U.S. Department of Transportation
Federal Highway Administration

TEACR Engineering Assessment

Temperature and Precipitation Impacts to Pavements on Expansive Soils: Proposed State Highway 170 in North Texas

This is one of nine engineering case studies conducted under the Transportation Engineering Approaches to Climate Resiliency (TEACR) Project.[1] This case study focuses on the impacts of changes in temperature and precipitation on pavements constructed over expansive soils.

Case Study Snapshot

Purpose: Evaluate potential impacts of projected changes in temperature and precipitation on pavement performance.

Location: State Highway 170, near Dallas, Texas

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: There will be a steady increase in ambient temperature and aridity over the course of the 21st century. These trends will result in a modest increase in pavement distresses.

Key Lessons: There is a need to monitor climate trends and re-evaluate future design related decisions using newly available climate information. Since the impacts are systemic, there is a need to evaluate the economic consequences of implementing adaption measures.

Overview

This case study considered how changes in future temperature and precipitation could affect pavement performance on State Highway 170, near Dallas, Texas.

Environmental conditions have a significant effect on the performance of both flexible[2] and rigid[3] pavements. Seasonal variations in climate factors affect both material properties and integrity of pavement layers and subgrade.[4] As a result, climate fluctuations can cause pavement rutting, cracking, and heaving, all of

[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] A flexible pavement is constructed with asphalt concrete (commonly simply called "asphalt") comprised of a surface layer resting on a base layer, typically made of crushed stone, crushed gravel, lime or cement-treated materials, and a subbase placed over a prepared soil bed. Flexible pavements are typically rehabilitated with either an asphalt concrete or Portland cement concrete (PCC) overlay. A subbase is a pavement layer that is placed below the base layer. The use of subbase is optional and often driven by economic considerations to substitute a portion of more expensive base materials with locally available and less expensive granular materials.

[3] A rigid pavement is constructed with Portland cement concrete (PCC) 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. Rigid pavements are typically rehabilitated with either an asphalt concrete or Portland cement concrete overlay. Commonly used rigid pavement types are Jointed Plain Concrete Pavement (JPCP), Jointed Reinforced Concrete Pavement (JRCP), and Continuously Reinforced Concrete Pavement (CRCP). The Texas Department of Transportation (TxDOT) predominantly uses the CRCP type of rigid pavements.

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

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which can affect the structural performance, riding comfort and ultimately, the safety of the road.

Most pavement design approaches recognize the importance of climate factors in materials selection, construction practices, and structural thickness determination; however, these design methodologies have traditionally relied on historical weather records to evaluate and incorporate climate-related considerations in design decisions. Therefore, pavement design decisions, as currently undertaken, do not take into account future changes in temperature and precipitation. This case study considered whether current pavement design in the study area is adequate in light of projected changes in temperature and precipitation during the expected useful life of the road.

This study considers flexible and rigid pavement designs for State Highway 170 (SH-170), a proposed new highway in the Dallas-Fort Worth Metroplex. The study area is known for the presence of expansive clay soils that are highly sensitive to fluctuations in moisture (i.e., they swell when wet and shrink when dry). Pavements constructed on expansive soils are generally more susceptible to significant loss in smoothness,[5] whether heavily trafficked or not, due to wet and dry seasonal cycles. Furthermore, the expansive behavior of clays in the Dallas-Fort Worth area, combined with the systematic effects of climate change, may significantly affect future pavement performance. The objective of this case study is to investigate how considerable variations in precipitation and temperature due to climate change may affect the performance of flexible and rigid pavements on expansive soils and investigate options to mitigate the risk and adapt to climate changes.

Expansive clays are not unique; such soils are widely prevalent in the continental U.S., particularly in the southwest, western mountains, central plains, and southeast. Figure 1 shows the prevalence of expansive soils in the conterminous U.S.[6] More than 36 states have some form of expansive clays; however, their swelling potential may vary depending upon the composition of clay minerals, the distribution of soil particle sizes, variations in soil moisture content, and the depth below the soil surface. According to a 1981 estimate by U.S. Department of Housing and Urban Development (HUD), expansive soils were blamed for approximately $9 billion in damages to houses, other buildings, roads, pipelines, and other structures.[7] Though commonly occurring elsewhere, the expansive soils in the Dallas-Fort Worth area are notable for their mineralogy and their potential for swelling and shrinkage.

[5] Smoothness is a measure that reflects irregularities in the pavement profile. Smoothness is often defined by a standard measure called the "International Roughness Index" or IRI. The higher the IRI is, the rougher the pavement surface is. In layman's terms, it is a measure of how bumpy the road is.

[6] Olive, et al, 1989.

[7] Miller, 1997.

The differential soil movements in expansive soils may have damaging effects on structural performance and ride quality of highway pavements, including waves of distortion or random unevenness on the roadway surface, localized pavement failures due to loss in subgrade soil support, and longitudinal[8] and transverse[9] cracks causing discontinuities in the roadway surface and between other roadway structures.

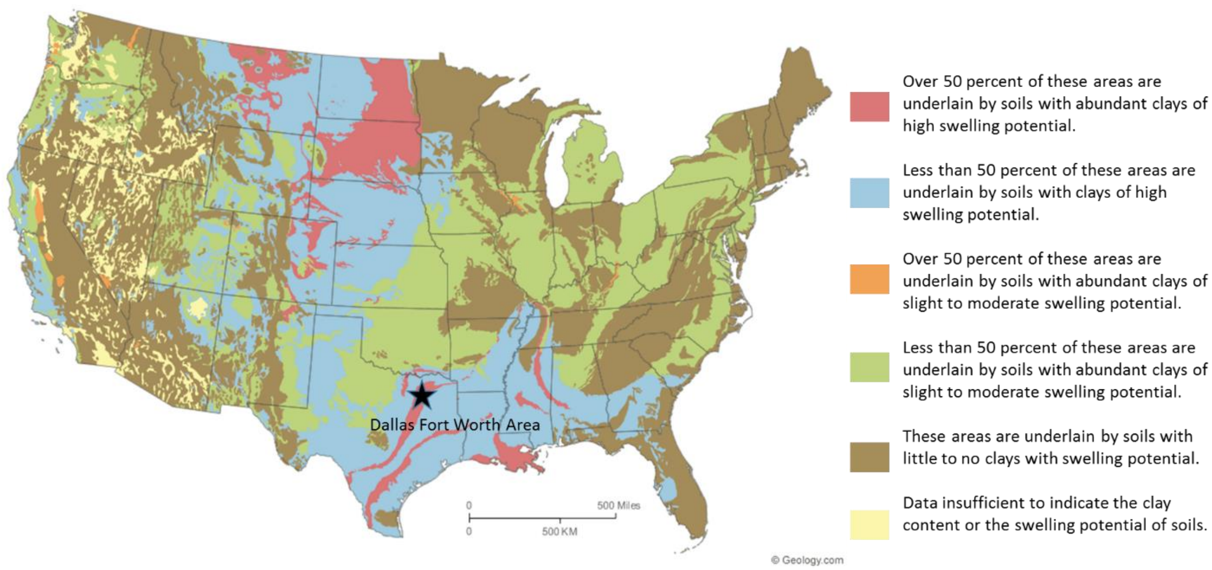

Figure 1: Expansive Soils in the U.S.[10]

Figure 2 presents examples of upward bulges or blowups on pavement surfaces caused by expansive soils.[11] Not only does this damage to pavement often require expensive fixes to restore ride quality and structural adequacy, they may also lead to operational and safety issues and potential vehicle damage.

[8] Longitudinal cracks are fissures or discontinuities in the pavement surface that run parallel to the direction of traffic. Longitudinal cracks can appear on or outside the wheel path.

[9] Transverse cracks are fissures or discontinuities in the pavement surface that run perpendicular to the direction of traffic. Transverse cracks in flexible pavements can occur due to low temperature or shrinkage of asphaltic material, while in rigid pavements, transverse cracks can occur due to inadequacy of the pavement structure to withstand loading and environmental stresses.

[10] Source: Olive et al, 1989.

[11] Source: Talluri, et al, 2013.

Figure 2: Examples of Damaging Effects of Expansive Soils on Pavements.[12]

As noted above, climate change will affect other elements of pavement design for SH-170 beyond just the soil moisture considerations. These effects were considered as well in order to develop a holistic understanding of future pavement performance, including the impacts of load-carrying capacity and rutting potential of the pavement structure.

The following is a complete list of items the research team has investigated:

- Changes in the Thornthwaite Moisture Index (TMI)[13] and their impacts on soil support conditions
- Changes in shrink-swell potential of soils that would contribute to loss in pavement smoothness
- Changes in performance grade requirements of asphalt binder[14] and dynamic modulus of asphalt concrete[15] mixtures due to changes in temperature

Key Pavement Distresses Related to Temperature and Precipitation

Rigid Pavement (CRCP only)

Shrinkage cracking: Hairline cracks formed during concrete setting and curing

Punchout: Localized slab portion broken into several pieces

Asphalt Pavement (AC)

Fatigue cracking: Interconnected cracks caused by the "fatigue" failure of the surface or base under repeated traffic loading

Longitudinal cracking: Cracks parallel to the pavement centerline

Rutting: Surface depression in the wheel paths

Transverse cracking: Cracks perpendicular to the pavement centerline

Source: Pavement Interactive, Pavement Management. http://www.pavementinteractive.org/category/pavement-management/pavement-distresses/

[12] Source: Talluri, et al, 2013.

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

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

[15] Asphalt concrete (commonly referred to simply as bituminous or AC), a key component of flexible pavement design, is the term given pavement comprised of a mixture of asphalt, aggregate, and other admixtures as may be required. 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

- Resulting changes in pavement distresses. For flexible pavements, (1) load-related fatigue cracking,[16] (2) subgrade rutting,[17] and (3) AC rutting[18] were considered. For rigid pavements, punchout[19] potential of continuously reinforced concrete pavements (CRCP)[20] was investigated.

Since SH-170 is yet to be fully designed and built, the study of these items provides an opportunity to *help influence the eventual design*. Key findings of this analysis that may be considered in future designs include:

- The climate change projections indicate a steady decrease in TMI over the next decades, suggesting an increase in aridity albeit at different rates under the different climate change scenarios evaluated. Increasing aridity and resulting reduction in soil moisture will potentially contribute to a marginal increase in soil support conditions in the long term.
- The drier condition of the subgrade will alter the potential of expansive soils towards more swelling and less shrinkage; however, the total deformation due to swelling and shrinkage may remain the same with no change in pavement smoothness lost over its lifetime. The increasing aridity may increase the likelihood of lateral vegetation growth with roots penetrating into the pavement structure and subgrade, and thus contributing to irregular cracking patterns and smoothness loss.

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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). The loading time is based on the average pulse time that a vehicle tire would cause stresses on the AC material. The loading time is a function of traffic speed and the depth below the pavement structure. i.e., at a given traffic speed, the AC material near the pavement surface will experience shorter loading time than the same material that is present deeper from the surface. Similarly, a given pavement depth, the AC material will experience shorter loading time at faster speeds than at slower speeds.

[16] 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.

[17] 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 permanent deformation or surface depression in the wheel paths of the roadway surface that is caused by consolidation of subgrade under repeated traffic loading. Significant rutting can lead to major vehicle safety issues and structural failures.

[18] AC rutting occurs when problems with the mix design are the cause of the rutting as opposed to issues with the subgrade.

[19] Punchouts refer to localized slab failures characterized by closely spaced transverse cracks often connected by short longitudinal cracks and joints. Punchouts are the predominant structural distress in continuously reinforced concrete pavements (CRCP) that develop over time under environmental and traffic loads.

[20] CRCP is a type of rigid or Portland cement concrete (PCC) pavement constructed with steel reinforcing bars placed within the concrete along the entire length of the pavement. CRCP is characterized by the absence of constructed transverse joints. CRCP naturally forms tight transverse cracks to evenly transfer loads. The following State highway agencies have used CRCP since the 1960s or 1970s - Illinois, Texas, Virginia, Oregon, Oklahoma, North and South Dakota. (Source: FHWA, 2012).

- The projected increase in air temperature may necessitate an increase in the performance grade of the asphalt binder. While no changes are required in low temperature grade of asphalt binders, there may be a need to upgrade the high temperature grade in the future. Without increasing the performance grade, more permanent deformation, load-related cracking and thermal cracking could occur.
- Both flexible and rigid pavements will perform less reliably in the future, as there will be a modest increase in key pavement distresses (see text box on Key Pavement Distresses) with changing climate conditions. When design decisions, such as the selection of asphalt binder, are made using historical climate data, flexible pavements will be more affected as the bituminous layer gets softer and less stiff due to rising temperatures. The softening of bituminous layers will increase the susceptibility of flexible pavement to fatigue cracking and rutting. For the CRCP type of rigid pavement, there will be an increase in punchout potential due to increasing temperatures and decreasing ambient relative humidity.[21]
- TXDOT will have to build stronger pavements, using a combination of strategies such as increasing the thickness of pavement, stiffer asphalt binders, additional steel content in CRCPs, stabilization of base and subgrades, and intervene more frequently to undertake pavement maintenance and rehabilitation[22] activities. These actions may result in an overall increase in their program expenditures.

This analysis is designed to address gaps in the assessments performed to date to consider climate change implications for engineering analyses, including the following:

- Selecting climate information, specifically changes in temperature and moisture
- Changes in temperature and precipitation patterns and soil moisture conditions
- Combining historical climate data with projected future climate changes
- Secondary impacts of climate stressors
- Incorporating climate change into design practices
- Phased adaptation strategies
- Climate change uncertainty and cost of adaptation

The remainder of this case study is organized around the 11 steps of the Adaptation Decision-Making Assessment Process (ADAP), as shown in Figure 3.

[21] Relative humidity is a measure of the amount of water vapor in the air. It is reported as a percent of the amount of water vapor one would need to achieve saturation at the same temperature.
[22] Pavement rehabilitation is the act of restoring a pavement to its former (new) condition.

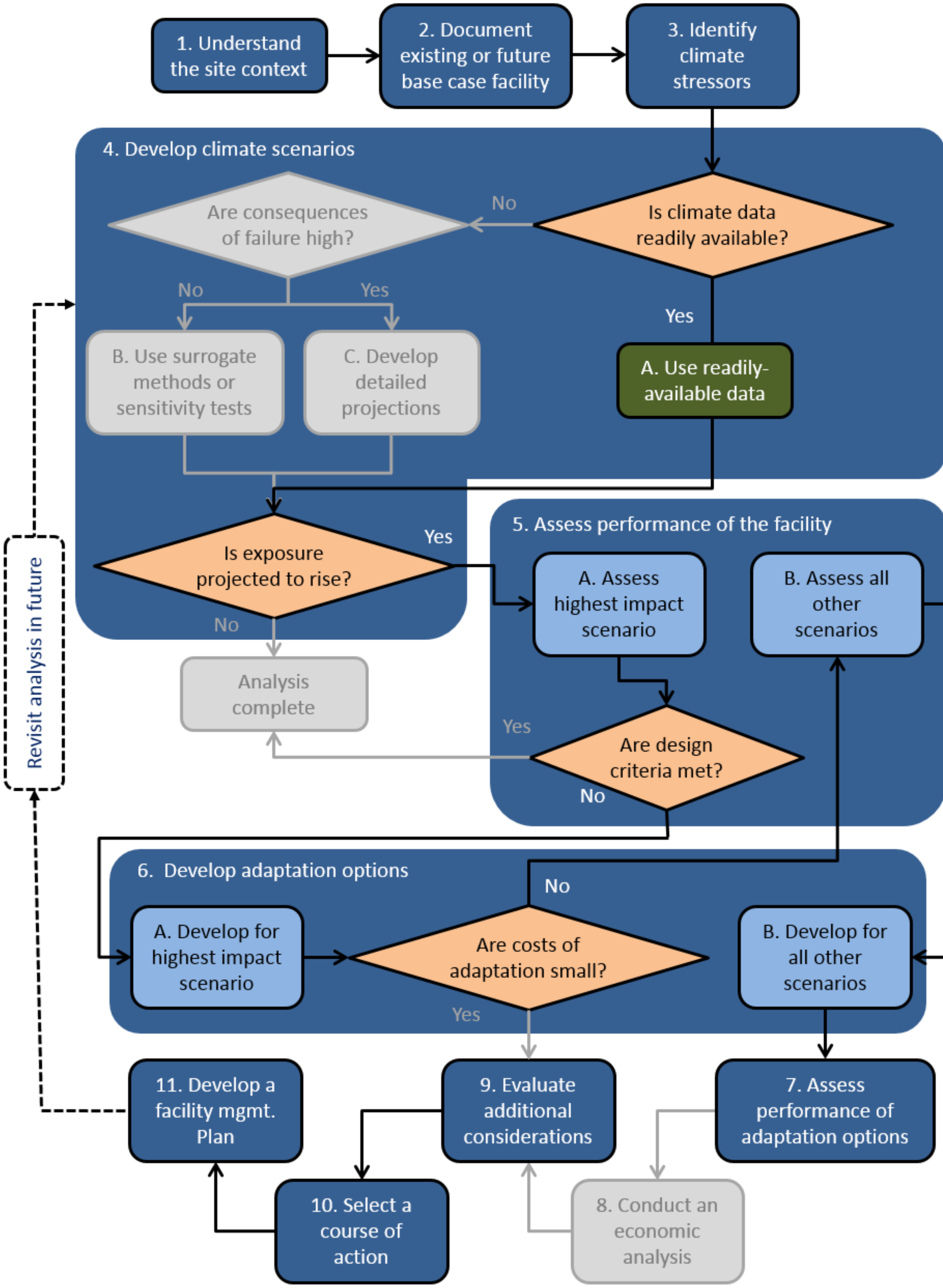

Figure 3: Adaptation Decision-Making Assessment Process (ADAP) Used for this Analysis (steps not completed are indicated in gray).

