

KENTUCKY TRANSPORTATION CENTER

College of Engineering

**PAVEMENT/SUB-GRADE CONDITION ASSESSMENT I-65
APPROXIMATE MILEPOST 97.5 TO 102.5
(TRANSITION FROM ASPHALT TO CONCRETE) TO (KY 313 OVERPASS)**





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**PAVEMENT/SUB-GRADE CONDITION ASSESSMENT 1-65
APPROXIMATE MILEPOSTS 97.5 TO 102.5
(TRANSITION FROM ASPHALT TO CONCRETE) TO (KY 313 OVERPASS)**

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Kentucky Transportation Cabinet

Commonwealth of Kentucky

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I. Introduction

The Kentucky Transportation Center was contacted by the Kentucky Department of Transportation (Division of Highway Design) to conduct a condition assessment of the pavement structure on I-65 from mileposts 97.5 (transition from asphalt to concrete) to 102.5 (KY 313 overpass). The following report contains the practices and procedures used to perform the condition assessment, the processed results obtained from the collected field data, and some preliminary recommendations that may assist designers in the event that this section of roadway is either reconstructed and/or rehabilitated.

II. Methodology

In efforts to perform an assessment of both the existing pavement and the underlying subgrade conditions, the Kentucky Transportation Center employed the use of three types of equipment/infrastructure analyzers: drill truck for taking field cores; falling-weight-deflectometer (FWD) to measure sub-grade modulus and load transfer across transverse pavement joints; and ground penetrating radar (GPR) to determine pavement layer thickness, subgrade thickness, and an estimate of subgrade moisture variability. The results from each test procedure may be found below.

III. Cores and Visual Assessment of Subgrade Conditions

Cores were extracted in three locations, in the northbound inside-lane, to verify both the concrete pavement layer thicknesses and the approximate depth of subgrade material and type. In addition, two more cores were taken to verify both the condition of the dowel bars located in the transverse pavement joints, and the condition of the tie assemblies used between adjacent lanes.

a.) Pavement Layer Thicknesses from Cores and Approximate Depth of Subgrade Material and Type (Measured in Field).

Table 1: Pavement layer thickness

Location from M.P. 100 (feet) north	Lane	Approximate Station Number	Concrete Thickness (inches)	Depth of D.G.A. beneath Concrete (inches)	Depth Subgrade/Type beneath D.G.A. (inches)
2175	NB inside	567+05	12	2	12 (rock & D.G.A. mixed)
3800	NB inside	583+30	10.75	8	4 (clay)
12550	NB inside	670+30	11	2	2 (clay)

b.) Dowel Bar Condition

One transverse joint dowel bar was extracted in the northbound middle lane at station number 567+00, to determine the integrity of the protective coating on the dowel bar (Figure 1). As seen in Figure 1, the protective coating appears to be in

place and in good condition. Therefore, it appears that the transverse joint dowel bars have not corroded.



Figure 1: Transverse Joint Dowel Bar

c.) Tie Assembly Between Adjacent Lanes

The tie assembly placed in the longitudinal joint between adjacent lanes was inspected to determine its long-term functionality at Station Number 567+00 NBFL (Figure 2). As seen in figure 2, the tie assembly appears to have sheared away from one of its connected sides. Therefore, it is speculated that this longitudinal tie will not support the pavement in the event of differential settlement between lanes. However, no differential settlement was observed in this area or throughout the majority of the project (Figure 3).



Figure 2. Longitudinal Joint Tie Assembly.



Figure 3. No Presence of Differential Settlement between Middle and inside Lane

IV. FWD Analysis

In-situ material testing was done utilizing the Falling-Weight-Deflectometer (FWD) at three locations along the project in the northbound direction. Testing was conducted in the inside and middle lanes adjacent to materials sampling locations. The testing consisted of evaluation of the load transfer of the transverse joints and estimation of the subgrade strength using backcalculation of layer moduli. The following table provides a summary of the testing results.

Table 2: FWD load transfer and CBR values

Station #	Load Transfer		Subgrade CBR	
	Inside Lane	Middle Lane	Inside Lane	Middle Lane
567+05	0.86	0.91	5.2	2.2
583+30	0.93	0.91	2.8	2.0
670+30	0.83	0.70	1.6	4.2

It may be seen from the above data that the load transfer is greater than 0.70 which is considered to be in good condition. Typical new pavements would have load transfer from 0.90 to 1.0. The subgrade CBR values are somewhat low, which is most likely a factor of the moisture conditions within the subgrade. This has been illustrated with the results of the ground penetrating radar testing, in that several areas exhibited above normal moisture conditions.

