 34 percent, under the climate conditions projected in the RCP 8.5 scenario. The projected increase in fatigue damage can be primarily attributed to loss of performance benefits from overall softening of the asphalt concrete across all seasons, particularly with shorter freezing seasons, over the next 85 years.

Table 15: Summary of Relative Change in Fatigue Damage under the RCP 8.5 Scenario.

Design Period (years)	Percent Increase in Fatigue Damage between Historical Climate Conditions and Projected Temperatures (RCP 8.5)
2020-39	9%
2040-79	16%
2060-79	25%
2080-99	34%

Subgrade Rutting

Subgrade rutting is a permanent deformation or surface depression in the wheel paths of the roadway surface caused by consolidation of the subgrade under repeated traffic loading. Figure 40 presents a schematic of pavement rutting caused by this damage.

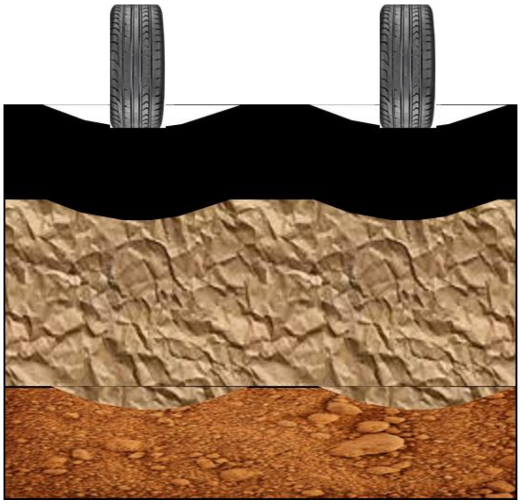

Figure 40: Schematic of Pavement Rutting in a Flexible Pavement Caused by Subgrade Consolidation.[114]

The Asphalt Institute method postulates that the permanent deformation of the subgrade can be limited when the vertical compressive strain on the top of the subgrade is controlled. The Asphalt Institute's equation for estimating the allowable number of load repetitions before permanent deformation of the subgrade is:[115]

$$N_d = 1.365*10^{-9} *(\varepsilon_c)^{-4.477}$$

Where,

- N_d is the allowable number of load repetitions to limit subgrade related permanent deformation, and
- ε_c is the vertical compressive strain on the top of the subgrade.

Table 16 presents a summary of percent change in accumulated damage due to subgrade rutting between current and future climatic conditions. Under the climate conditions projected for the 20-year design periods under RCP 8.5, the current pavement structure will undergo increasingly more damage, as high as 40 percent at the end of the century. This increase in damage is primarily due to loss of performance benefits from shorter freezing seasons as well as overall softening of the asphalt concrete due to systematic warming trends over the next 85 years.

[114] Image source: Geotechnical Aspects of Pavements Reference Manual, Publication No. FHWA NHI-05-037, May 2006. Accessible at: http://www.fhwa.dot.gov/engineering/geotech/pubs/05037/01.cfm
[115] Huang, 2003.

Table 16: Summary of Relative Change in Subgrade Rutting under the RCP 8.5 Scenario.

Design Period (years)	Percent Increase in Rutting Damage between Historical Climate Conditions and Projected Temperatures (RCP 8.5)
2020-39	10%
2040-79	19%
2060-79	28%
2080-99	40%

AC Rutting

AC rutting is a permanent deformation or surface depression in the wheel paths of the roadway surface that is caused by consolidation or lateral movement in AC layers due to repeated traffic loading and/or issues relating to mixture design and construction. Figure 41 presents a photograph of AC rutting in the wheel paths.

Figure 41: Example of AC Rutting in Flexible Pavements.[116]

[116] Image source: Pavement Interactive, Published May 6, 2008. Accessible at: http://www.pavementinteractive.org/article/rutting/.