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Contents

Details of the Analysis

Step 1. Understand the Site Context

The proposed roadway selected for the case study, SH-170 (also known as the Alliance Gateway Freeway), is to be located in the north of the Dallas-Fort Worth Metroplex (see Figure 4).

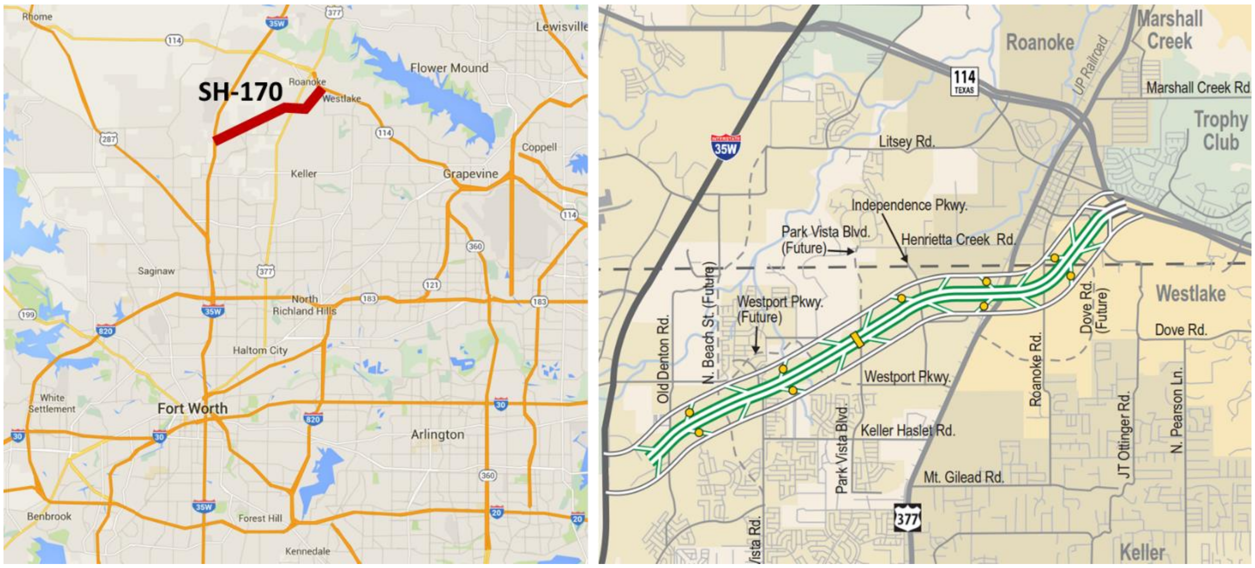

Figure 4: Location of SH-170 in the Dallas-Fort Worth Metroplex.[23]

The SH-170 corridor was originally constructed in the early 1990s by the Texas Department of Transportation (TxDOT) as frontage roads between Interstate 35W (I-35W) and SH-114. The frontage roads were constructed with a large median to allow space for future highway construction. The frontage roads are currently serving as the primary arterial to the substantial surrounding residential and industrial development in the vicinity.

The North Texas Tollway Authority (NTTA), a public entity of the State of Texas, now plans to construct six general purpose lanes between the existing frontage roads along a new alignment. The upgraded SH-170 corridor will be 6.5 miles long and will provide a limited access highway connection between SH-114 in Roanoke, Denton County and I-35W in Tarrant County. SH-114, the Northwest Parkway, is an important metro area expressway and I-35W, the North Freeway, is the western (Fort Worth) branch of I-35, a major north-south freight corridor in the Midwest connecting Laredo, Texas and Duluth, Minnesota. Figure 5 presents photographs of the existing frontage roads and the undeveloped land in the median where the future facility will be built. TxDOT is the owner of the frontage roads; however, the NTTA will build, maintain, and operate the new facility by raising revenues from tolls.

[23] Sources: Google Maps and NCTCOG, 2011.

Figure 5: SH-170 Frontage Roads and Future Highway Corridor.[24]

The project is currently in the preliminary design and environmental review stage. FHWA has provided a preliminary approval of environmental documents as being satisfactory for further processing (SFP). NTTA now plans to conduct public hearings and proceed further to receive final environmental clearance for the project. Per the 2013 estimates of the North Central Texas Council of Governments (NCTCOG), SH-170 currently carries an average daily traffic of 42,298 vehicles per day. Truck traffic on the SH-170 corridor is estimated at 2.4 percent of average daily traffic. Assuming an estimated annual linear growth rate of 2.46 percent, future year traffic is projected to be 58,400, 82,800, and 93,600 vehicles per day for 2020, 2040 and 2050, respectively.[25]

The geologic setting of the Dallas-Fort Worth area makes this corridor an ideal candidate for investigating the impacts of climate change. The SH-170 corridor is located in the Woodbine Sandstone formation and close to the Eagle Ford Shale sedimentary formation. Both of these

[24] Image Source: WSP | Parsons Brinckerhoff.
[25] North Central Texas Council of Governments, 2011.

geological formations are known to have highly expansive plastic[26] clays containing active clay minerals which shrink and become hard with drying then swell and turn soft with moisture.

Step 2. Design Base Case Facility

The pavement structure for the SH-170 facility is yet to be designed and built. Therefore, the research team developed typical designs using TxDOT-approved design methods, historical climate data, typical inputs of materials and design parameters, and forecasted traffic. Since the details of pavement design have not been finalized, both major pavement types—flexible and rigid pavement— were considered. The standard pavement design methodology recommends that an appropriate pavement type alternative should be selected based on life-cycle cost analysis. The life-cycle cost analysis considers both the initial and long-term costs for pavement type, including costs of building, maintaining, and rehabilitating the pavement structure while ensuring a desired level of performance[27] over a chosen analysis period.[28]

Each pavement type alternative has different performance patterns (in terms of longevity and distress types that develop over time) and life-cycle strategies.[29] Figure 6 shows a typical pavement performance timeline and life-cycle strategy. Different maintenance and rehabilitation treatments will be applied to correct or preserve the pavement for safe and efficient use (per TxDOT's standard practice, the preferred rehabilitation typically involves mill[30] and overlay[31] for flexible pavements and an overlay with bituminous layer for rigid pavement). Given the differences in performance among pavement alternatives, it is necessary to distinguish the cost-effectiveness of pavement type alternatives to ensure sound economic decisions. Hence, the analysis period must be sufficiently long to include at least one future major rehabilitation event when developing life-cycle strategies for each alternative.

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[26] 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.
[27] Pavement performance is a measure of the adequacy with which the pavement fulfills its intended purpose.
[28] An analysis period is the time period used for comparing pavement-type alternatives. An analysis period may contain several maintenance and rehabilitation activities during the life-cycle of the pavement being evaluated and therefore an analysis period is not the same as the design period.
[29] Life-cycle strategies include identification of the initial pavement section together with the timing and type of future maintenance and rehabilitation activities.
[30] Milling is the process of grinding and removing existing materials from the top layers of the pavement surface.
[31] Pavement overlays are new layers of asphalt concrete or Portland cement concrete (PCC) placed on the existing pavement structure. Pavement overlays are placed for either fixing functional (non-structural) deficiencies (e.g., loss in pavement smoothness) or both functional and structural inadequacies.

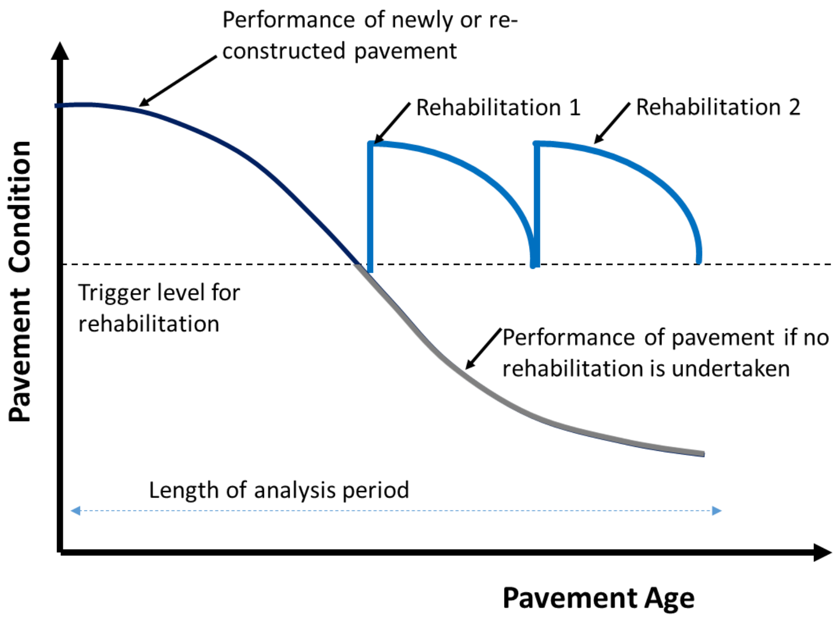

Figure 6: Typical Pavement Performance Timeline.

TxDOT recommends 20 and 30-year design periods[32] for structural design of flexible and rigid pavements, respectively, and typically uses a 30-year period to compare the lifecycle costs of flexible and rigid pavement alternatives. TxDOT's typical analysis period is not long enough to include at least one future major rehabilitation event for each pavement type. However, the research team believed it was important to do so and chose an analysis period of 50 years, 2020[33] through 2069, for this case study. Climate change adaptation analyses for actual projects should generally align with this approach for setting an analysis time period. That said, for research purposes, certain aspects of this case study also looked farther out to the end of the century to investigate longer-term climate impacts.

An analysis period of 50 years is deemed appropriate for multiple reasons: (1) the analysis period will be sufficiently long to include two major rehabilitation events for flexible pavements and at least one event for rigid pavements, (2) the end of the 50-year period will be an appropriate time for pavement reconstruction as any further application of major rehabilitation treatment may not be adequate and cost-effective, and (3) longer analysis periods may require extrapolation of future traffic forecasts that may lead to erroneous design decisions. The 50-year analysis period also facilitates an "apples to apples" comparison of two pavement type alternatives with different design periods and rehabilitation actions.

[32] A design period is the time period between new construction (or reconstruction) and a major pavement rehabilitation. It is the time period for which a pavement structure is designed to keep structural distresses under a given threshold.

[33] 2020 was selected as the assumed completion date for the project.

The remainder of this section considers each element in the design process. First, considerations that apply to both the flexible and rigid pavement design options are presented in the following paragraphs. Following this, specific design considerations for the flexible pavement design option are discussed followed by those for the rigid pavement design option.

Considerations for both the Flexible and Rigid Pavement Design Options

This section focuses on design considerations that pertain to both the flexible and rigid pavement design options: namely, estimating the subgrade support conditions and specifying asphalt binder performance grades.

In a pavement structure, the purpose of the subgrade is to provide foundational support against the excessive buildup of irrecoverable deformation under repeated traffic loads. The level of foundational support that the subgrade provides will mostly vary by its soil characteristics and moisture content, and is defined in terms of resilient modulus.[34]

While laboratory testing is generally recommended for measurement, the resilient modulus of subgrade can be estimated reasonably using soil index properties[35] such as the percent of soil particles passing 75 micron (often referred to as number 200 sieve) and 2 micron standard sieves,[36] liquid limits,[37] plastic limits,[38] and the plasticity index.[39] For the SH-170 site, the soil resilient modulus values were estimated using soil index properties obtained from the Arizona State University (ASU) Soil Unit Map Application website.[40] Table 1 summarizes the soil properties for the SH-170 site.

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[34] 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.

[35] Index properties are the properties of soil that help in their identification and classification.

[36] Standard sizes of sieves are used to measure the distribution of particles in a soil sample. The percent of particles passing the 2 micron sieve is typically used to indicate the plasticity of soils.

[37] The liquid limit is the water content at which soil begins to behave as a liquid material and begins to flow.

[38] The plastic limit is the minimum water content at which soil begins to be plastic. In material science terminology, the plasticity describes the deformation potential of a material under an applied force and the deformation is fully or partially irreversible when the applied force is withdrawn. In soil mechanics, the term plasticity indicates the deformation potential of a soil to readily become mud by losing its cohesion and resistance against sliding under wetting. Plastic is also a state between the soil's solid or semi-solid state and liquid state.

[39] The plasticity index is the numerical difference between liquid and plastic limits that indicates the range of water content over which the soil remains plastic.

[40] The ASU soil unit map application website was developed for the National Highway Cooperative Research Program (NCHRP) Project 9-23B. Available at: http://nchrp923b.lab.asu.edu/.

Table 1: Summary of Study Area Soil Profile Properties

Soil Sublayer	AASHTO [41] Soil Class	Particle Size Distribution		Soil Consistency			Soil Resilient Modulus, psi[42]
		Passing #200 %	Passing 2μ[43] (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	
1 (Top 32 Inches)	A-7-6	92.5	50	58.5	20	38.5	4,922
2 (32 to 80 Inches from Surface)	A-7-6	84	50	42.5	18.5	24	6,958

Note that the resilient moduli of soil sublayers one and two, as estimated by the ASU Soil Unit Map, are 4,922 and 6,958 pounds per square inch, respectively. The resilient moduli estimates have equivalent California Bearing Ratios (CBR)[44] of 2.8 and 4.8 and weighted plasticity indices (wPI)[45] of 38.5 and 20.2. The resilient modulus estimates of soil sublayer one are also consistent with TxDOT's Texas Triaxial Class[46] estimate of 5.0 for the Tarrant County-Denton soil series.

While the water table elevation is also an important consideration for subgrade support, the average water table is typically below 200 feet in the Dallas Fort Worth area, well below the need for concern in pavement design.

Considerations for Flexible Pavement Design Option

This section presents designs for both the initial (new) flexible pavement construction and subsequent rehabilitations.

Asphalt Binder Performance Grade

For flexible pavement design, the research team considered which asphalt binder grade would be an appropriate consideration for this road under current conditions. The selection of an asphalt binder grade is an integral component of pavement mix designs to ensure a specified level of pavement performance. The performance grade methodology allows the user to select a binder grade that is appropriate for their specific combination of climate, traffic loading that the pavement structure is expected to carry, and desired reliability.[47] An appropriate grade of asphalt binder is critical to address the concerns of permanent deformation, load-related fatigue

41 AASHTO stands for the American Association of State Highway and Transportation Officials.
42 Psi stands for pounds per square inch.
43 μ is the symbol used to abbreviate microns.
44 CBR is defined as the penetration resistance of a subgrade soil relative to a standard crushed rock and is used to indicate the relative load bearing capacity of soil.
45 The wPI is a product of the percent of soil particles passing a number 200 sieve and the plasticity index.
46 Texas Triaxial Classification is a method of soil classification adopted by TxDOT.
47 Reliability in pavement design is the probability that the pavement will perform as intended under the design traffic loading and other crucial design inputs (material properties, environmental factors, etc.).

cracking, and low temperature cracking. The degree days[48] greater than 50° Fahrenheit and a target rut depth are used to identify an initial performance grade binder at 50 percent reliability. The base performance grade binder is then adjusted for higher reliability, depth of pavement, and traffic levels to determine the recommended grade.

For any roadway project, the performance grade of asphalt binder is selected based on its ability to satisfy high and low pavement temperature needs of the project location. The asphalt binder selection methodology requires to make additional adjustments to account for higher reliability, additional damage potential under slower traffic and higher traffic volume, and the depth from pavement surface where the bituminous layer will be placed.

For the SH-170 project, the recommended binder grade to meet the expected high pavement temperature under today's climate conditions (and over the long term if these conditions prevail in the future) at 98 percent reliability is PG 70-YY for AC surface layer and PG 64-YY for lower layers[49], while the recommended binder grade to meet the expected low pavement temperature needs is PG XX-10 at 50 percent reliability and PG XX-16 at 98 percent reliability.

Per the estimates of FHWA's Long Term Pavement Performance Bind (LTPPBind) 3.1 software, the design low pavement temperatures at 50 and 98 percent reliability are 4.9° Celsius (40.8° Fahrenheit) and 11.4° Celsius (52.5° Fahrenheit), respectively, in today's climate. The warmest low temperature binder grade that is readily available in the Dallas-Fort Worth market is PG XX-10. PG XX-10 will satisfy the low temperature requirements of the Dallas-Fort Worth area at 94.65 percent reliability. Therefore, considering market factors such as price and availability, it is reasonable to assume that any locally available binder grade would satisfy the low temperature design requirements for the SH-170 project.

New Construction Design

The flexible pavement design alternative is a conventional AC pavement that includes two bituminous layers with a flexible base supported on a lime stabilized subgrade. The pavement

[48] Degree days are the accumulated total number of days with positive or negative differences between daily temperatures and a base temperature. In this case, degree days are calculated based on the positive differences between daily mean temperature and 50° Fahrenheit.

[49] The base binder grade recommended by TxDOT for both Tarrant and Denton counties at 50 percent reliability is PG 64-YY. Upon appropriate adjustment for traffic and design pavement layer thickness, PG 70-YY is recommended for any AC surface layer of the base case pavement design, while PG 64-YY will be adequate for lower AC layers. High temperature performance grade of asphalt binder was adjusted for traffic volume exceeding 3 million cumulative 18 kilo pound (80 kilo newton) equivalent single axle load applications, while slow traffic is not an issue at SH-170.

structure was designed to carry a design traffic of 3,886,000 flexible pavement[50] equivalent single axle load (ESAL)[51] repetitions over TxDOT's recommended 20-year design period.

Flexible Pavement System 21 (FPS-21) is TxDOT's approved method for flexible pavement structural design and was used to develop the base case design for this case study.[52] The key inputs used in FPS-21 for this case study include the following:

- Design reliability[53] = 95 percent
- Initial serviceability[54] = 4.5
- Terminal serviceability[55] = 2.5
- Length of design period = 20 years
- Design 18 kilo pound (kip) ESAL = 2.72 million (after adjustments for lane distribution)
- Subgrade resilient modulus = 4,900 pounds per square inch

Figure 7 presents the final design cross-section of the flexible pavement structure.

