Trunk Highway 13 Project Report

CEGE 4253: Pavement Design, Engineering, and Management

Group 1

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Table of Contents

1. Introduction

The Trunk Highway 13 (TH13) pavement rehabilitation project spans from the Scott-Rice County Line at TH19 to 0.15 miles east of Scott CSAH 17/TH282, covering a total length of 10.2 miles as shown in Figure 1. The highway has been divided into four segments A, B, C, and D based on the number of lanes, shoulder type, and shoulder width. Segment A is a four-lane section, featuring 12-foot-wide lanes and a bituminous pavement where shoulders are 2 feet on the left and 8 feet on the right. Segments B, C, and D consist of two-lane bituminous pavement, including a roundabout at CSAH2/260th. These segments have 12-foot-wide lanes, and their shoulders comprise both 2-foot-wide bituminous and 10-foot-wide aggregate shoulders. None of the segments are subject to seasonal load restrictions, and all are designed to support a 10-ton load. The speed limit across the highway is 55 mph.

The existing pavement conditions are evaluated using parameters such as the Ride Quality Index (RQI), Surface Rating (SR), Pavement Quality Index (PQI), and Remaining Service Life (RSL), indicating significant deterioration across all segments. The entire stretch of the highway suffers from a poor ride due to severe transverse cracking. Critical sections, particularly those from TH19 to 1.5 miles north of TH19 and from 1.5 miles north of TH19 to 0.13 miles north of CSAH2, were found to have an RSL of zero, necessitating immediate rehabilitation. The rehabilitation aims to achieve a 17-year RSL, along with improved ride quality represented by an RQI value of 4 and an SR value of 4.The roundabout at 260th exhibited the highest traffic volume and growth rates, while other segments showed varying growth rates, such as 0.1% between TH282 and 220th and 1.9% between 220th and 240th.