The research team used the AC rutting model from the MEPDG for the analysis of this phenomenon in this case study. The rutting model assumes, for all AC mixtures, the accumulation of permanent deformation in the AC is calculated as follows: [117]

$$\Delta_{p(AC)} = \varepsilon_{p(AC)} * h_{AC} = \varepsilon_{r(AC)} * k_z * 10^{-3.35412} * n^{0.4791} * T^{1.5606}$$

Where,

- $\Delta_{p(AC)}$ is the accumulated permanent deformation in the AC layer/sublayer,
- $\varepsilon_{p(AC)}$ is the accumulated permanent (plastic) axial strain in the AC layer/sublayer,
- $h_{(AC)}$ is the thickness of the AC layer/sublayer,
- $\varepsilon_{r(AC)}$ is the resilient or elastic strain calculated by the structural response model at the mid-depth of each AC sublayer,
- n is the number of axle load repetitions,
- T is the pavement temperature, and
- k_z is the depth confinement factor. The depth confinement factor is calculated using the following formula:

$$k_z = (C_1 + C_2 * D) * (0.328196)^D$$

$$C_1 = -0.1039(H_{HMA})^2 + 2.4868 H_{HMA} - 17.342$$

$$C_2 = 0.0172(H_{HMA})^2 - 1.7331 H_{HMA} + 27.428$$

Where,

- D is the depth below the surface, and
- H_{AC} is the total AC thickness.

Note that the model coefficients in the $\Delta_{p(AC)}$ equation (-3.35416, 0.4791, and 1.5606) are based on the calibration of national datasets.

Table 17 presents the percent change in accumulated AC rutting between current and future climatic conditions for the 20-year design periods under RCP 8.5. With the projected temperature increase, the current pavement structure will experience higher rutting in the AC layers, by approximately 40 percent, in relative terms, by the end of the 21st century, relative to an extrapolation of historic climate conditions. This may indicate the need for additional maintenance treatments or earlier resurfacing. However, given the unavailability of local calibration coefficients and an actual spectrum[118] of truck axle weights, the findings presented

[117] AASHTO, Mechanistic-Empirical Pavement Design Guide, 2007.

[118] Traffic load spectra capture the distribution of axle weights more accurately by vehicle types, number of axles, and seasons than using a standard 18 kilo pounds single axle load application.

herein are contingent upon further evaluation through detailed field investigations as well as more accurate characterization of traffic loads and materials to accurately predict AC rutting.

The selection of asphalt binder grade plays a critical role in providing the AC mix the resistance to rutting at high temperatures. As recommended in Table 14, the PG 64-YY should provide adequate resistance to control AC rutting under a threshold depth of 0.5 inches during a 20-year period.[119] However, note that the PG 64-YY binder, which is currently used by the MaineDOT, is one high temperature grade higher than the required binder grade PG 58-YY. With increasing propensity to rutting under warming trends, the results in Table 17 indicate that the likelihood of AC rutting not exceeding a threshold depth of 0.5 inches will decrease over time. In other words, the expected performance of the AC mix with the same PG 64-YY binder in the future will be less reliable than the performance expected under current conditions.

Table 17: Summary of Relative Change in Predicted AC Rutting under the RCP 8.5 Scenario.

Design Period (years)	Percent Increase in Rutting Damage between Historical Climate Conditions and Projected Temperatures (RCP 8.5)
2020-39	11%
2040-79	19%
2060-79	29%
2080-99	42%

Change in Serviceability Loss Due to Frost Heave

With the anticipated reduction in design freezing indices and frost penetration due to warming trends under RCP 8.5, there is lower potential for frost heave and hence smaller serviceability losses due to frost heave. These benefits are reflected in the design serviceability loss (ΔPSI) and the estimated number of load applications to terminal serviceability.