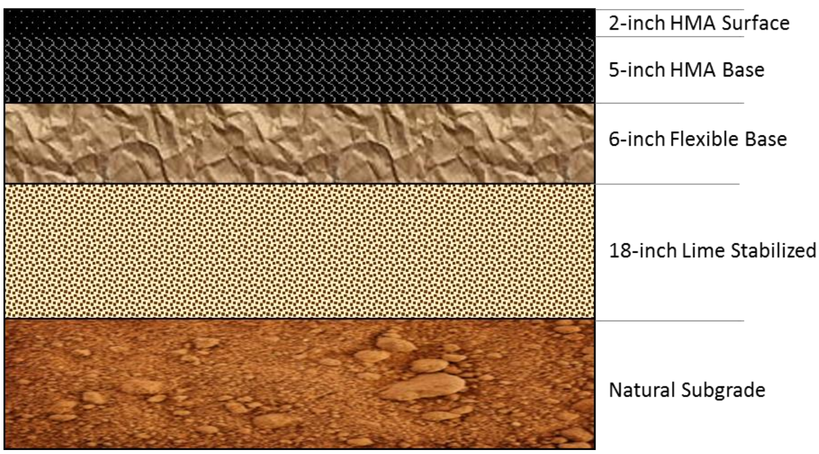

Figure 7: Base Case Flexible Pavement Design Structure.[56]

[50] For a given mix of vehicles, equivalent single axle repetitions are calculated differently for flexible and rigid pavements, since the damage caused by a single pass of a standard axle is not the same for flexible and rigid pavements.
[51] The ESAL is the equivalent number of repetitions of an 18 kilo pound (kip) standard axle load that causes the same damage to the pavement caused by one-pass of the axle load in question.
[52] TxDOT Pavement Design Guide, 2011.
[53]Design reliability is the probability that a given pavement design will last for the anticipated design life.
[54] Serviceability is the ability of the pavement to provide a safe and comfortable ride to users. Serviceability ranges from five (perfect) to zero (impassable). Initial serviceability is the highest practicable index score achievable after new pavement construction, reconstruction, rehabilitation, or resurfacing.
[55] Terminal serviceability is the lowest serviceability index that will be tolerated before rehabilitation, resurfacing, or reconstruction becomes necessary. The value chosen generally varies with the importance or functional classification of the pavement.

[56] Image Source: WSP | Parsons Brinckerhoff.

The proposed as-designed pavement structural cross-section includes:

- A two inch surface course with dense graded AC similar to TxDOT Standard Bid Item 340/341 Type D mix with PG 70-YY binder
- A five inch base course with dense graded AC similar to TxDOT Standard Bid Item 340/341 Type B mix with PG 64-YY binder
- Six inches of flexible base of TxDOT Standard Bid Item 247 material with a design resilient modulus of 40 to 70 kilo pounds per square inch
- 18 inches of lime stabilized subgrade of TxDOT Standard Bid Item 260 material with a design resilient modulus of 30 to 45 kilo pounds per square inch
- Natural subgrade

Rehabilitation Design

Major rehabilitation activities are proposed for the base case flexible pavement design in the years 2040 and 2055. Each rehabilitation activity would involve milling off the top two inches of the existing bituminous layer and resurfacing with a new two inch overlay of TxDOT's standard dense-graded AC surface mix. The proposed time interval between the two structural rehabilitation events is 15 years, which is consistent with TxDOT's typical service life[57] estimate for AC structural overlays. It is expected that TxDOT's routine, non-structural maintenance and rehabilitation activities will be undertaken during the life-cycle of the proposed pavement.

It is expected that TxDOT's routine, non-structural maintenance and rehabilitation activities will be undertaken during the life-cycle of the proposed pavement. Also, corrective actions, such as full-depth repairs, are assumed to be undertaken before the placement of any rehabilitation overlays. Before a final comparison is made, these costs would typically be included in the life-cycle cost analysis using TxDOT's standard procedures. The proposed life-cycle activities and capital costs for the base case design are presented in Table 2. The net present value of the total life-cycle capital costs are approximately $78.9 million for the flexible pavement design alternative.

[57] Service life is the period of time from completion of construction until the structural integrity of the pavement is determined to be unacceptable and rehabilitation/replacement is required.

Table 2: Summary of Life-cycle Activities and Capital Costs for the Base Case Flexible Pavement Design Alternative.

Year	Activity	Capital and Maintenance[58] Cost Estimates	Capital Cost Estimates in 2020 Dollars (3% Discount Rate[59])
2020	**New Construction** • 2 inch AC surface with PG 70-YY asphalt binder • 5 inch AC base • 6 inch flexible base 18 inch lime stabilized subgrade on natural subgrade	$62,608,000	$62,608,000
2025	Crack Sealing	$47,000	$41,000
2028	Seal Coat	$1,172,000	$925,000
2031	Crack Sealing	$47,000	$34,000
2034	Thin Overlay	$2,862,000	$1,892,000
2037	Crack Sealing	$47,000	$28,000
2040	**First Rehabilitation Overlay** Mill top 2 inches and place an AC overlay of 2 inches with PG 70-YY	$13,786,803	$7,633,000
2045	Crack Sealing	$47,000	$22,000
2048	Seal Coat	$1,172,000	$512,000
2055	**Second Rehabilitation Overlay** Mill top 2 inches and place an AC overlay of 2 inches with PG 70-YY	$13,786,803	$4,900,000
2060	Crack Sealing	$47,000	$14,000
2063	Seal Coat	$1,172,000	$329,000
2070	**End of Analysis Period**		
		Net present value of total life-cycle costs	$78,938,000

Considerations for Rigid Pavement Design Option

This section presents designs for both the initial (new) rigid pavement construction and its rehabilitation.

New Construction Design

The rigid pavement design alternative includes a CRCP with a one inch bituminous bond breaker[60] and six inch cement stabilized base on lime stabilized subgrade (see Figure 8). The pavement

58 Maintenance costs for flexible pavements were calculated using the following unit cost assumptions: crack sealing at $750/lane mile, seal coat @ $2.65/SY, and thin overlay at $6.47/SY.

59 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 (Source: USDOT, 2016).

60 A thin layer of asphalt concrete serves as a bond breaker to separate the overlaid CRCP from the underlying cement stabilized layer. The AC interlayer prevents the two rigid pavement layers from behaving monolithically

structure was designed to carry a design traffic of 7,198,000 rigid pavement ESAL repetitions for a 30-year period from 2020 to 2050. The American Association of State Highway and Transportation Officials (AASHTO) 1993 method, approved by the TxDOT Pavement Design Manual, was used for pavement structural design.[61]

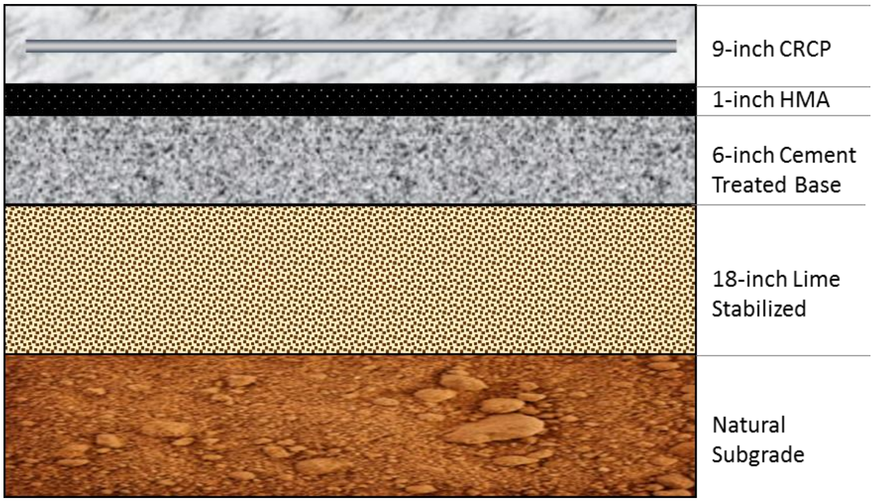

Figure 8: Base Case CRCP Pavement Design Structure.[62]

The key inputs used in the CRCP design include:

- Design Reliability = 95 percent
- Initial Serviceability = 4.5
- Final Serviceability = 2.5
- Length of Design Period = 30 years
- Design 18 kip ESAL = 5.04 million (after adjustments for lane distribution)
- Modulus of subgrade reaction[63] (k) = 300 pounds per cubic inch
- Portland cement concrete (PCC) modulus of elasticity[64] = 5 million pounds per square inch
- PCC modulus of rupture[65] = 620 pounds per square inch

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and provides the necessary support to the CRCP overlay. This reduces the environmental stresses in the overlaid slab, and thereby, improves the performance of the CRCP.

[61] TxDOT Pavement Design Guide, 2011.

[62] Image Source: WSP | Parsons Brinckerhoff.

[63] The modulus of subgrade reaction, used to estimate the support strength of the soils beneath the concrete slab, is the soil subgrade property used in rigid pavement design. It is closely related to the soil's resilient or elastic modulus (i.e., its stiffness).

[64] The modulus of elasticity is a measure of the concrete slab's resistance to deformation (i.e., stiffness).

[65] The modulus of rupture is an indicator of the flexural bending strength of concrete.

- Load transfer coefficient[66] = 2.6 (design includes tied shoulders[67])
- Drainage coefficient[68]= 1.03 (indicating fair to good quality of drainage)
- Longitudinal steel reinforcement = Number six size at eight inch spacing or 0.72 percent

Rehabilitation Design

The nine-inch CRCP structure was designed in accordance with the TxDOT standard practice for 30 years of traffic. To ensure that the CRCP design alternative is structurally adequate for 50-year cumulative traffic, an overlay design was performed using the AASHTO 1993 method. A timely placement of a thin AC overlay, even before the end of CRCP design life, may slow the progression of smoothness deterioration and reduce the potential for punchouts. Other rehabilitation alternatives that could be considered but were not studied here include an unbonded concrete overlay[69] or a bonded concrete overlay[70] on the existing CRCP.

To determine the thickness of the AC overlay, the rehabilitation design took various factors into account: (1) the condition of the CRCP structure expected at the time of overlay (anticipated to occur in 2049), (2) how much traffic the existing CRCP would have carried through 2049, and (3) how much traffic the CRCP overlaid with a new AC layer is expected to carry through 2069. To establish the condition of CRCP at the time of overlay, it was assumed that (1) there will be no durability cracking (D-cracking)[71] or spalling[72] issues, (2) the number of punchouts per mile will not exceed 12, and (3) the number of deteriorated cracks will not exceed 60 per mile or five percent, during the course of the CRCP's 30 year lifetime.

It is expected that TxDOT's routine, non-structural maintenance and rehabilitation activities will be undertaken during the life-cycle of the proposed pavement. Also, corrective actions, such as

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[66] Load transfer coefficients are standard factors recommended by the AASHTO 1993 method to reflect the quality of load transfer between two adjacent concrete slabs or the concrete slab and the adjacent roadway shoulder. Inadequate load transfer can increase the propensity for punchouts. Load transfers coefficients typically range between 2.6 and 3.8. The lower the load transfer coefficient, the better the load transfer.

[67] For the CRCP design, tie bars are proposed to facilitate the load transfer between the concrete slab and the shoulders.

[68] Drainage coefficients are standard factors recommended by the AASHTO 1993 method to reflect the quality of drainage of the subbase layers under the concrete slab. Coefficient values typically range from 0.8 to 1.2 with higher values indicating better drainage conditions. TxDOT recommends an appropriate drainage coefficient based on the annual precipitation levels at the project location and anticipated exposure of the pavement structure. The design presented here is consistent with those recommendations.

[69] An unbonded concrete overlay is a pavement rehabilitation technique that applies a concrete overlay on the existing pavement with an interlayer separator between them. Usually a thin AC layer or geosynthetic fabric is used to eliminate the bond between the overlay and existing concrete layers.

[70] A bonded concrete overlay is a pavement rehabilitation technique that applies a concrete overlay directly on the existing pavement.

[71] Durability cracking is a pattern of crescent shaped cracking near concrete slab joints, corners, or cracks that is caused by freeze-thaw expansion of the aggregate (sand or stones) in the concrete.

[72] Spalling is the breaking, cracking, or chipping of the edges of concrete slab joints or cracks.

full-depth repairs, are assumed to be undertaken before the placement of any rehabilitation overlays. Before a final comparison is made, these costs would typically be included in the life-cycle cost analysis using TxDOT's standard procedures. The proposed life-cycle activities for the CRCP design are presented in Table 3. The net present value of the total lifecycle capital costs is approximately $58.0 million for the CRCP design alternative.

Table 3: Summary of Life-cycle Activities and Capital Costs for the Base Case Rigid Pavement Design Alternative.

Year	Activity	Capital and Maintenance Costs[73] in Current Year Dollars	Capital Cost Estimates in 2020 Dollars (3% Discount Rate
2020	**New Construction** • 9 inch CRCP with 0.72% steel • 1 inch AC interlayer with PG 64-YY binder • 6 inch cement stabilized base • 18 inch lime stabilized subgrade on natural subgrade	$51,605,000	$51, 605,000
2050	**AC Overlay** • 2 inch AC surface with PG 70-YY • Full-Depth Repair of Punchouts	$13,787,000	$5,680,000
2065	Thin AC Overlay	$2,862,000	$757,000
2070	End of Analysis Period		
		Net present value of total life-cycle costs	$58,042,000

Step 3. Identify Climate Stressors

Most pavement materials are sensitive to changes in climate factors, particularly temperature and precipitation. For example, the stiffness of bituminous mixture is fundamentally dependent on temperature, while the soil subgrade stiffness is defined by the moisture content of the soil.

Climate Stressors

- Temperature
- Precipitation
- Relative Humidity

Furthermore, changes in temperature and precipitation also induce changes in other environmental variables such as relative humidity,[74] freeze-thaw cycles, ground water table levels, etc. These factors together affect the fundamental properties of pavement materials and the subgrade, control their structural responses under or in the absence of traffic loading, and contribute to distress accumulation of pavements.

[73] Maintenance costs for CRCP were calculated using the following unit cost assumptions: full-depth repair at $245.91/SY and thin overlay at $6.47/SY. For full-depth repair, it is assumed that a maximum of 10 punch outs per lane mile will be patched.

[74] Relative humidity controls the shrinkage of PCC which is due to loss of water.

Table 4 and Table 5Table 5 present a list of distresses for flexible and rigid pavements, respectively, 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 using design decisions, many of them are caused by deficiencies in materials, construction workmanship, and maintenance and are not covered in this case study. Instead, this analysis focuses on those climate-related distresses that are the most influenced by pavement design. For those interested in learning more about pavement distresses, FHWA's Pavement Distress Identification Manual[75] serves as an excellent reference including photographs.

Table 4: 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[76]	✓	✓
Settlement/Grade Depression	✓	✓
Swell/Upheaval*	✓	✓
Raveling/Weathering[77]	✓	✓
Pot Hole	✓	✓
Stripping[78]	✓	✓
Block Cracking[79]	✓	
Edge Cracking		✓
Pumping[80]	✓	
Slippage Cracks	✓	✓
Corrugation[81] and Shoving[82]	✓	✓
Bleeding[83]	✓	
Delamination[84]	✓	✓

*Distresses covered in this case study

[75] Miller and Bellinger, 2003.
[76] Reflective cracks are cracks that reflect directly over the underlying cracks or joints.
[77] Raveling is wearing away of the pavement surface caused by the dislodging of aggregate particles and loss of asphalt binder.
[78] Stripping is the loss of bond between aggregates and asphalt binder typically caused by the interaction of moisture with some minerals in aggregates.
[79] Block cracking is a pattern of cracks that divides the pavement into approximately rectangular pieces.
[80] Pumping is erosion of fine materials from support layers, accompanied with water seepage, through cracks or joints.
[81] Corrugation is the ripples of distortion on pavement surface.
[82] Shoving is a localized distortion on pavement surface that typically occurs at traffic stops.
[83] Bleeding is the excess asphalt binder occurring on the pavement surface.
[84] Delamination is loss of bonding between two pavement layers.

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

Rigid Pavement Distress	Temperature	Precipitation (Moisture)
Alkali-Aggregate Reaction[85]	✓	✓
Blow-up/ Shattered Slab[86]	✓	✓
Corner breaks	✓	✓
Depression	✓	✓
D-Cracking[87]	✓	✓
Transverse Joint Faulting	✓	✓
Joint Seal Failure	✓	✓
Lane/Shoulder Dropoff	✓	✓
Longitudinal Slab Cracking	✓	✓
Spalling[88]	✓	✓
Punchouts*	✓	✓
Pumping[89]	✓	✓
Map Cracking[90]		✓
Swell	✓	✓
Transverse Cracking (Slab Fatigue)	✓	✓

*Distresses covered in this case study

As discussed earlier, changes in temperature and precipitation affect soil moisture and, consequently, TMI, which results in swelling and shrinkage of expansive soils. The damaging effects of expansive soils are generally dormant as long as the soil moisture remains relatively constant (i.e., in an equilibrium condition).[91] Often induced naturally by changes in temperature and precipitation, any disruption to soil moisture (i.e., wetting and drying of soil) will cause an increase or decrease of soil moisture, and thus, result in volume change of soil particles (i.e., swelling and shrinkage). When there is an increase in soil moisture due to wetting, the active clay minerals of expansive soils absorb and store water molecules within their clay platelets resulting in volume increase (swelling). Alternately, with a reduction in soil moisture due to drying, the active clay minerals lose the stored water molecules thus resulting in volume decrease

[85] Alkali-aggregate reaction is the expansive reaction that takes place between alkali present in the cement paste and some minerals within an aggregate.
[86] Localized upward movement of the pavement surface at transverse joints or cracks, often accompanied by shattering of the concrete in that area.
[87] D-cracking is a closely spaced, crescent-shaped hairline cracking pattern that occurs adjacent to joints, cracks, or free edges.
[88] Spalling is cracking, breaking, chipping, or fraying of slab edges.
[89] Pumping is erosion of fine materials from support layers, accompanied with water seepage, through cracks or joints.
[90] Map cracking is a series of interconnected cracks that appear on the concrete surface.
[91] Note that the moisture content in the subgrade soil, as measured in absolute terms, defines its load bearing support (resilient modulus) while the relative change in moisture content influences the shrink-swell behavior of fine grained clay soils.