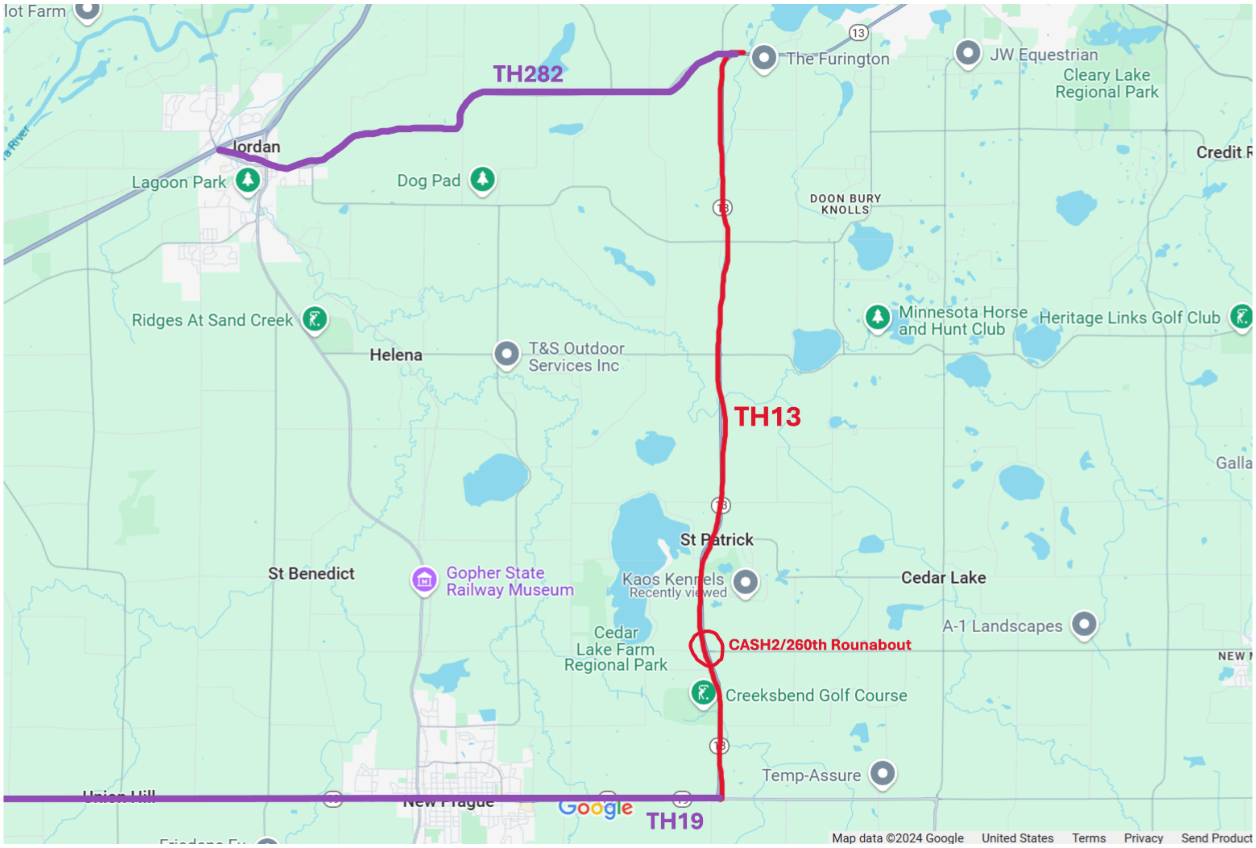

Figure 1: Route of TH13 from TH19 to TH282

Subsurface investigations are conducted using methods like Ground Penetrating Radar (GPR) and core sampling. The GPR result shows slightly higher and more consistent depths in the Southbound (SB) direction compared to the Northbound (NB). Average depths range from 10.6-9.3 inches (NB) and 10.8-9.1 inches (SB) with variability (CV) of 6-13% (NB) and 7-12% (SB). The 15th percentage depths indicating thinner sections range from 9.4-8.4 inches (NB) and 9.9-8.5 inches (SB). Depths decrease slightly along the roadway, reflecting variation in pavement conditions. Whereas from the core the subgrade materials were primarily classified as cohesive soils (C, CL, SL, SiCL). Certain areas, such as the shoulders, were found to have contamination issues requiring remediation, including the addition of 6 inches of excavation and aggregate base. Areas near TH282 were identified as material replacement with selected grading materials and geotextile fabric. Falling Weight Deflectometer (FWD) testing confirmed that the structural capacity of the pavement is sufficient for 10-ton routes under low R-value conditions. For water resource monitoring, nine piezometers were installed, though no further readings are required.

According to MnDOT's pavement design manual, a formal life cycle cost analysis (LCCA) was required for this project due to the pavement area exceeding 60,000 square yards and a design life category of 20 years or more. The analysis identified the most cost-effective opinion as a 2-inch mill + 4-inch cold-in place recycling (CIR) + 3-inch.

2. Rehabilitation Method

The three rehabilitation methods are proposed for this project: 2" Mill + 4" CIR + 3" OL, 5.5" Mill + 7" Whitetopping and 5.5" Mill + 7.5" Whitetopping. The Life Cycle Cost Analysis (LCCA) provides a comprehensive framework to ensure that the chosen pavement rehabilitation approach is cost-effective and sustainable over the long term. The selected and approved design pavement rehabilitation is a combination of 2 inches of milling, 4 inches of Cold In-Place Recycling (CIR), and a 3-inch bituminous overlay. This approach was chosen for its cost-effectiveness, ease of construction, and minimal disruption to traffic and residents.

Table 1: Comparison of proposed pavement rehabilitation methods

	2" Mill + 4" CIR + 3" OL	5.5" Mill + 7" Whitetopping	5.5" Mill + 7.5" Whitetopping
Description	Milling removes the top 2" of severely deteriorated pavement, while CIR targets issues within the recycled layer.	Milling removes the top 5.5" of existing pavement and whitetopping provides a stronger and durable surface for heavy traffic.	Milling removes the top 5.5" of existing pavement and whitetopping provides a stronger and durable surface for heavy traffic.

Pros	· CIR addresses intermediate-level distresses and improves the structural capacity of the pavement. · Cost effective and quicker to implement.	· It is suitable for severe damage/weak subgrade. · It is preferred where there are heavy trucks and repeated loading such as roundabouts, ramps and bypasses.	· It is suitable for severe damage/weak subgrade. · It is preferred where there are heavy trucks and repeated loading such as roundabouts, ramps and bypasses.
Cons	· It addresses moderate distresses. · It is not appropriate where the subbase is weak.	· It is not cost-effective for segments with moderate damage. · Concrete requires a longer curing time compared to asphalt, which increases the duration of lane closures.	· Overdesign for the pavement rehabilitation for our project.