Table 18 presents the change in serviceability loss for various design cases under the RCP 8.5 scenario. As the design freezing indices decrease over time, the contribution of environmental factors to allowable loss in pavement serviceability becomes smaller, and hence allows the pavement to withstand additional traffic over time. The serviceability improvements presented in Table 18 only captures the marginal benefit due to lower frost heave potential under RCP 8.5;

[119] The asphalt binder is typically designed to a tolerable rut-depth (wheel depression) of 0.5 inches over 20 years. It is widely acceptable to consider 0.5 inches as the threshold for tolerable rutting.

however, the overall serviceability of the pavement may get worse, as other distresses, i.e. load-related cracking and rutting, are expected to increase under RCP 8.5.[120]

Table 18: Change in Serviceability Loss under RCP 8.5.

Design Period (years)	Design Freezing Index	ΔPSI_{max}	Sandy Silt Soils		Silt Soils		Increase in ΔPSI due to frost heave
			Φ (mm/day)	ΔPSI_{FH}	Φ (mm/day)	ΔPSI_{FH}	
1980-99	2114	1.57	7	0.49	20	0.52	0
2020-39	2055	1.55	7	0.48	20	0.51	0.1
2040-79	1912	1.50	7	0.47	20	0.50	0.2
2060-79	1777	1.45	7	0.45	20	0.48	0.4
2080-99	1604	1.38	7	0.43	20	0.46	0.6

Summary

To summarize, there will be both beneficial and detrimental effects to flexible pavements under RCP 8.5. The benefits will mainly be through minimized contribution of frost heave to pavement smoothness loss. The potential vulnerabilities to pavement performance, which can be attributed to softening of bituminous pavement layers[121] due to warming trends, under RCP 8.5 include:

- Increase in load related fatigue damage or alligator cracking
- Increase in subgrade rutting
- Increase in AC rutting

Shorter freezing seasons will also result in fewer opportunities for WWPs and earlier posting of SLRs.

Overall, the economic costs of climate change due to increased pavement damage and lost opportunities for WWPs outweigh the benefit of marginal improvement in the contribution of frost heave to pavement smoothness loss. Therefore, the current flexible pavement systems are likely to be adversely affected by projected climate changes under RCP 8.5 and adaptation actions will need to be developed for each impact in Step 6.

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[120] The overall reduction in PSI due to increased rutting and cracking is not quantified herein due to unavailability of other inputs, such as pavement roughness. Further, there is no information available as to whether the AASHTO Road Test PSI model was locally validated in Maine.

[121] Recall that a decrease in AC dynamic modulus indicates the softening of bituminous materials.

Step 6: Develop Adaptation Options

Pavement infrastructure performance is a complex system with interactions of multiple factors including traffic, soil foundation, pavement structural types and layer thicknesses, climate, material properties, and construction and therefore the methods to address the impacts of climate change are diverse. There are a multitude of structural, material, and construction related strategies that can be used in some combination to address the impacts of climate change on pavement performance.

As specified in the ADAP methodology, the research team developed specific adaptation measures for each design period to mitigate the adverse impacts on pavement performance under RCP 8.5 (the highest impact scenario). Adaptation measures will specifically address the concerns of increased fatigue damage, AC and subgrade rutting under climate change.

Towards the end of the existing pavement's service life, in 2024 or before as warranted by the condition of the existing pavement, it is expected that MaineDOT will undertake rehabilitation measures to restore the functional and/or structural condition of the existing pavement. The rehabilitation will typically involve milling of the AC surface and replacing it with an overlay of the same material type, with or without additional AC thickness for structural enhancements.

The research team proposes an incremental change in the thickness of AC overlay to compensate for the softening of AC layers due to warming trends. The proposed adaptation measures will increase the structural capacity of pavements to compensate for the expected increase in fatigue damage and subgrade rutting, and can be easily incorporated as part of routine structural rehabilitation activities. The adaptation strategies are incremental over time in order to minimize the risk of over-spending on materials during each design period.

The increased susceptibility to AC rutting can be addressed using a combination of structural and material design strategies, as suggested below:

- Make adjustments to the AC mix designs and specifications, such as recommending higher percentages of crushed aggregates and manufactured fines, to improve the aggregate interlock, to improve rutting resistance of asphalt mixtures.
- Adjust the asphalt binder content (e.g. decrease the binder content for pavement layers closer to the surface to control AC rutting while increasing the binder content for layers closer to the bottom to improve the fatigue performance of pavement) and add lime to stiffen the mix.
- Use polymer modified binders such as PG 76-28 that would offset some of the softening effects due to increasing temperature and mitigate the propensity to additional AC rutting. Note that the use of stiffer asphalt binders will reduce the fatigue damage by up to 50 percent.