(shrinkage). Under alternate cycles of wetting and drying, the cyclic phenomena of swelling and shrinkage of expansive soils manifest as differential soil movements such as settlement, deformation, and cracking.

Given the concerns outlined above, this study focused on the impact of the following specific climate stressors:[92]

- Temperature
    - Annual mean
    - Annual maximum
    - Annual minimum
    - Degree days greater than 50° Fahrenheit
- Precipitation
    - Annual mean
    - Seasonal averages
- Mean annual relative humidity

In addition, TMI, a secondary climate variable derived from precipitation and temperature values, was generated. Any change in these climate stressors could have positive and/or negative impacts on the structural and functional performance of pavements and will need to be analyzed in Step 5.

Step 4. Develop Climate Scenarios

The Dallas-Fort Worth area experiences a hot and humid climate with an annual average mean temperature of 65.8° Fahrenheit, average annual total precipitation of 36.1 inches, and average relative humidity of 66%.[93] The spring and fall seasons are typically mild and receive significant levels of precipitation while the summers are hot and relatively dry. Given the tendency for hot and dry periods followed by mild and wet periods, the climate is conducive to creating extremes in wetting and drying of soils.

The remainder of this section presents a description of the greenhouse gas scenarios used in this case study followed by discussion of the climate projections developed for each of those scenarios.

Greenhouse Gas Scenarios

The climate scenarios used in this study include projections based on three plausible trajectories of future greenhouse gas emissions (GHGs), referred to as representative concentration

[92] Wind speed is another key climate variable that will influence (1) drying of soils and (2) moisture/temperature gradients in concrete. The effects of wind speed are not considered in this analysis as robust projections of wind speed with climate change could not be obtained.

[93] NOAA, 2016.

pathways (RCPs). The RCPs describe how global society may evolve in its use of fossil fuels, technology, population growth, etc. and the resulting emission trends and the total GHG concentration levels in the atmosphere. The three scenarios used here include:

Climate Data Overview

Level of Detail: Developed detailed projections of tailored temperature and precipitation climate indicators

Data Source: 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

Analysis: Simple Excel calculations, U.S. Geological Survey Modified Thornthwaite Monthly Water Balance Model

Scenarios: Representative concentration pathways 4.5, 6.0, and 8.5

- **RCP 4.5**, with a radiative forcing[94] 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
- **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 9 presents the 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.

[94] Radiative forcing is the change in the net (downward minus upward) radiative flux (expressed in watts per square meter) at the tropopause (top of the atmosphere) due to a change in an external driver of climate change such as, for example, a change in the concentration of carbon dioxide or the output of the sun (IPCC 2014 WGIII).

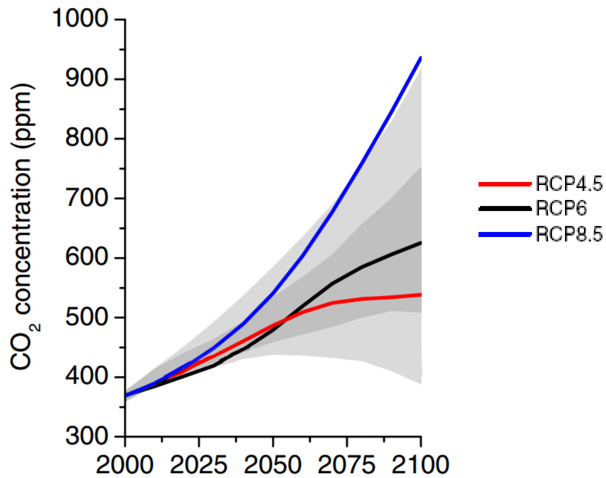

Figure 9: Radiative Forcing Trajectories of Different Representative Concentration Pathways.[95]

It is important to note that (1) 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) and (2) 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).

Climate Projections

The following weather sites at the Global Historical Climatology Network (GHCN) were used to obtain observed temperature and precipitation data for the project area in order to develop a baseline for the future projections:

- USW00003927, TX Dallas Ft Worth AP
- USC00413691, Grapevine Dam
- USC00412404, TX Denton 2 SE

For the future climate projections, the publically available statistically downscaled[96] data provided by the U.S. Bureau of Reclamation (USBR) was used.[97]. The USBR's website provides downscaled data from the World Climate Research Programme's (WCRP) Coupled Model

[95] The RCPs are available at: 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 optimistic/not conservative enough to include in this analysis.
[96] Statistical downscaling uses local observations to calculate finer spatial resolution climate data from coarser resolution global climate models.
[97] USBR, 2013.

Intercomparison Project 5 (CMIP5) that was used to inform the Intergovernmental Panel on Climate Change (IPCC) Assessment reports. These simulations, originally available at a spatial resolution around one degree,[98] have been statistically downscaled to 1/8 degree resolution for the United States by USBR. Daily values for minimum temperature, maximum temperature, and precipitation were downloaded for 11 global climate models, four USBR grid cells,[99] and the three RCPs from 1950 to 2099 (see Table 6). Specifically, all 11 global climate models were downloaded for each RCP.

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

This analysis required minimal effort to import the gridded data into Microsoft Excel and develop simple algorithms to calculate monthly average temperatures and total monthly precipitation. Because of this, the analysis did not use the U.S. Department of Transportation's (USDOT) CMIP Climate Data Processing tool. Though the CMIP Climate Data Processing tool provides monthly precipitation totals, it does not provide monthly temperature averages.

The remainder of this section discusses the projections of climate variables relevant to assessing pavement performance that were listed in Step 3: annual mean, maximum, and minimum temperatures; degree days above 50° Fahrenheit; mean annual and seasonal precipitation; TMI; and mean annual relative humidity

[98] The one degree value is approximate because each climate model's spatial resolution varies and may be smaller or larger than one degree.

[99] Four grid cells were chosen based on the center of the study location with latitude of 32.974108° North and longitude of 97.276382° West.

Annual Mean, Maximum, and Minimum Temperatures

For each climate model and RCP scenario, the following approach was coded in Microsoft Excel to generate the projections of annual mean, maximum, and minimum temperature for the project area from the USBR's statistically downscaled dataset:

- For calculating the average annual temperature, the daily average temperature was calculated from the minimum daily temperature and maximum daily temperature for every day from 1950 to 2099.
- The daily average temperature was then calculated across four grid cells for every day from 1950 to 2099.
- The monthly average temperature was calculated for each year as were the average maximums and minimums for each month
- The ensemble averages (i.e., average across all climate models for each scenario) were then calculated for each of the three temperature variables

Table 7 presents a twenty-year summary of ensemble averages of annual mean, maximum, and minimum temperatures. Figure 10 presents the 20-year moving averages of annual mean temperature for the historical observations (i.e., observed temperatures between 1950 and 2005) and future projections for different RCPs. Note that, for completeness, the projections in this table and many subsequent tables and figures are shown out beyond the period of pavement analysis (2070).

Consistent with the largest increase in future emissions of greenhouse gases, the RCP 8.5 scenario indicates the most significant increase in mean temperature, approximately nine to 10° Fahrenheit over the next 85 years. On the other hand, both the RCP 4.5 and RCP 6.0 scenarios indicate a moderate increase in mean temperature, approximately four to six degrees Fahrenheit, over the course of the 21st century. Similar observations are found for maximum and minimum temperatures as well.

Table 7: Twenty-Year Summary of Annual Mean, Maximum, and Minimum Temperature under Different RCP Scenarios.

Year	RCP 4.5			RCP 6.0			RCP 8.5		
	Tmean °F	Tmax °F	Tmin °F	Tmean °F	Tmax °F	Tmin °F	Tmean °F	Tmax °F	Tmin °F
1950-69	64.6	98.1	25.6	64.6	98.1	25.6	64.6	98.1	25.6
1960-79	64.6	98.2	25.6	64.6	98.2	25.6	64.6	98.2	25.6
1970-89	64.8	98.3	25.9	64.8	98.3	25.9	64.8	98.3	25.9
1980-99	65.1	98.6	26.3	65.1	98.6	26.3	65.1	98.6	26.3
1990-09	65.8	99.7	26.7	65.7	99.6	26.9	65.7	99.6	26.5
2000-19	66.4	100.2	27.0	66.4	100.5	27.5	66.4	100.4	27.0
2010-29	67.0	100.7	27.6	66.9	101.1	27.3	67.2	101.0	28.1
2020-39	67.8	101.8	28.1	67.3	101.5	27.6	68.0	101.9	28.6
2030-49	68.5	102.6	28.8	67.9	102.2	28.2	68.9	102.8	29.0
2040-59	69.1	103.2	29.4	68.6	103.2	28.7	70.0	103.9	30.0
2050-69	69.5	103.6	29.9	69.0	103.5	29.1	71.0	105.1	31.1
2060-79	69.7	103.9	30.1	69.7	104.1	29.2	72.1	106.4	32.1
2070-89	69.9	104.2	30.1	70.4	104.9	30.4	73.4	107.6	33.1
2080-99	70.0	104.0	29.9	70.9	105.4	31.0	74.4	108.9	34.0

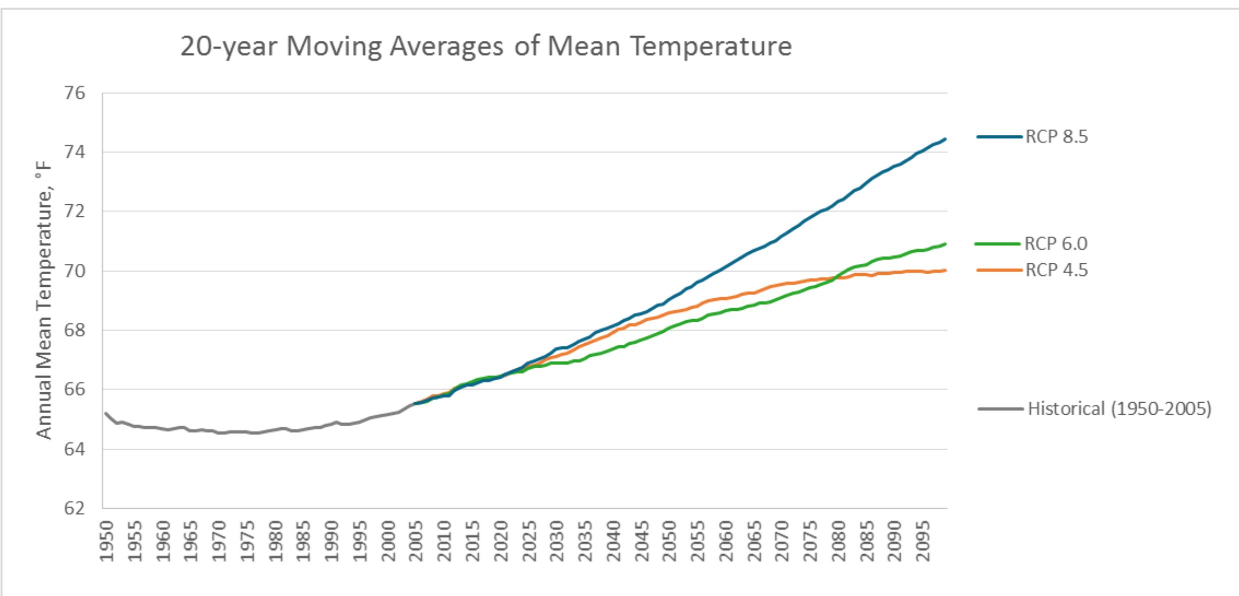

Figure 10: Annual Mean Temperature Projections under Different RCP Scenarios.

Degree Days above 50° Fahrenheit

To facilitate the analysis of high temperature requirements of asphalt binders, degree-days (DD) of daily mean temperatures greater than 50° Fahrenheit were calculated for different scenarios.[100] The projections, presented in Figure 11, indicate that the annual degree days increase steadily under different scenarios paralleling the trends of the mean temperature projections. As will be discussed further in Step 5, the increasing trends in annual degree days indicate the need for an increase in the high temperature performance grade of asphalt binders to minimize the rutting susceptibility of pavements surfaced with bituminous mixture.

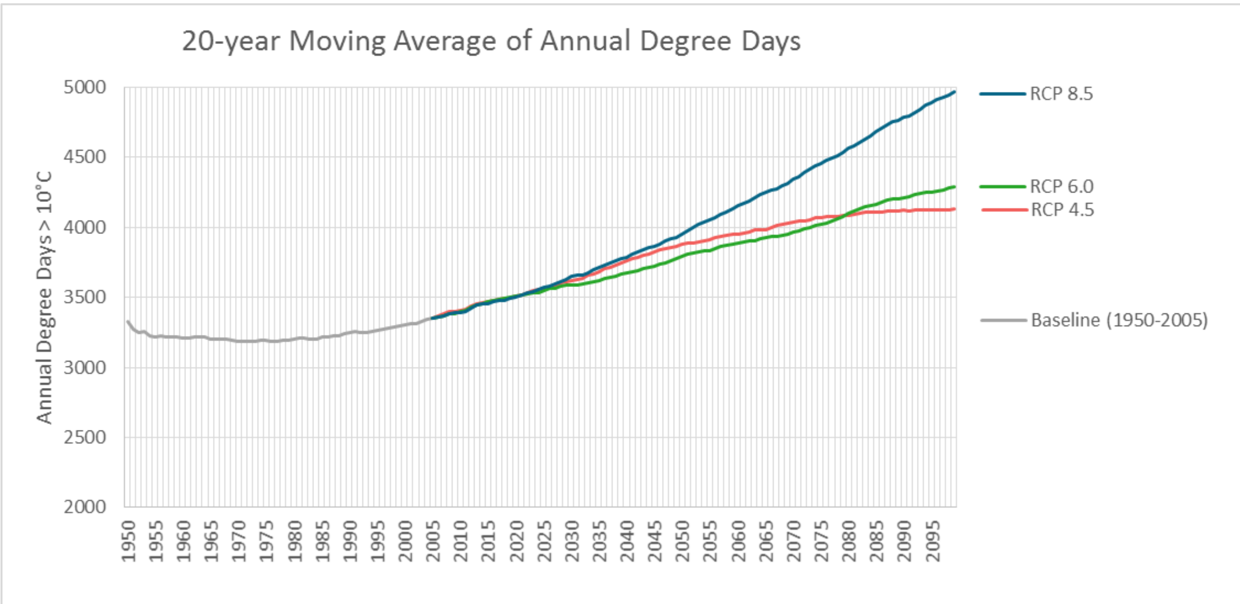

Figure 11: Projections of Annual Degree Days Over 50° Fahrenheit under Different RCP Scenarios.

Mean Annual and Seasonal Precipitation

Mean precipitation was calculated in a manner similar to that described above for mean temperatures. Table 8 presents the percent reduction in ensemble averages of seasonal total precipitation trends for three different time periods[101] under different RCP scenarios in comparison with historical averages of precipitation. RCP8.5 generally represents the driest projections for the study area. No scenario suggests wetter conditions overall. Also of note, historically, both the winter and summer seasons entail major dry spells in the Dallas-Fort Worth area; with projected climate change, these seasons will experience the highest reduction in total precipitation meaning these dry spells may become more pronounced.

[100] Mohseni, 2005.

[101] Twenty year periods were used: 2030-2049 (shown as 2040 in the table), 2050-2069 (shown as 2060 in the table), and 2070-2089 (shown as 2080 in the table).

Table 8: Percent Reduction in Seasonal Precipitation under Different RCP Scenarios Relative to 1990-2009 Observations.

Season[102]	RCP 4.5			RCP 6.0			RCP 8.5		
	2040 (2030-2049)	2060 (2050-2069)	2080 (2070-2089)	2040 (2030-2049)	2060 (2050-2069)	2080 (2070-2089)	2040 (2030-2049)	2060 (2050-2069)	2080 (2070-2089)
Winter	4%	9%	4%	1%	4%	6%	2%	5%	12%
Spring	1%	-1%	1%	0%	2%	4%	1%	5%	7%
Summer	9%	5%	5%	7%	7%	10%	4%	7%	14%
Fall	6%	2%	1%	1%	9%	4%	0%	6%	7%

Figure 12 presents a 20-year moving average of the annual average precipitation. As one can see, there is generally a slight downward trend and, as with the seasonal precipitation values, RCP 8.5 is the scenario with the most drying.