The current pavement performance condition of TH13 has low and medium severity transverse cracking, spalling and longitudinal multiple, and alligator cracking, along with a poor ride quality due to deteriorated transverse cracking. Core samples revealed significant damage or layers separating from each other at depths between 4 and 5 inches. To address these issues, the damaged layers will be removed through milling and replaced using a Cold In-Place Recycling (CIR) process. The milling and recycling thickness have been carefully chosen to eliminate the identified distresses in the existing bituminous and restore the roadway's condition. In areas with soft shoulders, an additional 6 inches of excavation and the placement of 12 inches of aggregate base is recommended. The shoulders will undergo saw cutting, milling to remove the existing material, before being rebuilt. At TH282 (roundabout), the black clay should be removed and replaced with selected grading material, reinforced with geotextile fabric to stabilize the embankments. For existing right-turn lanes, bypasses, and ramps at TH19 the material will be milled and replaced with 3 inches of a wearing course. For the cattle pass, all existing pavements will be removed and replaced with a series of layers including a 3-inch wearing course, 4-inch CIR, 8-inch non-wearing course, and a 17-inch aggregate base reinforced with geotextile fabric.

3. Coring vs. GPR

Two testing methods, coring/boring and GPR, were used to determine the thickness of the pavement on this project, which are described in the section below.

3.1. Coring/Boring

Coring and boring are destructive testing methods for pavements. Coring refers to cutting out a cylindrical sample from the pavement, which can then be taken back to a lab for further testing and analysis. Coring uses cylindrical core drills to drill through the pavement, and then specialized tongs/extractors are required to extract the core from the pavement without damaging it; the hole in the pavement is then filled. Boring refers to drilling a hole into the pavement. While no sample can be extracted, it’s still possible to measure the thickness of the pavement structure. Borings can typically go much deeper than cores.

Coring/boring equipment is relatively inexpensive (compared to GPR) and doesn’t require a trained technician to interpret the results. Additionally, the results require little to no interpretation since they can be seen and directly measured. However, it is a destructive testing method; taking cores introduces distresses into the pavement surface, which needs to be filled. This means it’s not possible to take very many cores/borings, otherwise the pavement will become very distressed. Because of this, cores/borings are typically only taken about every hundred yards or so at randomly determined points. Due to the nature of random samples, the core samples might not necessarily be a representative sample of the entire pavement structure. Additionally, localized features can be easily missed by cores (for example, if there is a patch of stripping which is less than 100 feet long). Coring/boring can also be disruptive to traffic as well as dangerous for the ones testing, since it requires them to stand still on the road for a period of time while they are taking the samples. Finally, cores can be somewhat unreliable in certain circumstances, as they may partially disintegrate for various reasons (such as stripping), making the pavement seem thinner than it is in reality.

3.2. GPR

Ground penetrating radar (GPR) is a non-destructive testing method for pavements. GPR units consist of a transmitter and a receiver. The transmitter emits electromagnetic signals, which can be of fixed or variable frequencies. These signals then penetrate downwards into the pavement. When the signal encounters a new material with a different dielectric constant (for example, when it goes from the asphalt layer to the base layer), a portion of the signal will be reflected back towards the surface and is received by the receiver. The time taken for the signal to be emitted and received is measured. If the dielectric constant of the material is known (which, for pavements, is typically done by taking cores), then the depth that the signal reaches can be calculated, giving the depth of the pavement layers. GPR units can be divided into 2d GPR and 3d GPR. 2d GPR measures the depth over a longitudinal distance, and 3d GPR measures the depth over longitudinal and transverse distance. GPR units can also be divided into air-coupled and ground-coupled GPR. Ground-coupled GPR units are placed directly on the pavement surface, and can accurately measure deeper structures like the base and subgrade layers, but cannot be used at traffic speed. Air-coupled GPR units are held a few inches above the pavement surface, and cannot penetrate as deep as ground-coupled GPR, so they are generally used only for the surface layer (and in some cases the base layer). However, they can be used at traffic speed.