V. Ground Penetrating Radar (GPR) Analysis

Two 900 MHz. ground coupled antennas were used to quantify the degree of saturation of the subgrade material, the concrete pavement layer thickness, and the approximate subgrade thicknesses. The GPR collection rate was performed at three scans per foot.

a.) Degree of saturation

In an attempt to quantify the degree of saturation of the subgrade material, the amplitudes of the reflected radar signal were analyzed at the concrete/D.G.A. interface. Areas that displayed amplitudes greater than 8000 dB were considered to be fully saturated areas. Areas with amplitudes less than 4000 dB were considered to be areas with both dry and normal moisture contents for subgrade material. Finally, areas that had amplitudes ranging between 6000-8000 dB's were considered transition areas--from fully saturated to normal/dry moisture contents.

Figure 4 displays the degree of saturation for the three northbound lanes. Note areas marked with PW, on Figure 4, were areas observed in the field to be pumping water between the shoulder and driving lane as truck traffic passed by (Figures 5, 6). Figure 7 displays the degree of saturation for the three southbound lanes.

Northbound I-65: degree of moisture beneath concrete
 approx.milepoints 97.5 (asphalt to concrete) to 102.5 (KY 313 overpass)

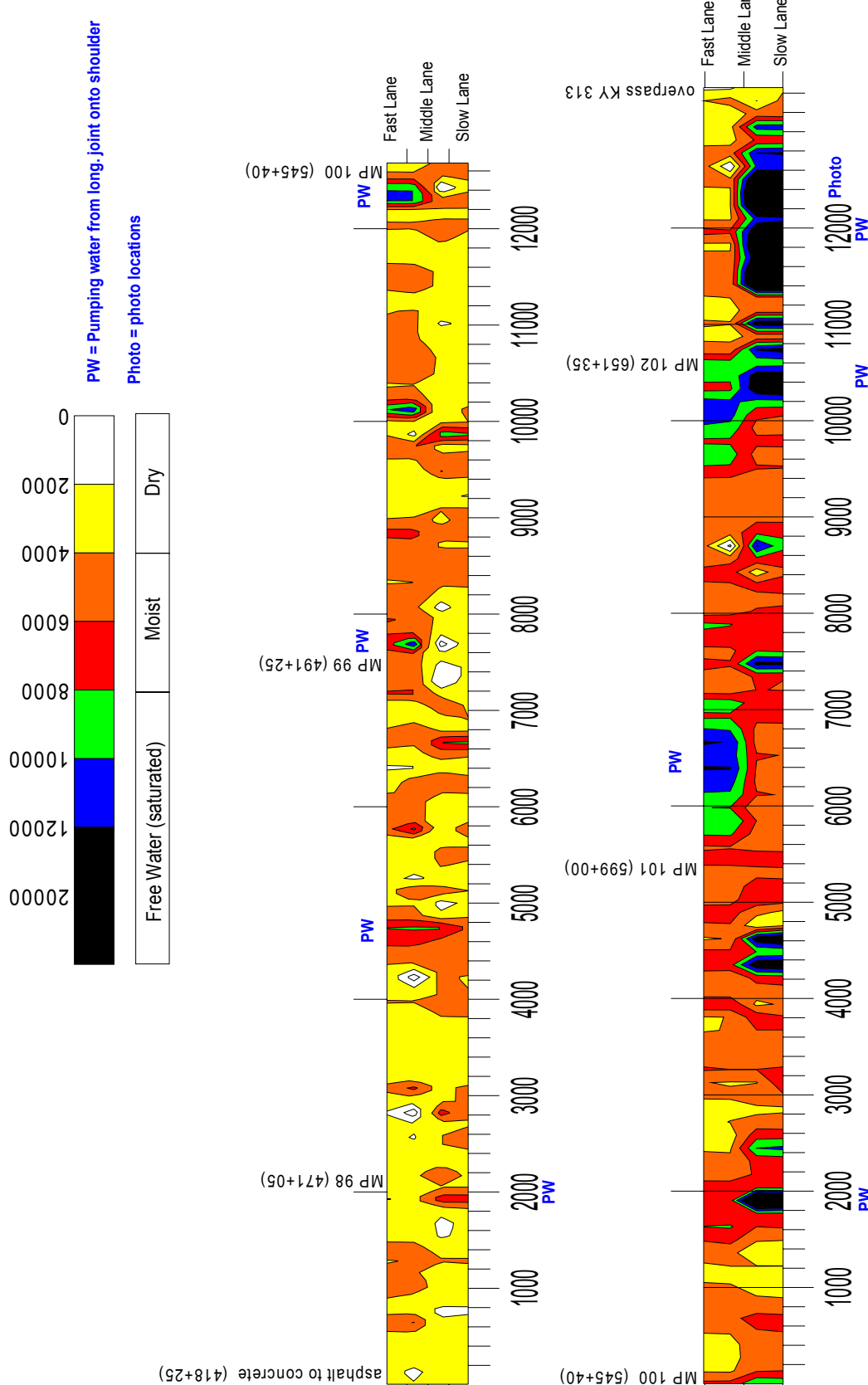


Figure 4: Degree of Moisture Beneath the Concrete Surface (Northbound)