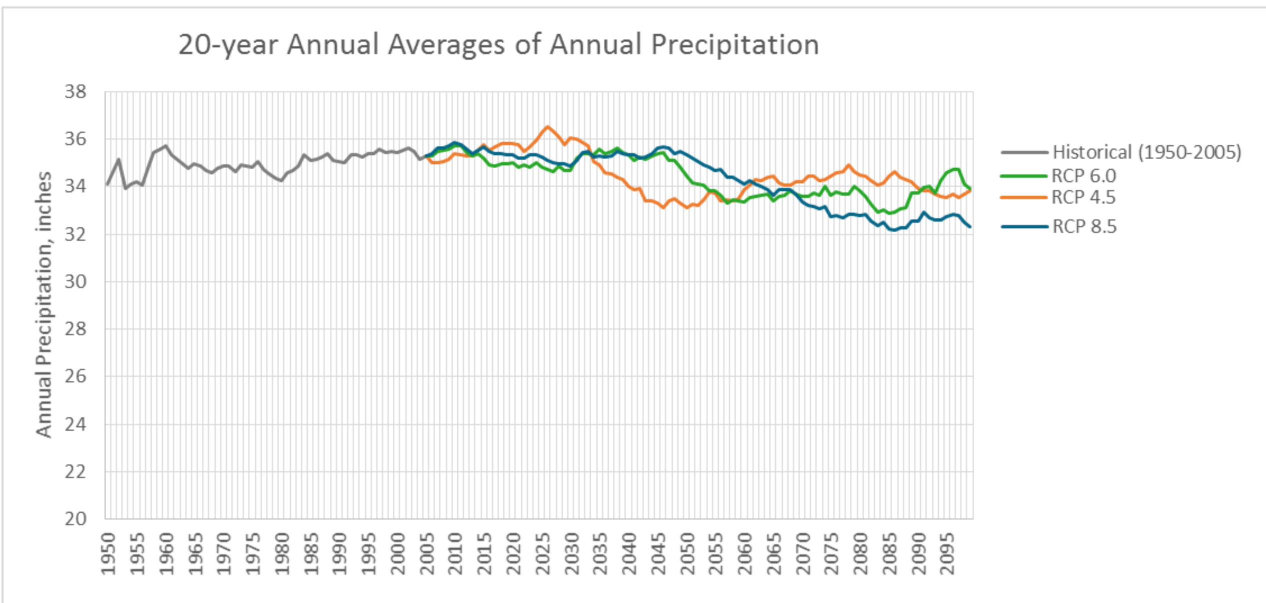

Figure 12: Average Annual Precipitation Projections under Different RCP Scenarios.

[102] Winter months include December, January and February; summer includes June, July and August; spring includes March, April and May; and fall includes September, October and November.

Thornthwaite Moisture Index (TMI)

TMI is a dimensionless measure that indicates the humidity or aridity in a geographic region. TMI is modeled as a function of temperature, precipitation, and the resulting potential evapotranspiration[103] 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 potential evapotranspiration) while negative values of TMI indicate an arid climate with water deficit (i.e., the amount of water lost through potential 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.[104] In pavement design, TMI is used as the key parameter to estimate moisture availability in soil.

TMI is used in the estimation of soil suction values, which are then used in the estimation of soil moisture at equilibrium, wet, and dry states. Note that the analytical models adopted in the Mechanistic Empirical Pavement Design Guide (MEPDG)[105] use a 12-month rolling summation of precipitation in TMI calculations in lieu of monthly precipitation-based TMI for pavement analysis.[106] The use of annual precipitation values in the TMI calculation are deemed adequate to account for seasonal variations in lieu of seasonal resolution (e.g., monthly values) of precipitation.[107]

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[103] Evapotranspiration is defined as the total water lost to the atmosphere from the ground surface (from the capillary fringe of the groundwater table) through evaporation and the transpiration of groundwater by plants whose roots tap the capillary fringe of the groundwater table.

[104] Thornthwaite, 1948.

[105] The MEPDG is the current state-of-the-art methodology in pavement design. The research products of the MEPDG were developed under the Transportation Research Board's NCHRP Project 1-37A and 1-40D. The methodology was later adopted by AASHTO under the Mechanistic-Empirical Pavement Design Guide: A Manual of Practice, 2nd Edition and the AASHTOWare Pavement ME Design software.

[106] Witczak, et al, 2006 and TRB, 2004.

[107] The 12-month rolling summation of precipitation in TMI calculations is adequate to establish the subgrade support conditions and shrink-swell potential. Note that the annual TMI value is correlated with the equilibrium moisture content of the soil that represents the long-term moisture state of the soil underneath the pavement. In simple terms, depending on factors such as drainage, soil properties, permeability and the depth of ground water table, the wetting and drying events induce changes in soil suction, which then eventually results in redistribution of soil moisture to attain an equilibrium state. Similarly, the shrink-swell potential of soil is determined by simulating the effects of change in moisture between the equilibrium moisture state and driest state (i.e., the wilting point of plants) and between the equilibrium moisture state and the wettest state (i.e., saturation capacity of soil). The mechanistic analysis of the shrink-swell model captures only the magnitude of change in moisture due to drying and wetting while the empirical portion of the model implicitly assumes the recursive effects of the frequency of alternative drying and wetting cycles in the smoothness loss calculations. While not the case in this study area, the frequency of wetting and drying cycles may change with climate change in some regions and this is an area where further research may be needed to incorporate this possibility into analyses.

TMI is a function of precipitation and potential evapotranspiration. The TMI formula developed for NCHRP Project 1-40D by ASU[108] was adopted for this case study:

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

Where:

- P is the annual total precipitation
- PE is potential evapotranspiration; PE is a function of mean monthly temperature and the heat index

Figure 13 presents the summary of temporal change in annual TMI values calculated using ensemble averages of monthly temperature and precipitation data. The climate projection data show a steady decrease in TMI indicating an increase in aridity over the next decades, albeit at different rates for the different RCP scenarios.

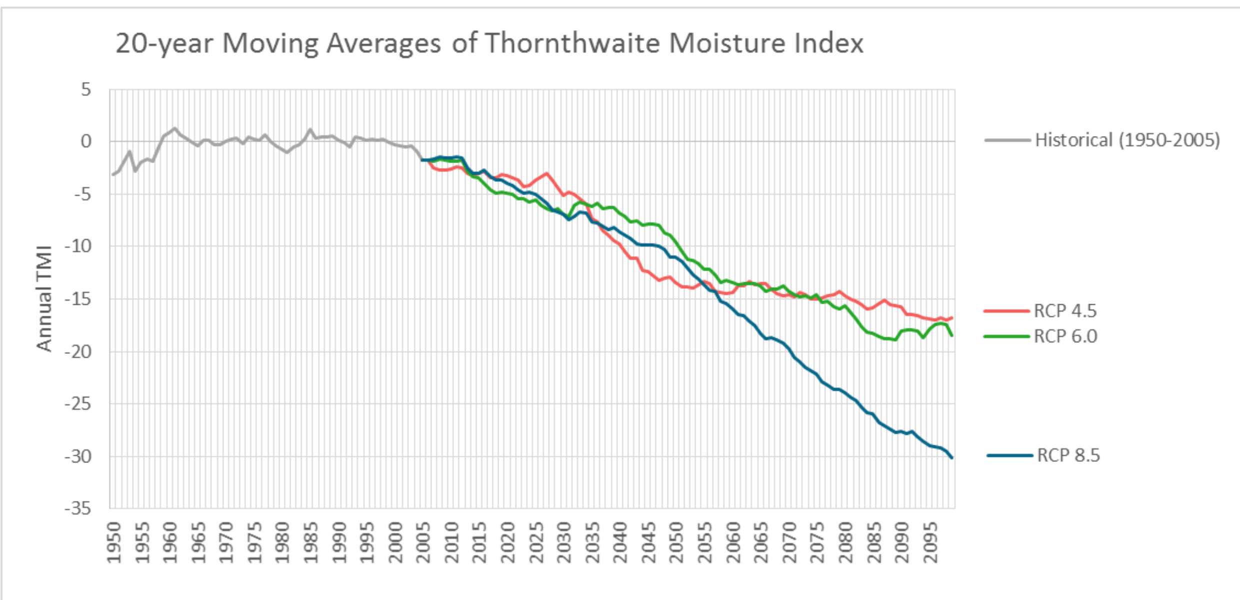

Figure 13: Annual TMI Projections under Different RCP Scenarios.

The average TMI values for the baseline period (1950-2005) range between approximately zero and negative three for the Dallas-Fort Worth area. Meanwhile, the RCP 8.5 scenario indicates a significant decrease in TMI to a value as low as -35 in the 2090s while the RCP 4.5 and RCP 6.0 scenarios indicate a more moderate, but still significant, decrease to a TMI value of -20 during the same timeframe. The TMI trends are consistent with the daily temperature and precipitation projections. Given that a slight decrease in future annual precipitation is expected along with

[108] Note that the definition used in the TMI-ASU model is slightly different from the original TMI formula developed by Thornthwaite in 1948 (Thornthwaite, 1948).

significantly rising temperatures, the aridity is expected to increase since the potential evapotranspiration component of TMI is heavily dependent on temperature-based heat indices.

Mean Annual Relative Humidity

Unlike the temperature and precipitation projections, the relative humidity projections were not direct outputs of the publically available statistically downscaled climate models from USBR. Instead, future projections of relative humidity were derived using an empirical model reported in the literature.

The empirical model is dependent on the interrelationships among ambient temperature (t), dew point temperature (t_d),[109] and relative humidity (RH) and is formulized as follows:[110]

$$t_d = t - \left(\frac{100 - RH}{5}\right)\left(\frac{t + 237.15}{300}\right)^2 - 0.00135(RH - 84)^2 + 0.35$$

The relationship between ambient and dew point temperatures was empirically derived, as shown below, using the above formulation and the historical weather records collected at the Dallas-Fort Worth International Airport:

$$t_d = 1.02*t - 0.6$$

$$R^2 = 0.964$$

In conjunction with this relationship, the future daily mean temperature projections were used to estimate the daily mean relative humidity values.

Figure 14 presents the future projections of annual ambient mean relative humidity for different climate change scenarios. The figure indicates a decrease in mean relative humidity over the next 85 years, by two, three and four percent for the RCP 4.5, 6.0 and 8.5 scenarios, respectively. Note that these trends are consistent with future projected increase in mean temperature. Given that there is a slight decrease in future precipitation under various RCP scenarios, it is rational to assume that the available ambient moisture will decrease with increasing temperature.

[109] The dew point temperature is the temperature to which air must be cooled to in order to reach saturation, assuming air pressure and moisture content are constant.
[110] Lawrence, 2004.

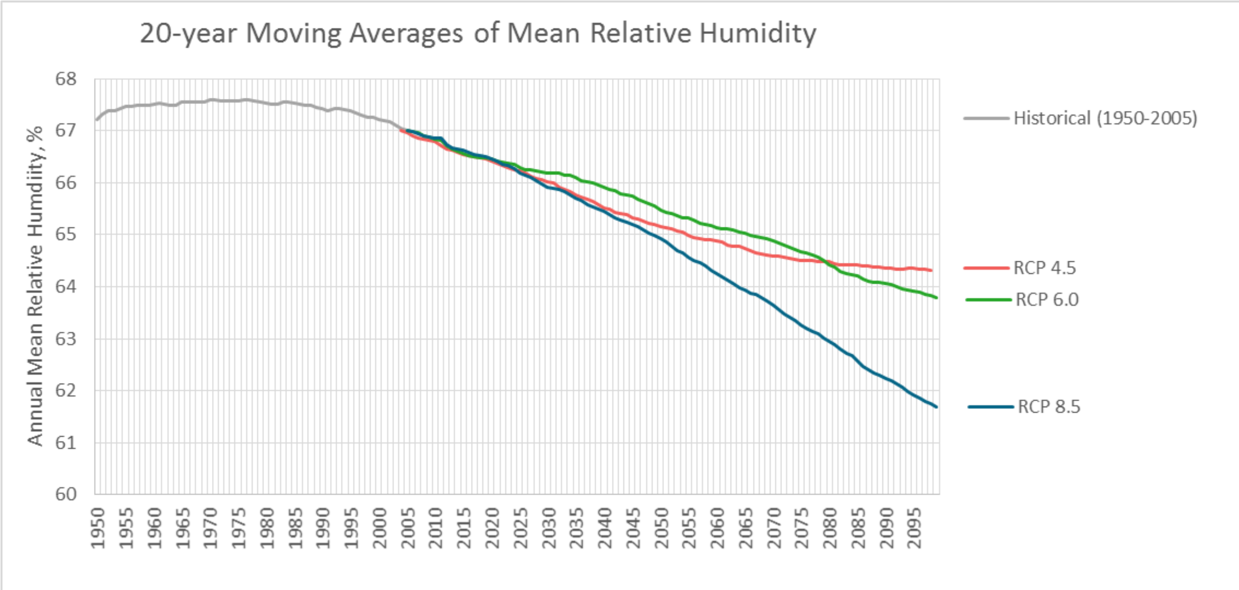

Figure 14: Annual Mean Relative Humidity Projections under Different RCP Scenarios.

Summary

To summarize, the climate data indicate more arid conditions for the Dallas-Fort Worth area with climate change, albeit at different rates. In addition to temperature effects, the increase in aridity will also have profound effects on the secondary climate variables, such as relative humidity and soil moisture, and consequently affect the performance of pavement materials and subgrade as well. Thus, the changing climate trends clearly indicate a potential for increasing damage to the pavement. Further analysis is needed to verify the net effect of climate stressors on future pavement performance as discussed in Step 5 of ADAP.

Step 5. Assess Asset Performance

In the ADAP methodology shown in Figure 3, Step 5 is broken into Step 5a (assess performance highest impact scenario) and Step 5b (assess performance under all other scenarios). Similarly, Step 6 is also broken down into Step 6a (develop adaptation options for highest impact scenario) and Step 6b (develop adaptation options for other scenarios). The reason these steps are bifurcated is that it may be possible to streamline the analyses by first looking at the highest impact scenario. For example, if it is determined that the asset would not be damaged under the highest impact scenario, then there is no need to evaluate the asset performance for lower impact scenarios.

Although the work was conducted in the order of Steps 5a, 6a, 5b, 6b, in this write up, we combine 5a and 5b into a single Step 5; we took a similar approach for Step 6. This approach was taken for improved ease of reading.

As previously discussed, increasing air temperatures can potentially cause adverse effects on pavement performance. This includes, for flexible pavements, a reduction in the stiffness of bituminous mixture, which can weaken its resistance against repeated traffic loading (i.e., reduce its long-term fatigue performance) and result in more deformation under heavier and repeated wheel loads (i.e., more enhanced rutting depressions). For rigid pavements, as noted above, the decrease in mean relative humidity due to increasing temperature can worsen the shrinkage of PCC, and thus, lead to increased punchout potential in continuously reinforced concrete pavements. However, with the slight decrease in long-term precipitation there might actually be a marginal increase in the load bearing support offered by the subgrade soil as the soil tends to dry out in the long-term.

In this step, the base case designs of both flexible and rigid pavement types were analyzed to evaluate their adequacy under the climate conditions projected for three scenarios - RCP 4.5, RCP 6.0 and 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. This section presents a series of analyses that take just such a holistic approach. The analyses are broken down as follows depending on whether they apply to both flexible and rigid pavement designs or to each design individually:

- Analyses for both the flexible and rigid pavement designs
    - Impacts on subgrade support conditions
    - Impacts on asphalt binder performance grade[111]
    - Shrink-swell potential
- Analyses for the flexible pavement design only[112]
    - Impacts on AC dynamic modulus
    - Structural distresses
        - Load-related “bottom-up” fatigue cracking
        - Subgrade rutting
        - AC rutting
- Analyses for the new rigid pavement design only
    - Punchout potential of CRCP

[111] Asphalt binder requirements apply to the AC overlay of CRCP as well.

[112] Note: Load-related fatigue cracking and rutting are typically not the key causes of concern in the AC overlays of rigid pavements due to the structural support provided by the underlying intact concrete slabs. Therefore, no further analysis is deemed necessary to evaluate the distress potential of AC overlays under climate change. The primary purpose of the AC overlays is to retard structural deterioration and formation of punchouts in CRCP, and provide a smooth and safe riding surface. Likewise, the AC interlayer in the CRCP serves as a separator “cushioning” layer to prevent the reflection of cracks from the underlying cement treated base to the overlaid continuously reinforced concrete slabs. Due to its presence between rigid pavement layers, the AC interlay typically does not undergo “major” distresses that would require immediate or expensive fixes.

The information relating to the analyses for both the flexible and rigid pavement design are discussed first, followed by analyses that pertain solely to the flexible and rigid pavement alternatives.

Analyses for both the Flexible and Rigid Pavement Designs

This section discusses the analyses of subgrade support conditions, asphalt binder requirements, and shrink-swell potential. Each of these analyses has applicability to both the flexible and rigid pavement design options.

Impacts on Subgrade Support Conditions

The impacts of future temperature and precipitation trends, as captured in TMI, on the subgrade support conditions were investigated. Note that any change in TMI will have significant effects on soil suction[113] and in-situ resilient modulus[114] of subgrade soils. With the anticipated increase in aridity (decrease in TMI) over time, the annual projected change in TMI may bring an improvement in subgrade support conditions.

The TMI-Matric Suction models, developed by ASU for NCHRP Project 1-40D and adopted in the MEPDG, were used to evaluate the impacts of TMI change on subgrade resilient modulus.[115] Supported by the evidence gathered in field testing and numerical modeling, the TMI-Matric Suction models are based on the assumption that subgrade beneath a pavement eventually stabilizes to attain an equilibrium moisture content over a few weeks or months in fine-grained soils. Note that the moisture equilibrium can later be affected by moisture infiltration through shoulders and/or potential cracks and poor drainage that may typically appear later during service.

A schematic of the TMI-Matric Suction model used to estimate subgrade moisture at equilibrium and the corresponding resilient modulus is shown in Figure 15. The model requires the following inputs for different soil sublayers: TMI, particle size distribution, soil consistency (as defined by liquid and plastic limits), specific gravity, optimum water content and maximum dry density from standard compaction tests, and coefficients of the soil water characteristics curve.

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[113] Soil suction is a measure analogous to "pulling force" that indicates the intensity with which the soil will attract the water. The drier the soil, the greater the soil suction is.
[114] In-situ resilient modulus indicates the load bearing capacity of the soil at its present state.
[115] Witczak, et al, 2006 and TRB, 2004.