When pavement reconstruction is considered as a viable option, it is recommended that the MaineDOT evaluate the following strategies:

- Install subsurface drainage features to quickly remove water entering the pavement system. The subsurface drainage can include a combination of: (i) permeable asphalt stabilized base layer[122], (ii) separator layer[123] between the subbase and subgrade, (iii) daylighting[124] of subbase layers, and (iv) edge drains[125] and outlets. An adequate and well-maintained subsurface drainage quickly removes the water from the pavement system that would otherwise contribute to the degradation of pavement quality.
- Consider stabilizing gravel and/or subgrade with asphalt, cement or lime to improve structural integrity and performance of pavement structure. Base and/or subbase stabilization will provide greater resistance against fatigue damage and subgrade rutting. As concluded in a 2006 MaineDOT study,[126] soil cement[127] is a viable option to strength the pavement foundation, particularly where the quality of gravel subbase is an issue.

Further, preserving the existing pavement proactively, such as with the use of seal coats and crack sealing, can help eliminate surface defects and discontinuities, prevent the infiltration of water into the pavement structure, and retard the structural deterioration of pavements. Pavement preservation strategies can also help mitigate the adverse effects of warming trends on pavement performance in the early years (i.e. 2020 to 2040).

In order to estimate the incremental cost of these adaptation strategies, the research team assumed that MaineDOT will mill the top three inches of existing AC and replace with a three-inch overlay in the year 2020. With expected climate change under RCP 8.5, the MaineDOT will have to increase the thickness of AC overlay in accordance with the adaptation measures in Table 19.[128]

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[122] A permeable asphalt stabilized base is a porous pavement layer that is stabilized with an asphalt binder to maintain integrity. The purpose of this layer is to collect water infiltrating the pavement and to move it to edge drains within an acceptable timeframe.

[123] A geotextile-based impermeable layer is placed between the permeable base or subbase and the subgrade. The purpose of the separator layer is to prevent the migration of fine particles that could clog the subbase layer and to provide an impermeable barrier that deflects water from the base horizontally toward the pavement edge.

[124] Daylighting is the process of extending and grading the base and subbase layers laterally to pavement edge, so that the water collected in this layers flow directly into a side ditch.

[125] Edge drains are pipes that run along the pavement length (or traffic direction). The purpose of edge drains is to collect water and feed into outlets. Outlets are pipes that carry water from edge drains to side ditches.

[126] Colson and Peabody, 2006.

[127]Soil-cement is a highly compacted mixture of soil/aggregate, cement, and water. Soil cement provides greater strength and durability to pavement foundation typically at relatively low costs.

[128] Note that the recommended adaptation measures will be applicable even if the thickness of the AC overlay were increased to accommodate future traffic growth.

Table 19: Recommended Adaptation Measures for Fatigue Damage, AC and Subgrade Rutting under RCP 8.5.

Design Period	Recommended Adaptions
2020-39	Increase the thickness of the AC overlay by at least 0.25 inches
2040-59	Increase the thickness of the AC overlay by at least 0.5 inches
2060-79	Increase the thickness of the AC overlay by at least 0.5 inches. Use polymer modified asphalt binder, such as PG 76-28.
2080-99	Increase the thickness of the AC overlay by at least one inch. Use polymer modified asphalt binder, such as PG 76-28.

Table 20 presents the cost estimates of AC overlay[129] for the base case design as well as the recommended adaption measures. As expected, the cost of AC overlay is expected to increase gradually with climate change by up to 35.1 percent in the year 2080.