Figure 5: Water pumping Up from Shoulder Joint (Northbound)



Figure 6: Water Bleeding Out of Shoulder Joint in Patched Area

Southbound I-65: degree of moisture beneath concrete
 approx. milepoints 102.5 (KY 313 overpass) to 97.5 (asphalt to concrete)

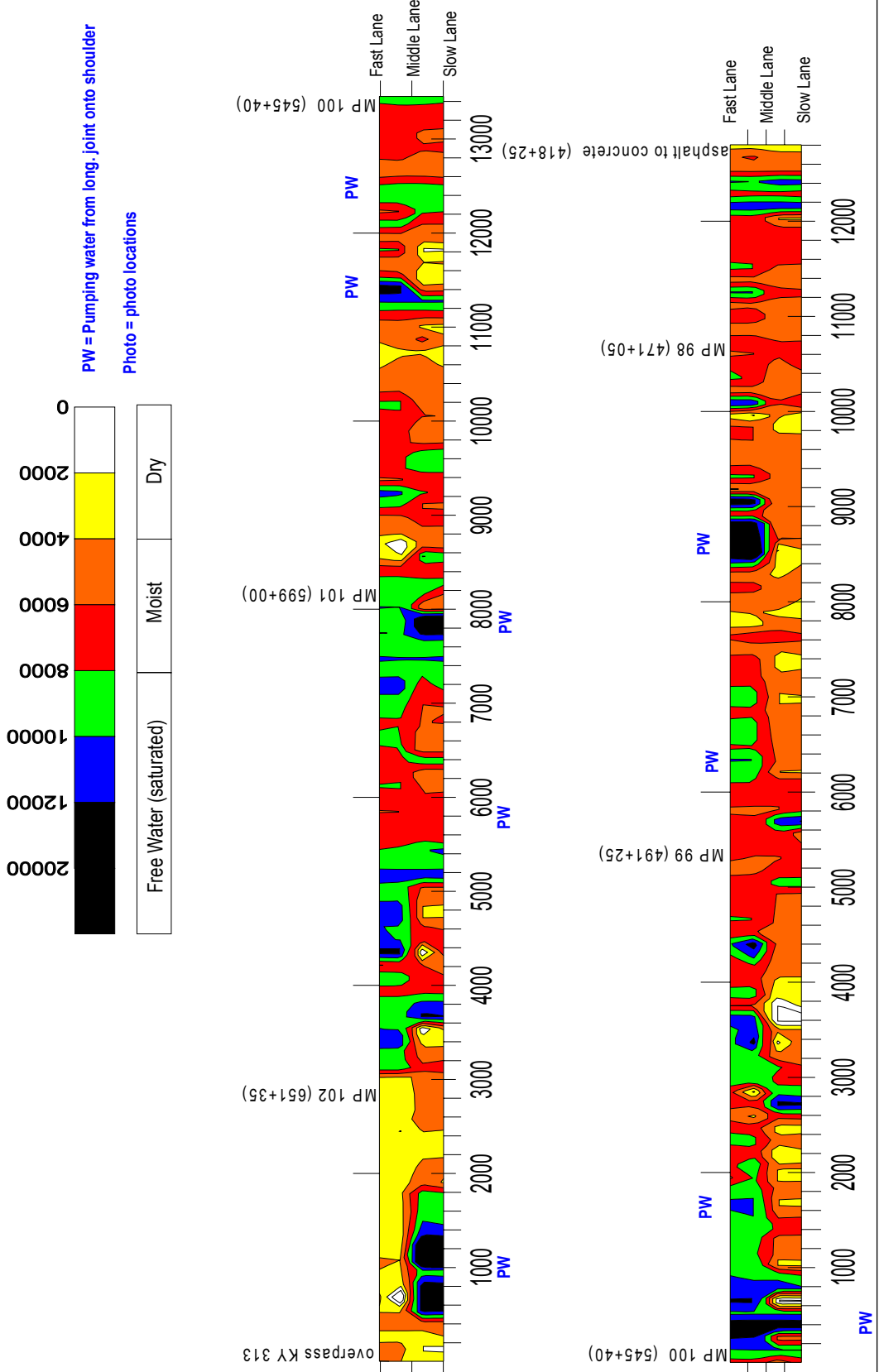


Figure 7: Degree of Moisture Beneath the Concrete Surface (Southbound)

b. Concrete Thickness

GPR was utilized to determine the thickness of the concrete in three of the six driving lanes. The thickness averages for the three lanes may be found in the table below. Note, the recorded thicknesses are plus/minus 1.0 inch.