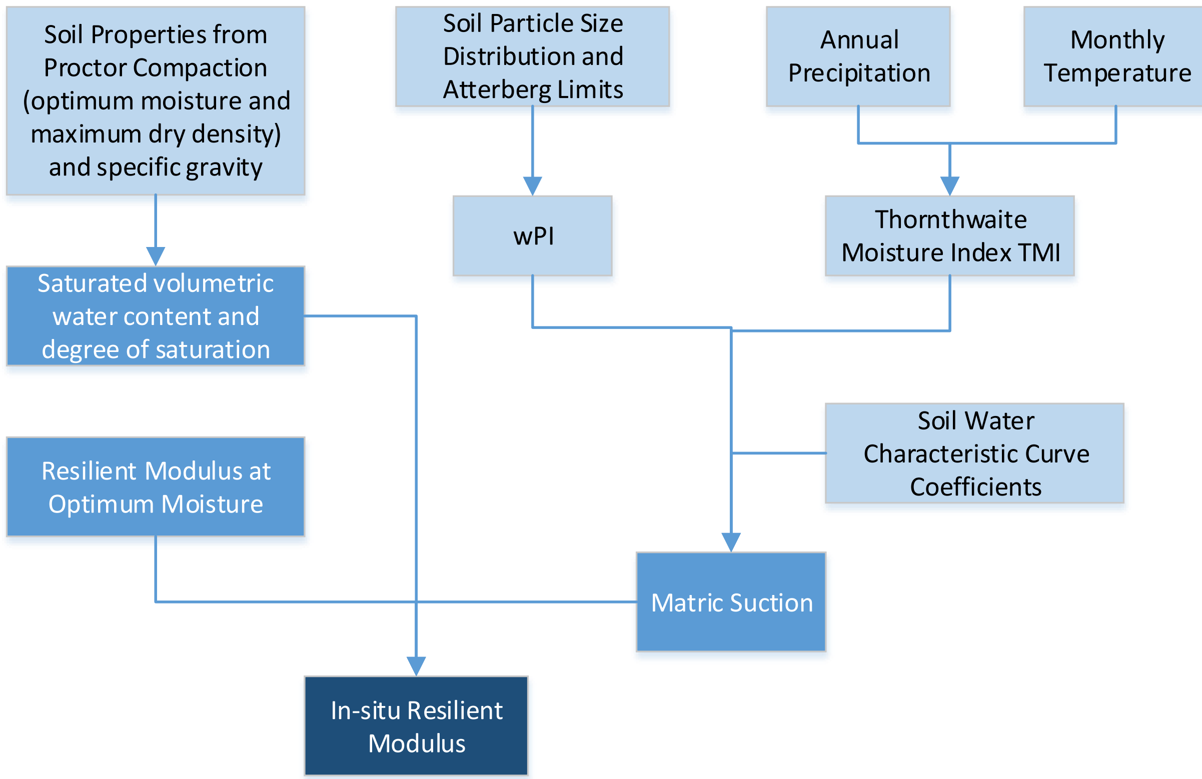

Figure 15: Schematic of the ASU TMI-Matric Suction Model.

For SH-170, the soil properties presented in Table 1 were used in the estimation of subgrade resilient modulus. Using the ASU TMI-Matric Suction model and projected TMI values (see Figure 13), the resilient modulus of subgrade soils was estimated for both the baseline climate (for reference) and the RCP 8.5 scenario.

Table 9 presents the summary of relative increase in resilient modulus with projected change in climate condition under different RCP scenarios in comparison with historical climate trends. Consistent with increasing aridity, the in-situ resilient modulus for the RCP 8.5 scenario may see a significant increase of 25 and 15 percent for subgrade sublayers one and two, respectively, while for scenarios RCP 4.5 and RCP 6.0, the in-situ resilient modulus of subgrade sublayers one and two may see a relatively moderate increase of 9 to 10 and 5 to 6 percent, respectively. However, it should be noted that the benefits of increasing aridity on subgrade support are contingent on providing adequate drainage and timely pavement maintenance to reduce moisture infiltration.

Table 9: Summary of the Relative Change in Soil Resilient Modulus under different RCP scenarios.

Year	RCP 4.5		RCP 6.0		RCP 8.5	
	Subgrade Layer 1	Subgrade Layer 2	Subgrade Layer 1	Subgrade Layer 2	Subgrade Layer 1	Subgrade Layer 2
2010-19	0%	0%	3%	2%	2%	1%
2020-29	3%	2%	2%	1%	3%	2%
2030-39	6%	3%	3%	2%	5%	2%
2040-49	7%	4%	5%	3%	6%	3%
2050-59	8%	4%	8%	5%	11%	6%
2060-69	7%	4%	6%	4%	12%	7%
2070-79	7%	4%	11%	6%	19%	11%
2080-89	9%	5%	11%	6%	20%	11%
2090-99	9%	5%	10%	6%	25%	15%

Impacts on Asphalt Binder Performance Grade

LTPPBind 3.1 software was used to evaluate the adequacy of the asphalt binder used in the base case design and assess the influence of the future temperature projections on asphalt binder requirements. The LTPP 3.1 algorithm uses a rutting damage based model to determine the high temperature component of the binder grade.[116] The process is as follows:

1. Calculate the average annual degree days (DD) for the design period using the following formula: DD = Degree-Days> 50° Fahrenheit
2. Calculate the unadjusted performance grade high temperature (PGd) to determine the initial performance grade using degree days (DD) for an assumed rut depth (RD) of 0.5 inches[117] at 50 percent reliability: $PGd = 48.2 + 14\,DD - 0.96\,DD^2 - 2\,RD$
3. Adjust the PGd for 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 percent reliability) and CVPG is the yearly coefficient of variation, in percent, calculated using the formula: $CVPG = 0.000034\,(latitude-20)^2 * (RD)^2$
4. Apply appropriate adjustments to PGrel for design traffic and pavement depth. Assuming that a two inch surface layer will be used for flexible pavements, a negative temperature adjustment of 5.2° Fahrenheit was applied for a design depth of one inch below the pavement surface.[118] For an assumed traffic range of 3 to 10 million

[116] Mohseni, 2005.
[117] 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.
[118] The design depth of one inch is the midpoint of the two inch AC surface layer.

ESALs[119] over the design period, a positive temperature adjustment of 11.7° Fahrenheit was applied in accordance with the recommendations of the LTPPBind 3.1 software.

Tables 10 through 12 present the high temperature performance grade requirements of asphalt binder for AC surface layer under different RCP scenarios. As the tables indicate, the current high performance grade of PG 70-YY will be adequate until the conclusion of the analysis period (2070) under RCP 4.0 and RCP 6.5 scenarios, beyond which the binder grade may have to be upgraded to PG 76-YY.

The RCP 8.5 scenario would require higher grades of asphalt binder to withstand the increasing trends of air temperature. The current PG 70-YY binder may not be adequate beginning around 2040 after which the binder grade may need an upgrade to PG 76-YY or higher for the AC surface layer of the flexible pavement alternative as well as the rehabilitation overlay of the CRCP. Appropriate mix design adjustments must be made with the use of higher binder grade to avoid the potential for mix placement problems and/or premature cracking.

Since lower AC layers are less susceptible to rutting damage, it is not critical to determine high temperature performance grade requirements for these layers. The base binder of PG 64-YY is deemed adequate for lower AC layers including the lower two layers of the flexible pavement alternative as well as the one inch bituminous bond breaker in the CRCP.

[119] 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 3 million cumulative 18 kip equivalent single axle load applications. As previously noted, slow traffic is not expected to be an issue for SH-170.

Table 10: Asphalt Binder High Temperature Requirements for the AC Surface Layer Under the RCP 4.5 Scenario.

Year	Average Annual Degree Days	PG Damage (PGd, °C)	PG with Reliability (PGrel, °C)	PG with Traffic and Depth Adjustments (PGadj, °C)	PG W/ Revised High Temperature
2010-29	3,604.9	61.2 (142.2°F)	63.6 (146.5°F)	67.2 (153.0°F)	PG 70-YY
2020-39	3,738.8	62.1 (143.8°F)	64.6 (148.3°F)	67.5 (153.5°F)	PG 70-YY
2030-49	3,856.1	62.9 (145.2°F)	65.4 (149.7°F)	68.3 (154.9°F)	PG 70-YY
2040-59	3,953.0	63.5 (146.3°F)	66.0 (150.8°F)	68.9 (156.0°F)	PG 70-YY
2050-69	4,024.3	64.0 (147.2°F)	66.5 (151.7°F)	69.4 (156.9°F)	PG 70-YY
2060-79	4,078.3	64.3 (147.7°F)	66.9 (152.4°F)	69.8 (157.6°F)	PG 70-YY
2070-89	4,112.5	64.5 (148.1°F)	67.1 (152.8°F)	70.0 (158.0°F)	PG 76-YY
2080-99	4,126.0	64.6 (148.3°F)	67.2 (153.0°F)	70.1 (158.2°F)	PG 76-YY

Table 11: Asphalt Binder High Temperature Requirements for AC Surface Layer Under the RCP 6.0 Scenario.

Year	Average Annual Degree Days	PG Damage (PGd, °C)	PG with Reliability (PGrel, °C)	PG with Traffic and Depth Adjustments (PGadj, °C)	PG W/ Revised High Temperature
2010-29	3,582.3	61.0 (141.8°F)	63.4 (146.1°F)	67.0 (152.6°F)	PG 70-YY
2020-39	3,662.3	61.6 (142.9°F)	64.0 (147.2°F)	66.9 (152.4°F)	PG 70-YY
2030-49	3,772.7	62.4 (144.3°F)	64.8 (148.6°F)	67.7 (153.9°F)	PG 70-YY
2040-59	3,874.5	63.0 (145.4°F)	65.5 (149.9°F)	68.4 (155.1°F)	PG 70-YY
2050-69	3,950.9	63.5 (146.3°F)	66.0 (150.8°F)	68.9 (156.0°F)	PG 70-YY
2060-79	4,068.4	64.3 (147.7°F)	66.8 (152.2°F)	69.7 (157.5°F)	PG 70-YY
2070-89	4,200.8	65.1 (149.2°F)	67.6 (153.7°F)	70.5 (158.9°F)	PG 76-YY
2080-99	4,288.7	65.6 (150.1°F)	68.2 (154.8°F)	71.1 (160.0°F)	PG 76-YY

Table 12: Asphalt Binder High Temperature Requirements for the AC Surface Layer under the RCP 8.5 Scenario.

Year	Average Annual Degree Days	PG Damage (PGd, °C)	PG with Reliability (PGrel, °C)	PG with Traffic and Depth Adjustments[120] (PGadj, °C)	PG W/ Revised High Temperature
2010-29	3,617.3	61.3 (142.3˚F)	63.7 (146.7˚F)	67.3 (153.1˚F)	PG 70-YY
2020-39	3,771.3	62.3 (144.1˚F)	64.8 (148.6˚F)	68.4 (155.1˚F)	PG 70-YY
2030-49	3,929.9	63.4 (146.1˚F)	65.9 (150.62˚F)	69.5 (157.1.8˚F)	PG 70-YY
2040-59	4,123.4	64.6 (148.3˚F)	67.2 (153.0˚F)	70.8 (159.4˚F)	PG 76-YY
2050-69	4,306.7	65.7 (150.3˚F)	68.3 (155.0˚F)	71.9 (161.4˚F)	PG 76-YY
2060-79	4,527.1	66.9 (152.4˚F)	69.5 (157.1˚F)	73.1 (163.7˚F)	PG 76-YY
2070-89	4,760.4	68.1 (154.6˚F)	70.8 (159.4˚F)	74.4 (165.9˚F)	PG 76-YY
2080-99	4,959.6	69.0 (156.2˚F)	71.7 (161.1˚F)	75.3 (167.6˚F)	PG 76-YY

Shrink-Swell Potential

The shrinking and swelling potential of soils was evaluated using the Texas Transportation Institute's (TTI) Potential Vertical Rise (PVR) method using the Windows version of the Prediction of Roughness in Expansive Soils (WINPRES) software.[121] TTI's PVR method calculates vertical movement of soil, as in shrink and swell, by taking into consideration the fluctuations in soil moisture. Since the methodology of one dimensional (1-D) vertical deformation computations is the same for both flexible and rigid pavements,[122] the observations relating to future shrink-swell potential made herein are applicable to both flexible and rigid pavements. The 1-D vertical deformation movements, estimated from soil shrink-swell potential of subgrade soils, are computational properties, which can be used to estimate vertical profile changes and further predict changes in pavement smoothness over time, when locally-calibrated pavement performance models are used.

Table 13 presents a summary of the shrink-swell analysis performed using TTI's WINPRES program. The table provides the following information for the various RCP scenarios:

[120] This includes a positive adjustment of 11.7°Fahrenheit (6.5° Celsius) for traffic and a negative adjustment of (5.2° Fahrenheit (2.9° Celsius) for pavement depth. The net adjustment is thus 6.5° Fahrenheit (3.6° Celsius).
[121] Lytton, et al, 2005.
[122] An exception is with empirical models that estimate smoothness loss from 2-D vertical movement which have different methodologies for flexible and rigid pavements.

- Equilibrium suction (in pF[123]) that is correlated with soil moisture[124]
- The depth of available moisture that refers to the quantity of water that plants can retain
- Soil moisture active zone that indicates the depth up to which the weather events can bring in changes in soil moisture
- Resulting 1-D vertical deformation.[125]

In a nutshell, the results presented in Table 13 indicate that the soil condition adjusts itself as the TMI decreases due to climate change to become more arid and consequently the soil becomes drier (i.e., soil moisture under changing climate conditions is drier than the soil moisture under historic climate conditions).[126]

Table 13: Predicted Shrink-Swell 2-D Movement in the Soil Profile under Various RCP Scenario.

Scenario	Design Period	TMI	Equilibrium Suction (pF)	Depth of Available Moisture (ft)	Depth of Moisture Active Zone (ft)		1-D Vertical Deformation (in)		
					Wettest	Driest	Swell	Shrink	Total
Baseline[127]	2020-2069	-3	3.7	3.76	6.80	18.55	2.91	1.78	4.69
RCP 4.5 & RCP 6.0	2020-2039	-7	3.75	3.64	7.00	16.47	3.05	1.63	4.68
	2040-2054	-10	3.78	3.55	7.00	14.38	3.16	1.52	4.68
	2055-2069	-12.5	3.81	3.47	7.00	14.38	3.26	1.43	4.68
RCP 8.5	2020-2039	-7	3.75	3.64	7.00	16.47	3.05	1.63	4.68
	2040-2054	-12.5	3.81	3.47	7.00	14.38	3.26	1.43	4.68
	2055-2069	-18	3.88	3.30	7.19	12.30	3.44	1.24	4.68

This drying trend of soil moisture is further illustrated in Figure 16, which presents the volumetric moisture content of the soil profile at wettest and driest conditions. In the long term, the soil will be in a relatively drier state under normal conditions due to increasing aridity. In the shorter

[123] Suction is measured in terms of pF. One unit of suction in pF is equivalent to a base 10 logarithm of 1 cm suction of water.

[124] Soil suction describes the moisture state of soil. Higher values of suction indicates the soil state is drier and pulls moisture away from soil masses of relatively wetter state.

[125] Pavement roughness (or vertical irregularities) due to shrink-swell is controlled by vertical 2-D movement (the total of shrinkage and swelling). The two dimensions represent the vertical direction, as in a bump, and the transverse direction perpendicular to the direction of the traffic. Note that the vertical movement at the edge of the pavement (i.e., wheel path farthest from the centerline of a multi-lane roadway) due to shrinking and swelling is higher than that of the interior of the pavement (i.e., wheel path nearest to the centerline of a multi-lane roadway). 1-D movement indicates shrinkage or swelling of soil in the vertical direction only.

[126] The state-of-the art models utilize the intensity of wet-dry cycles to simulate the effects on swelling and shrinkage potential of soils, respectively. However, the models do not "explicitly" consider the effects of frequency of alternative wet-dry cycles on shrink-swell potential and the rate of pavement smoothness loss; rather, the effects of frequency of wet-dry cycles are intrinsically captured in the empirical portion of the model. There is a need for future research to investigate and incorporate the effects of frequency of wet-dry cycles of smoothness loss predictions.

[127] The baseline represents an extrapolation of historic climate.

term, the soil moisture will fluctuate between the possible wettest and driest conditions due to cyclical seasonal weather events. With drying, the soil builds up suction pressure to pull water away from surrounding wetter soil masses to maintain an equilibrium state. Once the moisture redistribution is attenuated due to lack of moisture supply, the soil mass will begin to shrink steadily. The magnitude of shrinkage is governed by the frequency of the wetting events and the severity of aridity.

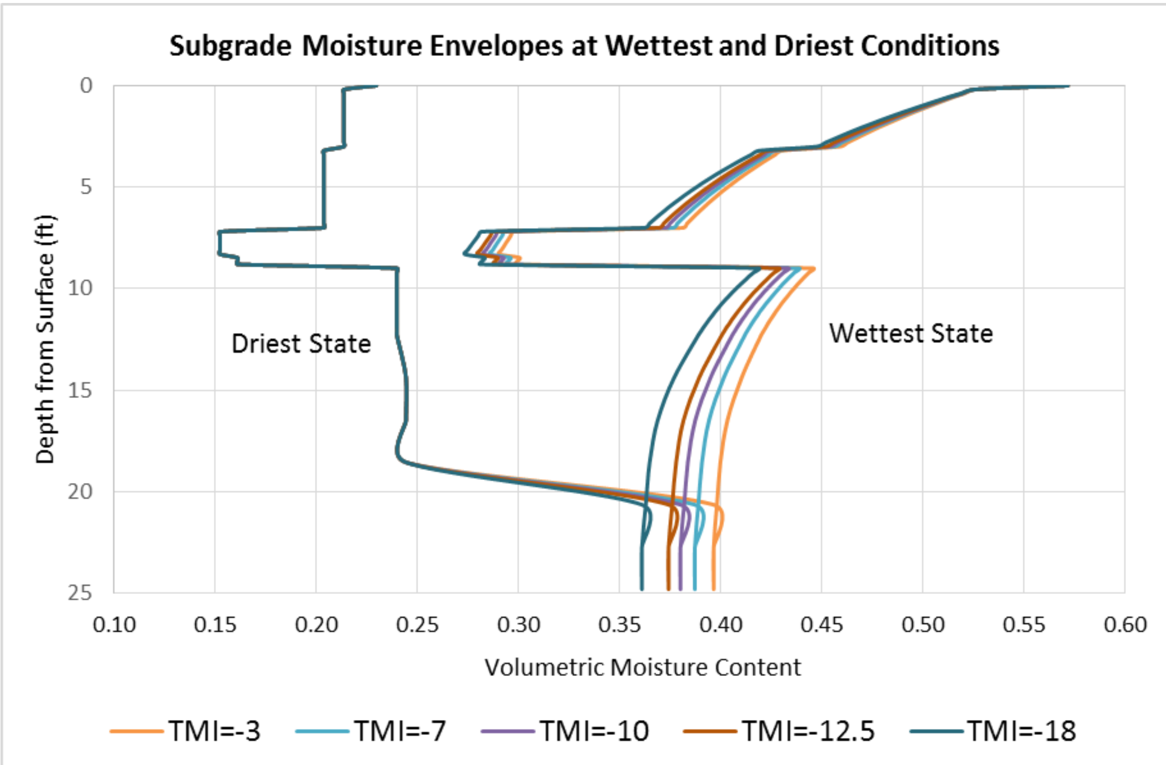

Figure 16. Volumetric Moisture Content of Soil Profile at Wettest and Driest Conditions.