The research team determined that the cost of the adaptation options for the highest impact scenario (RCP 8.5) is sufficiently low so that an economic analysis of design alternatives for multiple climate scenarios is not necessary. The research team recommends use of the RCP 8.5 adaptation as a cost-effective alternative to cover all possible future climate changes. Per ADAP, the analysis proceeded directly to Step 9.

[129] This cost estimate was based on a unit price of $110 per ton of AC surface mix with PG 64-28 binder. For periods beginning 2060, a cost premium of 20 percent was added to the unit price to include polymer modified asphalt binders.

Table 20: Cost Estimates for the Recommended Adaptation Measure for RCP 8.5.

Design Case	Recommended Adaptions	2020 Dollars	Cost Difference		Net Present Value in 2020 Dollars using 3 percent discount rate[130]
			Cost Increase	Percent Increase	
Typical Overlay	No adaptation measure is implemented.	$4,142,000	N.A.	0.0%	$4,142,000
2020-39	Increase the AC overlay thickness by 0.25 inches in 2020	$4,420,000	$278,000	+6.7%	$4,420,000
2040-79	Increase the AC overlay thickness by 0.50 inches in 2040	$4,697,900	$555,900	+13.4%	$2,243,700
2060-79	Increase the AC overlay thickness by 0.5 inches in 2060	$5,038,500	$896,500	+21.6%	$1,332,400
2080-99	Increase the AC overlay thickness by one inch in 2080	$5,594,300	$1,452,300	+35.1%	$819,100

Step 7: Assess Performance of Adaptation Options

This step was not completed for this analysis because the cost of adapting to the highest impact scenario was minimal.

Step 8: Conduct Economic Analysis

This step was not completed for this analysis because the cost of adapting to the highest impact scenario was minimal (see Step 6).

Step 9: Evaluate Additional Considerations

Generally speaking, the adaptation measure proposed in Step 6 involves a simple and fairly reasonable cost adjustments at the project level. However, although, the cost premium of adaptation measures may be fairly low for an individual project, the effects of climate change will be systemic and statewide, and there may be negative implications on the reliability of network-level pavement performance and programmatic budgetary needs with climate change. If climate change is not properly planned for and conditions and problems are allowed to accumulate, this could affect the overall perception of the driving public. Other considerations

[130] Discounting is a financial technique used to account for the time value of money. A three percent discount rate was used because it is the rate suggested for the U.S. Department of Transportation's (USDOT) Transportation Investment Generating Economic Recovery (TIGER) grant program.

will include the availability of funds and decision-makers' tolerance for risk brought on by the uncertainty of the climate projections.

To mitigate the lost opportunities with WWPs and early SLRs, MaineDOT can explore the feasibility of redrawing freight corridors serving the economy of the Piscataquis County and implementing specialized or dedicated truck corridors for local industries, such as mining, livestock, agricultural and logging industries. In addition, MaineDOT can also explore the possibility for alternative highway funding streams to meet the budgetary shortfalls for pavement strengthening needs.

Step 10: Select a Course of Action

Barring any additional considerations discussed in Step 9, it is recommended that the adaptation options presented in Step 6 be implemented to ensure the pavement system performs adequately in the future.

The specific course of action recommended for this roadway is to opportunistically implement the adaptation measures over time in a cost-effective manner. The first step is to strengthen the pavement structural capacity with thicker overlays during mill and fill rehabilitation cycles. The use of specialty asphalt binder may not be necessary until 2060s, as the agency already uses a stiffer binder grade. If the pavement were to be reconstructed, it is recommended that MaineDOT consider the stabilization of subbase and subgrade, and install an adequate and functional subsurface drainage to expedite the removal of water from the pavement system. Adopting a proactive pavement preservation approach will retard the structural deterioration of pavement, while providing a safe and rideable pavement surface. This phased approach will also allow MaineDOT to track the effectiveness of the measures over time and to only implement the next measure when it is necessary.