Figure 2: Photo of a core taken from TH 13

Unlike coring/boring, GPR is a non-destructive testing method, and so it can be used over the entire pavement structure without damaging it. This gives more complete and accurate data of the pavement structure, and allows for more detailed analysis. GPR data can also be collected at traffic speeds or close to traffic speeds, making safety and traffic disruption less of a concern. However, GPR units are very expensive, costing hundreds of thousands to millions of dollars for a single unit. Additionally, since GPR data can be difficult to interpret at times, a technician needs to be hired to interpret the results, adding to the time and cost. Finally, GPR data is very context-dependent and may not be possible to interpret by itself; as a result, other data from different types of tests (such as cores or FWD data) should be used in conjunction with GPR data to be able to interpret it. However, GPR can be used to reduce the number of cores taken by identifying areas of interest.

4-5. GPR & Recycling Depth Analysis

Figure 3 below shows the GPR data for both lanes, the coring/boring point data, and the milling depth for the selected rehabilitation strategy (which was 2 inches). The coring/boring data mostly lines up with the GPR data, although there are a few points that lie above or below the GPR thicknesses. As mentioned in the previous section, this may be because the bottom of the cores have disintegrated, or because the cores were taken at a point where the pavement was thicker/thinner than the local average, as the GPR data is averaged over every 100 feet. Around the 2 mile mark, the GPR shows a thickness of 0 at one point, which is most likely an outlier. There is also a section from around 5.2-5.9 miles where the pavement becomes thicker. One reason for this is that the pavement structure is different in this section, possibly due to a past rehabilitation. Another possibility is that the GPR data was falsely interpreted in this section; for example, maybe the bottom of the base layer was mistaken as the bottom of the bituminous layer due to ambiguous data. Save for these two exceptions, the pavement thickness seems to be relatively uniform. The mill depth is above the bottom of the bituminous layer and doesn't go into the base layer, except of course for the one outlier at 2 miles.

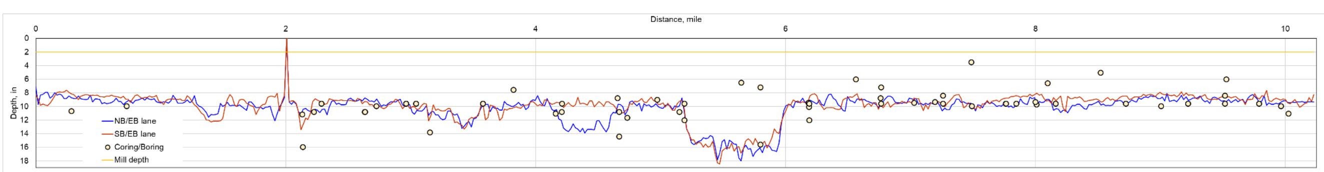

Figure 3: plot of GPR data, coring/boring data, and the recommended mill depth

Table 2: Summary statistics of GPR data

	NB/EB	SB/EB
Average	10.19177	9.842834
Std. dev	1.949885	1.928289
Max	18.00004	18.43987
Min	0	0
Min (no outlier)	6.924128	7.604861

The average depth agrees with the average reported in the MDR. However, the standard deviation, as well as the maximum and minimum, show a much larger variability. This is likely due to the one outlier, skewing the minimum, as well as the deeper section around 5.2-5.9 miles, which is skewing the maximum and the standard deviation. In the MDR, it was decided to remove this section of the pavement from the analysis because it did not line up with the core data. If this was due to misinterpretation of the GPR data, then removing this section of the data would be appropriate. However, if this section truly does have a different structure, then it should be analyzed separately. Given that the core data doesn't show any difference in structure, the former explanation is more likely. To confirm this, the two sections of the pavement were analyzed separately in the following table:

Table 3: Summary statistics of GPR data separated

	NB, miles 0-5.2 and 5.9-10.2	NB, miles 5.2-5.9	SB, miles 0-5.2 and 5.9-10.2	SB, miles 5.2-5.9
Average	9.762167	15.66893	9.402381	15.44354
Max	13.86753	18.00004	13.41534	18.43987

Min	6.924128	12.63735	7.604861	12.31336
Stdev	1.074791	1.112434	0.972574	1.169776

The construction records show that this road was initially built with a 5 inch aggregate base, with 10 inches of bituminous on top after a series of overlays. Therefore, the bottom of the base layer is expected to be 15 inches deep, and the bottom of the bituminous layer is expected to be 10 inches deep. Given that average depth of the GPR data from miles 5.2 to 5.9 is 15 inches, and the average depth outside of this section is 10 inches, and the core data shows a depth of about 10 inches in this section, the most likely conclusion is that the bottom of the base layer was mistaken for the bottom of the bituminous layer in the GPR data, which is why this section seems thicker than the rest. Therefore, the decision to remove this section from the GPR data is appropriate.