When the equilibrium is disturbed by cyclical wetting events, the drier soil will have more potential for swelling than shrinkage. Note that the extreme wet and dry conditions (suction envelopes)[128] of the soil profile are bound by limits and controlled by non-climatic factors (e.g., soil properties). The shift in moisture equilibrium towards the drier side creates a wider difference between the equilibrium state and the extreme wet boundary than between the equilibrium state and the extreme dry boundary. In other words, the soil in a drier state has a higher propensity to absorb moisture, and hence undergoes more deformation, which increases its swelling potential. Similarly, the soil in a wetter state has a higher propensity to lose moisture,

[128] The shrink-swell model uses upper and lower bounds for drying and wetting. While the soil suction on the driest side is limited by the wilting point of vegetation (i.e., approximately 31623 cm of suction), the limit for soil suction on the wettest side is limited approximately at 100 cm of suction. The swelling and shrinkage movements are bound within these limits. Bound by these limits, the swelling and shrinkage movements are computed based on the vertical strains i.e., the relative change from its current state to a deformed state.

and hence undergoes more deformation, which increases its shrinkage potential. The total movement due to shrinkage and swelling will remain almost unchanged; upon wetting, the soil will have the propensity to swell quickly and confined to shallower depths; and comparatively, depending on the frequency of the wetting events, the drying shrinkage in soil will be slower and will deeper into the soil profile.

As the Table 13 depth of moisture active zone values indicate, the increasing aridity will reduce the zone of moisture activity under driest condition; and hence, there will be less moisture for evapotranspiration of tree roots. Furthermore, in comparison with the surrounding areas, the subgrade, especially underneath the center of pavement, will be in a relatively wetter condition than the edges and surrounding areas. This differential in soil moisture may attract the roots of vegetation and cause them to grow laterally from the pavement edges to reach out to the subgrade underneath the pavement.

These roots may induce or penetrate through cracks in the subgrade and/or pavement structure in irregular spatial patterns, and further create a lifting pressure on the overlying pavement structure. This lifting pressure may decrease pavement smoothness and lead to cracking-related problems. No computational models are available to quantify the impacts of vegetation growth on pavement performance.

In a nutshell, since the shrink-swell potential of expansive soils are bound by non-climatic factors, climate change will not have significant enough of an impact on the total movement of the soil mass to affect pavement smoothness. However, climate change is likely to increase the risk of lateral vegetation growth, which in turn may lead to changes in the pavement smoothness and cracking. These consequences are applicable to both flexible and rigid pavements, although the likelihood of occurrence and severity of impacts will be specific to the pavement type and structure.

Analyses for the Flexible Pavement Design Only

The following section discusses the adequacy of the as-designed base case flexible pavement structure to the changing climate conditions projected under the RCP 8.5 scenario. Specific analyses include assessment of the AC dynamic modulus and key structural distresses such as load-related fatigue cracking, subgrade rutting, and AC rutting.

Impacts on AC Dynamic Modulus

Future projections of temperature will have an impact on the dynamic modulus (|E*|) of AC layers. Among other factors, both temperature and asphalt binder grade are considered to have profound effects on bituminous layer's resistance against cracking and permanent deformation under repeated loading. It is necessary to evaluate how the stiffness of a typical TxDOT bituminous mixture will change over time under different RCP scenarios in comparison with historical temperature conditions. The AC |E*|prediction model,[129]adopted in the MEPDG, was used to estimate the AC dynamic modulus values.[130]

Figure 17 presents the temporal distribution of dynamic modulus of the AC surface layer estimated using the future projections of annual mean temperatures under different scenarios. The base case design asphalt binder grade of PG 70-YY was unchanged for all three scenarios. With a projected increase in future temperature, the trends indicate a steady decrease in AC stiffness under all three scenarios. The estimated reduction in AC stiffness is approximately by one-fourth in 2070 and one-third by 2099 under the RCP 8.5, while the reduction is slightly lower under the RCP 4.5 and RCP 6.0 scenarios at about 15 to 20 percent. Similar trends of dynamic modulus are expected for the lower AC layers.

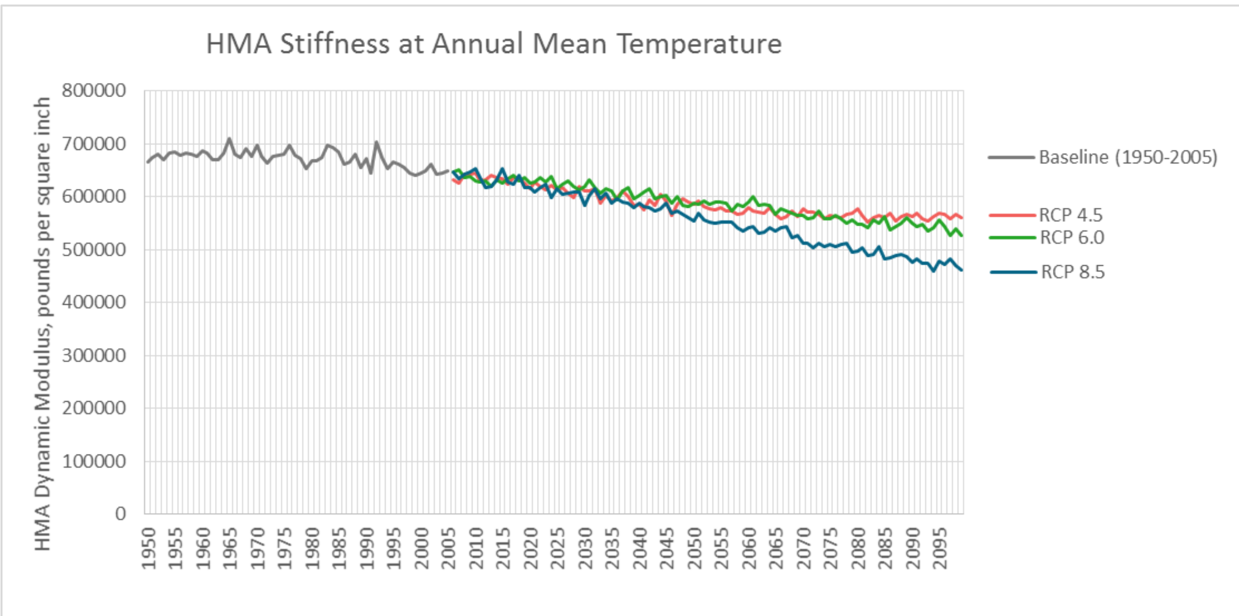

Figure 17: Stiffness of the AC Surface with Annual Mean Temperature Projections under different RCP Scenarios.

Structural Distresses

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

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

[129] The AC |E*| prediction model uses pavement temperature, performance grade of asphalt binder, and other properties in dynamic modulus estimations.
[130] TRB, Mechanistic-Empirical Pavement Design Guide, 2004.

propagates to manifest on the surface of the pavement structure, as more fatigue damage is accumulated under repeated traffic loading. The evaluation of fatigue damage is paramount because the bituminous layer becomes softer and less stiff due to systematic warming trends under climate change scenarios, which will in turn, affect the ability of the pavement to withstand repeated traffic loading.

The research team evaluated the fatigue cracking potential of the existing pavement under different RCP scenarios using the methodology proposed by the Asphalt Institute.[131] The Asphalt Institute's fatigue cracking model postulates the following fatigue equation for a standard asphalt concrete mixture 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 lane area,
- ε_t is the horizontal strain at the bottom of the HMA layer[132], and
- $|E^*|$ is the HMA dynamic modulus.

The FPS-21 design procedure employs the Asphalt Institute's fatigue criterion as a mechanistic check for TxDOT designs. Hence, the load-related fatigue cracking is used as a performance criterion for reconfiguration of design cross-section, if needed.

The cumulative index for fatigue damage (DI_{FC}) is calculated as the ratio of actual number of axle load applications within a specific time period (n) over 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[133] is computed as a summation of incremental damage indices over time for each period or loading subgroup:

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

[131] Huang, 2003.

[132] 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.

[133] A damage index of one indicates that there is a 50 percent probability that fatigue damage (manifested as 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.

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. The analysis showed that the structural capacity of the base case design structure (i.e., the N_f or the allowable number of load applications that the pavement can carry under historical climate conditions) is 8.2 million ESALs. Note that the Asphalt Institute does not include reliability considerations in damage estimations.

Table 14 presents a summary of percent change in fatigue performance of the base case design under different RCP scenarios. Note that the cumulative damage indices of the base case design were found to be within the allowable damage threshold under all three scenarios and historical climate conditions. Nonetheless, the fatigue damage is expected to increase by up to 10 percent under these scenarios relative to an extrapolation of historical conditions[134] and this is expected to become somewhat more pronounced with the RCP 8.5 scenario. The base case pavement structure would be adequate for all climate change scenarios; however, there is a greater likelihood for additional cracking due to harsher climatic conditions under future scenarios. In other words, though the fatigue-related cracking is expected not to exceed 20 percent of the lane area under all scenarios, the pavement structure could only withstand 10 percent less vehicles (in terms of ESALs) under the RCP 8.5 scenario than under historic climate conditions to reach a specific threshold of cracking. This increase in fatigue damage is due to the expectation that the rising temperature will make the bituminous layer softer and less stiff and hence will incur more damage under traffic loads. Fatigue cracking or damage is a pavement structural distress; an increase in pavement fatigue damage under climate change scenarios may lead to earlier-than-expected structural failures and/or more localized failures and thus require more frequent interventions for pavement maintenance and repair.

[134] An extrapolation of historical climate conditions assumes that the historical averages of temperature and precipitation will not change in the future, and hence, the inputs (e.g., TMI, AC dynamic modulus, soil resilient modulus, and asphalt binder grade) considered in the base case design will remain the same.

Table 14: Summary of Relative Change in Flexible Pavement Distresses under different RCP Scenarios.

Lifecycle Activity and Year	Percent Change in Pavement Distress Under Various Scenarios Relative to an Extrapolation of Historical Climate Conditions								
	Fatigue Damage			Subgrade Rutting			AC Rutting		
	RCP4.5	RCP6.0	RCP8.5	RCP4.5	RCP6.0	RCP8.5	RCP4.5	RCP6.0	RCP8.5
New construction, 2020-2039	5%	3%	5%	0%	0%	0%	12%	9%	13%
Rehabilitation 1, 2040-2054	8%	6%	9%	9%	9%	9%	21%	16%	27%
Rehabilitation 2, 2055-2069	6%	5%	9%	8%	8%	8%	20%	16%	33%

Subgrade Rutting

In a multi-layered flexible pavement structure, as temperature increases under RCP 8.5, the AC pavement layers above the subgrade will become softer and less stiff, and thus, make the lower pavement layers take up more stress under traffic. This phenomenon contributes to more consolidation of subgrade under repeated traffic loading and manifests as "wheel path depressions" on the roadway surface. This form of structural distress is called subgrade rutting.

The methodology proposed by the Asphalt Institute was used to evaluate the impacts of climate change on the subgrade rutting potential of the base case design. [135]

The methodology postulates that the permanent deformation due to subgrade consolidation can be limited when the vertical compressive strain on the top of the subgrade is controlled. The Asphalt Institute's subgrade rutting criterion is:

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

Where:

- N_d is the allowable number of load repetitions to limit subgrade related permanent deformation
- ε_c is the vertical compressive strain on the top of the subgrade (calculated by the structural response model).

The cumulative damage index for rutting is calculated as the ratio of the actual number of axle load applications within a specific time period (n) over N_f. A damage index of one indicates that there is a 50 percent probability that the total rutting will not exceed 0.5 inches on the

[135] Huang, 2003.

surface.[136]Note that the FPS-21 design procedure employs the Asphalt Institute's subgrade rutting criterion as a mechanistic check for TxDOT designs.

Table 14 presents the percent change in accumulated damage in terms of subgrade rutting. The base case pavement structure was considered adequate to withstand subgrade rutting, as the cumulative damage indices were found to be within the acceptable damage threshold of 1.0 under all scenarios. However, under the climate conditions projected in the RCP scenarios, the as-designed base case pavement structure will experience higher subgrade rutting, as high as nine percent, in comparison with historic climatic conditions. This increase in subgrade rutting under the RCP scenarios can be primarily attributed to softening of the AC layers above the subgrade. Although the subgrade resilient moduli are expected to increase under the RCP scenarios due to drying out of soils, this modest improvement in subgrade support conditions is not adequate to offset the increasing stresses from traffic loads.

Permanent surface depressions caused by subgrade rutting in the wheel paths can lead to driving hazards and hydroplaning[137] during rain events. The deformities can be corrected using milling and resurfacing.

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. Rising temperature due to climate change increases the risk of rutting within the AC layer.

The AC rutting model used by the MEPDG is used for this purpose.[138] For all AC mixtures, the accumulation of permanent deformation in the AC is calculated using the following equation:

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

[136] Total accumulated rutting on the pavement surface is mostly caused by the rutting that occurs in the bituminous layer and the subgrade. The Asphalt Institute method postulates that if the subgrade rutting is controlled, the surface rut depths will not exceed reasonable depths. Further, the rutting that occurs in the bituminous layer can be controlled effectively through the selection of an appropriate binder grade, bituminous mixture design and good quality construction.

[137] Hydroplaning is the buildup of a thin layer of water between the pavement surface and vehicle tires during rain events that results in a loss of traction and steering control.

[138] TRB, Mechanistic-Empirical Pavement Design Guide, 2004.

Where:

- $\Delta_{p(AC)}$ is the accumulated permanent deformation in the AC layer/sublayer
- $\varepsilon_{p(AC)}$ is the accumulated permanent or 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
- k_z is the depth confinement factor (see below)
- n is the number of axle load repetitions
- T is the pavement temperature in degrees Fahrenheit.

k_z 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
- 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. Since the AC rutting model was not locally calibrated for Texas conditions, such as TxDOT mix types and construction practices, the absolute AC rut depth estimations are not presented. Instead, only the percent changes in predicted rutting are reported for the purpose of comparing various climate change scenarios.

Table 14 presents the percent change in accumulated AC rutting under different RCP scenarios. With the projected climate conditions, the as-designed base case pavement structure will experience higher rutting in the AC layers, by up to approximately one-third under RCP 8.5 in comparison with historic climatic conditions, and by 20 percent under RCP 4.5 and RCP 6.0. However, given the unavailability of local calibration coefficients, the findings presented herein must be vetted through detailed field investigations and material characterization to accurately predict AC rutting. Nonetheless, the tendency for more AC rutting under these scenarios will likely remain the same after calibration.

Analyses for the Rigid Pavement Design Only

Only one analysis pertained solely to rigid pavement design: the assessment of punchout potential within the CRCP. This analysis is described in depth below.

Punchout Potential of CRCP

Punchouts are the predominant structural distress in CRCP that develop over time under environmental and traffic loads. Punchouts are localized slab failures characterized by closely spaced transverse cracks, often connected by short longitudinal crack(s) and joints. Punchouts also control the deterioration of pavement smoothness in CRCP. The following factors typically contribute to the initiation and progression of punchouts:

- Tensile stresses caused by bending of slabs under repeated traffic loading leading to fatigue damage
- Environmentally induced stresses caused by temperature and moisture gradients across the slab thickness (discussed in more detail below)
- Loss of load transfer efficiency (LTE) across the transverse cracks which is significantly controlled by lower crack width
- Loss of support due to base or subbase erosion which is affected by the erodibility potential of the base material and moisture infiltration through cracks
- Absence of optimal transverse cracking spacing. Optimal crack spacing is based on the considerations of punchouts and spalling[139] and is controlled by steel content

Climate factors have a significant influence on punchout potential of CRCP. Changes in temperature and relative humidity, among the other design factors listed above, control the crack width and the degree of load transfer across cracks. Specific climactic influences include:

- The ambient relative humidity which greatly controls the amount of drying shrinkage in concrete during construction as well as over the long term
- The moisture gradient between the top and bottom of PCC slabs which can contribute to warping stresses
- Curling[140] and warping stresses[141] due to temperature and moisture gradients, respectively, which contribute to increase in tensile bending stresses at the top of concrete slabs and are critical to top-down slab cracking in CRCP

In summary, both temperature and relative humidity together have significant effects on load transfer efficiency, fatigue damage, and punchout potential of CRCP.