Step 11: Develop a Facility Management Plan

Climate change poses no immediate vulnerability to the expected performance of pavements on SR-6/SR-15/SR-16, however, there are potential long-term adverse but manageable consequences on the performance of pavements as discussed in earlier sections. Pavements, as an infrastructure system, involves a complex interaction of multiple factors; similarly, a multitude of strategies can be utilized to address the impacts of climate change on pavement performance. Considering this complexity, a management plan for pavement facilities should have more than the specific measures proposed in Step 6. The facility management plan should adopt a comprehensive and holistic approach that includes good design, construction, and asset management practices.

In line with the above discussion, the following measures are proposed for managing the proposed pavement facility on SR-6/SR-15/SR-16:

- Re-evaluate all future operational decisions, such as the winter weight premium and seasonal load restriction policies, as updated climate change projections become available.
- Re-evaluate the timing and, perhaps, the type of rehabilitation measures to implement as updated climate change projections become available.
- Proactively monitor the condition of pavements. Undertake detailed field investigations, including deflection testing[131] and condition surveys, for pavement reconstruction rehabilitation design.
- Adopt a proactive pavement preservation approach, such as selection of appropriate treatment type and timely application, to make better decisions and help retard the faster progression of distresses.
- Re-evaluate all design-related decisions, including those proposed under Step 6, using newly available climate information during the pavement reconstruction and rehabilitation planning process.

Lessons Learned

This case study provides an engineering analysis of how projected changes in temperature and precipitation trends due to climate change might impact pavement performance for a two-lane rural highway on frost-susceptible soils in the Guilford, Maine area. While the effects of climate change may not be catastrophic in comparison to potential climate change impacts on other critical highway assets with longer expected lifespans, the climate data projections indicate systematic and long-term adverse consequences on the performance of pavements that warrant corrective action.

As various climate change scenarios indicate, ambient temperatures are expected to increase steadily over the course of 21st century. The projected changes in climate patterns will result in higher pavement temperatures and shorter freezing seasons, which could lead to a modest increase in flexible pavement distresses including fatigue damage and rutting. However, due to lower depths of frost penetration, there may be some benefits associated with lower serviceability loss due to frost heave. Overall, the increase in predicted pavement distresses can be handled using increases in pavement thicknesses and improvements to material selection and mix design criteria, construction practices, and specification requirements.

Climate change will have implications for seasonal truckload restriction policies. Shorter freezing seasons will lead to shorter and lighter allowances in WWPs. With shorter freezing seasons, there will be fewer opportunities for MaineDOT and the trucking industry to take advantage of lower

[131] Deflection testing is a non-destructive method to measure deflections on the pavement surface. Deflection measurements are primarily used to evaluate the structural capacity of in-place pavements.

damage potential of pavements under frozen conditions. Similarly, to accommodate the early on-set of spring thaw, there will be a need for an early posting of spring load restrictions. Furthermore, climate change may exacerbate pavement damage caused by trucks. Thus, there is a need to evaluate the economic impacts of changing seasonal load restriction policies on the trucking industry system-wide in terms of repurposing freight networks, truck user fees, and pavement strengthening measures.

The primary lesson learned from this case study is the need to monitor changes in climate trends and periodically re-evaluate all future design decisions and seasonal load restriction policies as updated climate change projections become available. There is a need to move away from the sole dependence on historical climate records such as the "State of Maine Design Freezing Index". This chart appears to have been prepared from data that is now decades old. Prescriptive design recommendations will need to move toward a more project-level assessment and decision-making.

While the proposed adaptation strategies are routinely used by state highway agencies, there is a cost premium, at least in the short run, to adopt enhancements in design and construction practices. The cost premium for such upgrades may be fairly low at a project level; however, since the effects of climate change will be systemic and statewide, the budgetary implications of adopting enhancements at the agency level must be investigated. For instance, the cost premium associated with upgrading the asphalt pavement by 0.5 inch may be low at a project level; however, when required on all projects, the cumulative cost premium will most likely impact the capital improvement or maintenance budget significantly at the district or state level. Using a proactive approach to preservation, maintenance, and renewal decisions will offset some of the budgetary constraints.

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