Table 4: Summary statistics of boring/coring data

	Borings	Cores
Average	8.793333	10.04706
Stdev	2.474447	1.673277
Max	13.8	15.96
Min	3.48	6.6

The table above presents the summary statistics for the coring/boring data. These statistics line up with those presented in the MDR. They also corroborate the statistics from the GPR data, minus the section from 5.2-5.9 miles. The boring statistics seem to be more variable, but this is likely because there are fewer samples. The core statistics are very similar to the GPR statistics.

Another statistical measure that could be useful is the coefficient of variation (CV). Unlike the standard deviation, which is an absolute measure of variation, the CV is a relative measure of variation, and so it can be used to compare the variability of this pavement thickness with other pavements that may be thinner or thicker. For the GPR data, the CV for the NB and SB lanes are 11% and 10% respectively, which lines up with the CV in the MDR. Reporting the interquartile range can also be a useful measure of variation as opposed to the maximum/minimum, since this measure is not affected by outliers as much. For the GPR data, the 1st and 3rd quartiles for the NB lane are 9.1 in. and 10.4 in. respectively, and for the SB lane they are 8.8 in. and 10.0 in. Finally, hypothesis testing can be used to compare the mean pavement thickness. For example, if we wanted to know whether or not the mean GPR thickness was significantly different from the expected thickness of 10 inches, then we can construct a t-test with a significance level of 0.05. Doing so, we calculate a p-value of 0.29, which is greater than 0.05. Therefore, we conclude that the mean thickness of the GPR data is not significantly different from the expected mean of 10 inches. Alternatively, if we wanted to know whether or not the mean GPR thickness and the mean core thickness were statistically different, we could construct a two-sample t-test with a significance level of 0.05. Doing so, we calculate a p-value of 0.91, which is greater than 0.05, and so we conclude that the mean GPR thickness and core thickness are not significantly different.

6. Proposed Design Analysis

In analyzing the proposed design MnDOT created for Truck Highway 13 (TH13), multiple factors were considered in making a pavement design. The most important factors are Traffic forecast and overall depth of the road in designing a way to rehabilitate the pavement. Knowing how both present and future traffic would be on the section of the road proves a way to estimate the 20-year reliability of any design. The overall depth of the road provides a good estimate of what the pavement design should fall in. The design should make sure the overall road is even with itself to cause no bumps to appear. As for where this information is located in the report, the Material Design Recommendation provides the traffic and depth information for the design. The GPR thickness report goes more in-depth on the overall depth and thickness of TH13, which can help know the specific depths for certain sections of the highway for a design.

For this report, section B of TH13 will be used to provide a constraint variable to help compare the design that MnDOT picked compared to different hypothetical designs. Since section B is constrained, the traffic forecast and overall depth will also stay roughly the same as well. This means that the traffic forecast used in FM-1702A while section B has a growth rate of 1.9% with the BESALs value being 1,252,000. As for depth, it is roughly around 9 inches for the HMA depth, so the design will keep this in mind.

To run each different design, MnPave was used to help design and test out the 20-year reliability of each design. The chosen design that MnDOT used, Design #1, was run through MnPave to help provide a comparison point. It would show why this strategy was chosen through different designs while also proving a control design to be compared to different hypothetical designs. For the custom designs, two are created to test theoretical pavement designs that MnDOT could have used for TH13. The two designs are named Design #1A and #1B to further show that they will be compared to the original Design #1.

For the chosen Design #1, it's a two-layer pavement design with a focus on the HMA layer. Two inches of old asphalt layer was milled, which is why it's 7 inches, and 7 inches of asphalt was added on top. Making it so the design HMA layer is 14 inches in total, proving 100% 20-year reliability. This shows why this design was chosen, with having high reliability and having a CIR layer to reduce the overall cost of the rehabilitation.