As discussed earlier, the TxDOT Pavement Design Manual recommends the use of the AASHTO 1993 method for analyzing punchout potential. The CRCP design methodology, specified in the AASHTO 1993 method, is based on pavement serviceability concepts that provide design

[139] Narrower crack spacing increases the potential for punchouts while longer crack spacing increases the potential for spalling.
[140] Curling stresses are caused by the differences in temperature between the top and bottom surfaces of a PCC slab.
[141] Warping stresses are induced by the differences in moisture between the top and bottom surfaces of the PCC slab.

recommendations for slab thickness and steel content. While punchout is not a direct design criterion, the CRCP design recommendations in the AASHTO 1993 method are based on limiting crack spacing, crack width, and steel stresses within thresholds and thereby indirectly controlling the progression of punchouts. However, the AASHTO 1993 method has limitations to fully understanding the impacts of climate factors, temperature, and relative humidity in particular, on the performance of CRCP.

To understand the effects of temperature and relative humidity variations on long-term performance of CRCP, it is essential to understand how these climate factors fundamentally affect the structural responses that contribute to punchouts. Hence, the methodology used herein relied on analytical models[142] adopted by the MEPDG combined with sensitivity analyses of key climate factors and design parameters.[143] The key findings of the sensitivity analyses are presented as follows:

- **Drying Shrinkage**: The change in ambient relative humidity will affect both relative humidity in the concrete and the drying shrinkage. Within the prevailing range in the Dallas-Fort Worth area, the sensitivity analysis indicates the initial drying shrinkage will increase by approximately 2.5 percent with every percent decrease in ambient relative humidity. The drying shrinkage will also be accelerated by increasing ambient temperature albeit marginally in comparison with relative humidity.
- **Warping Stresses**: The difference in relative humidity between the top and bottom of the PCC slab may remain significantly unchanged indicating minimal, if any, expected change in warping stresses.
- **Curling Stresses**: The change in ambient temperature will increase the temperature gradients between the top and bottom of the PCC slab thus resulting in more curling stresses. Both the environmentally induced stress and associated strains may increase by approximately one percent with each degree Fahrenheit increase in ambient temperature.

For the SH 170 case study, the effects of changes in ambient temperature and relative humidity on structural responses, as discussed above, were applied on the key design parameters, including crack spacing and crack width.

Table 15 presents the summary of expected changes in key design parameters and performance of CRCP under three different RCP scenarios for the time period until 2070. As the ambient temperature and relative humidity change, the predicted crack width is expected to increase over time: the wider cracks will have negative impacts on load transfer and thus increase the potential

[142] Refer to Appendix LL of the MEPDG (TRB, 2004) for additional details on the analytical models.
[143] TxDOT Pavement Design Guide, 2011.

for punchouts. However, a smaller impact on crack spacing, in the range of a few tenths of an inch, is expected with changing climatic conditions. Due to the lack of calibrated performance models, the precise percent change in the number of punchouts could not be estimated. Nonetheless, the as-designed pavement structure is considered adequate to withstand the projected climatic conditions under all scenarios. Given the projected increase in crack widths, the performance of the base case CRCP is expected to be affected by climate change, and more pronounced under the RCP 8.5 scenario and modestly under the RCP 4.5 and RCP 6.0 scenarios; however, this impact can be effectively addressed through design adjustments in the base case design.

Table 15: Estimated Changes in CRCP Design Parameters and Performance under Different RCP Scenarios.

Percent Change in CRCP Performance Under Various Scenarios in Comparison with an Extrapolation of Historical Climate, 2020-2069			
Factor	**RCP 4.5**	**RCP 6.0**	**RCP 8.0**
Change in annual mean air temperature	+1.7°F	+1.7°F	+3.1 °F
Change in annual mean relative humidity, %	-1.7%	-1.7%	-3.7%
Change in crack width	+3%	+3%	+6%
Change in crack spacing	-1%	-1%	-1%

Summary

To summarize, there will be both beneficial and detrimental effects to the flexible and rigid pavement design options under all scenarios, but more pronounced under the RCP 8.5 scenario. The benefits will mainly be a result of decreased soil moisture and an accompanying decline in the intensity of soil shrink-swell cycles which could increase subgrade support and improve pavement smoothness (for both). The distresses are projected to increase into the future over the base case scenario, albeit at different rates for different RCP scenarios. Since the predicted distresses are well within the design thresholds under all scenarios, the as-design pavement structure did not warrant changing across various scenarios. The trends imply that the pavement subjected to the harsher climate should likely have lower performance expectations than under a more favorable climate scenario. The potential vulnerabilities of pavement performance due to climate change include:

- For the flexible pavement design option:
    - Inadequate asphalt binder performance grade for the AC surface layer
    - Decrease in AC dynamic modulus
    - Increase in load related fatigue damage or “alligator” cracking
    - Increase in subgrade rutting
    - Increase in AC rutting
- For the rigid pavement design option:

- Increase in CRCP punchout potential (or increase in crack width) and full-depth repairs
- Inadequate asphalt binder performance grade for the rehabilitation overlay

Thus, both base case design options are likely to be adversely affected by projected climate changes and adaptation actions will be needed for each in Step 6.

Step 6. Develop Adaptation Options

Potential adaptation measures were developed for both the flexible and rigid pavement design options to mitigate the adverse impacts on pavement performance. These are summarized in Table 16 and Table 17.

Table 16: Recommended Adaptation Measures for the Flexible Pavement Alternative.

Pavement Design Alternative	Life-cycle Activity and Year	Base Case Design Capital Cost ($2020, 3% Discount Rate)	Recommended Adaptations	Additional Adaptation Capital Cost ($2020, 3% Discount Rate)
Adaptation Option 1 - Flexible Pavement Design (All Scenarios)	Flexible pavement: new construction, 2020-2039	$65,528,000	None	$0
	Rehabilitation 1: mill & overlay, 2040-2054	$8,167,000	Use stiffer binder (e.g., PG 76-YY or equivalent)	$769,000
	Rehabilitation 2: Mill & overlay, 2055-2069	$5,243,000	Use stiffer binder (e.g., PG 76-YY or equivalent)	$493,000
	Net present value of total life-cycle capital costs	$78,938,000		**$1,262,000 (1.6% of base cost estimate)**

Table 17: Recommended Adaptation Measures for the Rigid Pavement Alternative.

Pavement Design Alternative	Life-cycle Activity and Year	Base Case Design Capital Cost ($2020, 3% Discount Rate)	Recommended Adaptations	Additional Adaptation Capital Cost ($2020, 3% Discount Rate)
Adaptation Option 2 - Rigid Pavement Design (RCP 4.5 and RCP 6.0)	Rigid pavement: new construction, 2020-2049	$51,605,000	Increase steel content from 0.72% to 0.74%	$2,083,061
	Rehabilitation 1: AC overlay 2040-2069 (Use PG 70-YY binder)	$6,437,000	None	0
	Net present value of total life-cycle capital costs	$58,042,000		**$2,083,061 (4.0% of base cost estimate)**
Adaptation Option 3 - Rigid Pavement Design (RCP 8.5)	Rigid pavement: new construction, 2020-2049	$51,605,000	Increase steel content from 0.72% to 0.74%	$2,083,061
	Rehabilitation 1: AC overlay 2040-2069 with PG-76	$6,437,000	Use stiffer binder (e.g., PG 76-YY or equivalent)	$3376
	Net present value of total life-cycle capital costs	$58,042,000		**$ 2,086,437 (4.0% of base cost estimate)**

The adaptation measure proposed for the flexible pavement design alternative under all climate change scenarios involves the use of stiffer asphalt binders (e.g., PG 76-YY or equivalent polymer modified binder alternatives). Although the high temperature performance grade requirements for asphalt binders did not warrant a change until 2070 (see Tables 10 and 11), the stiffer binders will compensate for the softening of AC due to increasing temperature under scenarios RCP 4.5 and RCP 6.0 and further mitigate the increase in pavement structural distresses.

For the rigid pavement design alternative, a marginal increase in steel content from 0.72 percent to 0.74 percent for the CRCP design alternative is recommended under all climate change scenarios. As with the base case, the placement of a 2-inch AC overlay is recommended for rehabilitation at year 30. While the use of PG 70-XX binder will be adequate for the AC overlay under the RCP 4.5 and 6.0 scenarios, a stiffer binder, say PG 76-XX or equivalent, will be required under the RCP 8.5 scenario. Table 17 does not include foundation strengthening options because they are considered in TxDOT Local District’s practices.

Pavement infrastructure is a complex system with interactions of multiple factors including traffic, soil foundation, pavement structural types and layer thicknesses, climate, material properties, and construction. There is 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. While the most promising specific adaptation measures were selectively proposed in Table 17 for SH-170, other alternative measures that could be considered include:

- For both the flexible and rigid pavement design options:
    - Make adjustments to construction specifications to improve quality characteristics and reduce variations such as reducing the air voids or more stringent tolerances for mix and mat acceptance
    - Improve drainage and use other remedial measures, such as the use of geotextiles or mulch in the shoulders, to reduce infiltration of moisture into the subgrade and prevent base erosion.
    - Strengthen the pavement foundation by extending the depth of soil stabilization.
    - Pre-paving and post-paving treatments of roadside vegetation using herbicides and soil sterilants.
- For the flexible pavement design option only:
    - Make adjustments to the AC mix specifications such as (1) recommending higher percentages of crushed aggregates and manufactured fines to improve the aggregate interlock, (2) adjusting the asphalt binder content (decreasing 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), and (3) addition of lime to stiffen the mix.
    - Use high quality AC, such as performance design mixtures[144] (TxDOT Standard Bid Item 344) and stone matrix asphalt[145] (TxDOT Standard Bid Item 346) to ensure better performance
    - Use stiffer asphalt binders (e.g., PG 76-YY or equivalent polymer modified binder alternatives)
    - Increase the thickness of AC layers to improve fatigue, rutting, and smoothness performance

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[144] Performance design mixtures are bituminous mixtures designed specifically to an intended application; for example, a mix for a low volume roadway can be designed to provide durability and resistance against cracking with higher binder content whereas a mix for a high volume roadway can be designed with lower binder content to provide greater resistance against rutting. These type of mixtures are typically used in medium to high volume roadways in Texas.

[145] Stone Matrix Asphalt (SMA) is a gap-graded mixture with a high percent of coarse aggregate that forms a stone skeleton, while the inter-particle voids are filled with mastic consisting of mineral filler, fiber, and asphalt binder. Typically used on surface layer, SMA mixtures are designed to provide greater resistance against rutting and durability.

    - Explore the use of stabilized or stiffer base layers to reduce rutting of bituminous layers.
- For the rigid pavement design option only:
    - Use non-erodible materials for CRCP bases
    - Use optimized concrete mix designs to control for shrinkage
    - Use good construction practices such as membrane-forming curing compounds to reduce the onset of early permanent curling and shrinkage

The specific adaptation options proposed for this analysis in Table 17, changing the binder and (for the rigid pavement design option) using a higher steel content, were deemed to be of high enough cost to warrant an economic analysis as to whether they are cost effective.

Step 7. Assess Performance of Adaptation Options

This step entails the assessment of the performance of proposed adaption measures for both flexible and rigid pavement design alternatives. Using the methodologies described in Step 5, the performance of the adaptive design options was assessed over each alternative's lifecycle. Table 18 presents the anticipated performance of the adaptation measures under the different RCP scenarios.

Table 18: Anticipated Performance of the Adaptation Measures under Different RCP Scenarios.

Adaptation Option	Climate Change Scenarios	Recommended Adaptation Measure	Anticipated Performance
Adaptation Option 1 – Flexible Pavement Design	All Scenarios	Use stiffer Binder	Decreases fatigue damage and rutting by up to 50 percent[146]
Adaptation Option 2 – Rigid Pavement Design	RCP 4.5 and RCP 6.0	Use additional steel in CRCP	Decreases crack width by 6%
Adaptation Option 3 – Rigid Pavement Design	RCP 8.5	Use additional steel in CRCP	Decreases crack width by 6%
		Use stiffer binder for AC overlay	Improves the rutting resistance of the AC Overlay

[146] The pavement performance prediction models used in this study are inadequate to capture the full benefits of polymer modified asphalt binders (e.g., PG 76-YY or equivalent). More detailed characterization of the asphalt binder properties and local calibration of performance prediction models are necessary to facilitate a comprehensive site-specific analysis. However, a nationwide study by Von Quintus and Mallela (2005) concluded that the use of polymer modified binders can reduce the fatigue damage and rutting potential by up to 50 percent.

Step 8. Conduct an Economic Analysis

An economic analysis of each adaptation option is warranted but was not conducted for this case study; however, it is recommended that the project-level pavement design process include a life-cycle cost comparison of alternatives that accounts for the following details specific to the site:

- AADT Construction Year (total for both directions)
- Cars as Percentage of AADT (%)
- Single Unit Trucks as Percentage of AADT (%)
- Combination Trucks as Percentage of AADT (%)
- Annual Growth Rate of Traffic (%)
- Speed Limit Under Normal Operating Conditions (mph)
- Number of Lanes in Each Direction During Normal Conditions
- Free Flow Capacity (vphpl)
- Rural or Urban Hourly Traffic Distribution
- Queue Dissipation Capacity (vphpl)
- Maximum AADT (total for both directions)
- Maximum Queue Length (miles)
- Agency Construction Cost ($1000)
- User Work Zone Costs ($1000)
- Work Zone Duration (days)
- Number of Lanes Open in Each Direction During Work Zone
- Activity Service Life (years)
- Activity Structural Life (years)
- Maintenance Frequency (years)
- Agency Maintenance Cost ($1000)
- Work Zone Length (miles)
- Work Zone Speed Limit (mph)
- Work Zone Capacity (vphpl)
- Traffic Hourly Distribution
- Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)
    - Inbound
        - First period of lane closure
        - Second period of lane closure
        - Third period of lane closure
    - Outbound
        - First period of lane closure
        - Second period of lane closure
        - Third period of lane closure

TxDOT and FHWA have software to facilitate performing this detailed analysis.

Step 9. Evaluate Additional Considerations

Generally speaking, the adaptation measures proposed in Step 6 involve simple and fairly reasonable cost adjustments at the project level and may prove economically beneficial in the longer horizon. The proposed adaptation measures are proven strategies and are routinely used by TxDOT for highly trafficked pavements. Furthermore, the impacts of climate change on pavement performance is not imminent and the adaptation measures can be gradually implemented.

The cost premium of adaptation measures may be fairly low at a project level. However, since the effects of climate change will be systemic and statewide, 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 on road conditions.

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

Step 10. Select a Course of Action

The specific course of action recommended for flexible pavements is to use bituminous mixtures that offer more rut-resistance and fatigue-resistance. Currently, Texas DOT uses performance-based bituminous mixtures that are designed to be rut-resistant or fatigue resistant in some locations. These mixtures, which are typically used for high volume roadways, can also be used for the TX-170 roadway. Additionally, as recommended in Step 6, the use of stiffer binders in the surface layers would satisfy the temperature requirements of the future.

For CRCP, it is recommended to use concrete mix designs that are optimized to reduce long-term drying shrinkage. Additionally, as recommended in Step 6, the use of additional steel will reduce the potential for punchouts.

Step 11. Develop a Facility Management Plan

Climate change poses long-term adverse consequences on the performance of pavements as discussed in earlier sections. Pavements, as an infrastructure system, involve complex interactions 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 Steps 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 SH-170, regardless of which design option is chosen.

- Monitor temperature and precipitation trends over time and assess whether the actual trends are consistent with what the climate models had projected. This monitoring will allow changing the timing and, perhaps, the type of rehabilitation measures.
- Undertake detailed field investigations, including deflection testing[147] and condition surveys, for informing the rehabilitation design.
- Re-evaluate all design-related decisions, including those proposed under Steps 6, using newly available climate information during the rehabilitation planning process.
- 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.

Lessons Learned

This case study provided an engineering analysis of how projected changes in temperature and precipitation trends due to climate change might impact pavement performance for a highway on expansive soils in the Dallas-Fort Worth 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, a steady increase in ambient temperature and aridity is expected over the course of the 21st century. The projected changes in climate patterns will result in higher pavement temperatures along with drier ambient and subgrade conditions which could lead to a modest increase in pavement distresses including AC fatigue cracking, AC rutting, and CRCP punchout potential. However, due to drier subgrade conditions, there may be some benefits from a changing climate associated with a marginal increase in subgrade support conditions. The drier subgrade condition might increase the swelling potential of soil in comparison with shrinkage potential; however, considering the limits posed by non-climatic factors, there might be no change in the total deformation of the pavement. However, the drier subgrade condition may accelerate the lateral root growth of vegetation. Overall, the increase in predicted pavement distresses can be handled using improvements to design pavement thicknesses, material selection and mix design criteria, construction practices, and specification requirements.

[147] Deflection testing applies a pre-defined load to the pavement and measures the pavement's vertical movement in response to that load. Such testing provides a wealth of information on pavement condition.

The primary lesson learned from this case study is the need to monitor changes in climate trends, and to periodically re-evaluate all future design decisions using the newly available climate information. There is a need to move away from the sole dependence on historical climate records and catalogue-type design recommendations 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 shorter 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 binder grade or using shrinkage resistance concrete 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. There are also supply-chain aspects of such changes that need to be considered in the cost calculations. Using a proactive approach to pavement preservation, maintenance, and renewal decisions will offset some of the budgetary constraints, as will an asset management approach that considers risks and solutions for whole categories of assets.

In general, climate plays an influencing role on how a pavement performs over its lifetime. However, there are challenges, such as gaps in climate data and limitations with analytical models, to facilitate a comprehensive evaluation of climate change on pavement performance. For example, future projections of other critical climate variables, such as wind speed and relative humidity, are needed at smaller temporal resolutions (ideally, hourly) with robust data quality. On the other hand, most pavement performance prediction models, which are based on the state-of-the-art mechanistic-empirical concepts, fairly but not explicitly capture the full effects of climate on pavement performance. More often, the pavement performance prediction models incorporate the effects of climate through surrogate measures, such as TMI or freezing index, or indirectly through empirical relationships that correlate physical or engineering responses with observed performance. Therefore, additional research is necessary to overcome the current challenges in the availability of climatic data and pavement performance analytical models.

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