20 Year Reliability: 99.9% (90% recommended) 5,000 cycles

Project Information

District: Metro	**County:** Scott	**City:**
Project No.: 7001-123	**Route:** TH19	**Ref. Post:** 72.249 to 72.917
Letting Date: 12/1/2024		
Designer:	**Soils Engineer:**	

Climate Information

Seasons: 5 **Location:** 44° 42.06' Latitude, 93° 30.2' Longitude

Structural Information

Layer	Type	Subtype	Thickness, in.
1a	Hot-Mix Asphalt	C - PG58H-34, 4% Pb, Size A	3.00
1b		Cold Recyc (CIR), 5% Pb, Size A	4.00
1c		Old PG58H-34, 4% Pb, Size A	7.00
2	Engineered Soil	Clay Loam	

Traffic Information **Forecast #:** FM-1702A

Speed: 55 mph **Growth Rate:** 1.9% **Design Flexible ESALs:** 1,252,000

Figure 4: MnPave report of Design #1 that MnDOT created for TH13

For Design #1A, the focus of this design is to limit the added asphalt and CIR as much as possible while keeping the old HMA layer within reasonable bounds. As seen below, this design works if only the old asphalt is especially thick. The old asphalt needs to be at least around 12 in for this design to handle the necessary traffic. The GPR report shows that it's uncommon for TH13 to have asphalt that deep on any section of the highway. So, compared to Design #1, it is inferior due to it being limited to only being usable to thicker pavement.

20 Year Reliability: 92.9% (90% recommended) 5,000 cycles

Project Information

District: Metro	**County:** Scott	**City:**
Project No.: 7001-123	**Route:** TH19	**Ref. Post:** 72.249 to 72.917
Letting Date: 12/1/2024		
Designer:	**Soils Engineer:**	

Climate Information

Seasons: 5 **Location:** 44° 42.06' Latitude, 93° 30.2' Longitude

Structural Information

Layer	Type	Subtype	Thickness, in.
1a	Hot-Mix Asphalt	C - PG58H-34, 4% Pb, Size A	1.00
1b		Cold Recyc (CIR), 5% Pb, Size A	2.00
1c		Old PG58H-34, 4% Pb, Size A	9.50
2	Engineered Soil	Clay Loam	

Traffic Information **Forecast #:** FM-1702A

Speed: 55 mph **Growth Rate:** 1.9% **Design Flexible ESALs:** 1,252,000

Figure 5: MnPave report for Design #1A that was created for this report

Design #1B is a three-layer pavement design where focus was placed on the effects of having 1 ft of engineered has on reducing the amount of HMA needed. The reports show that two inches can be removed from the design compared to Design #1. This is important to note as there aren't any plans to fully reconstruct any existing part of TH13. Some research can be done to determine if the soil underneath the pavement is compacted enough while seeing if past construction had the soil engineered to be a foot in depth. Since the report doesn’t mention if there has been research on this information, it is inconclusive whether or not this design is viable during construction. This means that Design #1, with the current information, is still the better design for TH13. Though for future rehabilitation of TH13, this design should be kept in mind if further research is conducted for TH13.

20 Year Reliability: 94.0% (90% recommended) 5,000 cycles

Project Information

District: Metro **County:** Scott **City:**

Project No.: 7001-123 **Route:** TH19 **Ref. Post:** 72.249 to 72.917

Letting Date: 12/1/2024

Designer: **Soils Engineer:**

Climate Information

Seasons: 5 **Location:** 44° 42.06' Latitude, 93° 30.2' Longitude

Structural Information

Layer	Type	Subtype	Thickness, in.
1a	Hot-Mix Asphalt	C - PG58H-34, 4% Pb, Size A	2.00
1b		Cold Recyc (CIR), 5% Pb, Size A	3.00
1c		Old PG58H-34, 4% Pb, Size A	7.00
2	Engineered Soil	Clay Loam	12.00
3	Undisturbed Soil	Clay Loam	

Traffic Information **Forecast #:** FM-1702A

Speed: 55 mph **Growth Rate:** 1.9% **Design Flexible ESALs:** 1,252,000

Figure 6: MnPave report for Design #1B that was created for this report