Table 3: Average concrete thickness

Driving lane	Average concrete thickness	Figure number
Southbound outside	11.08	8
Northbound outside	10.74	9
Northbound inside	10.54	10

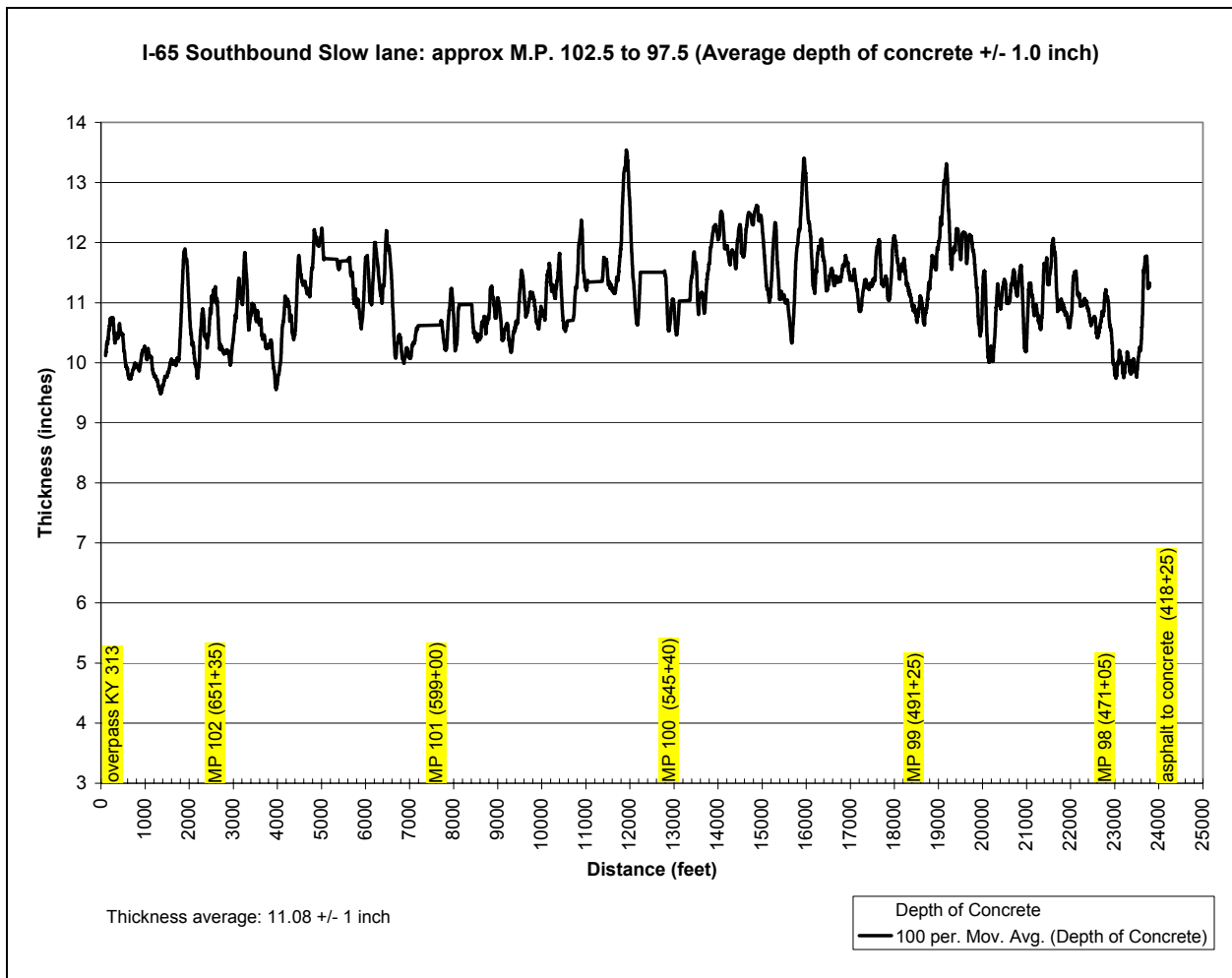


Figure 8: Concrete Thickness Southbound outside lane

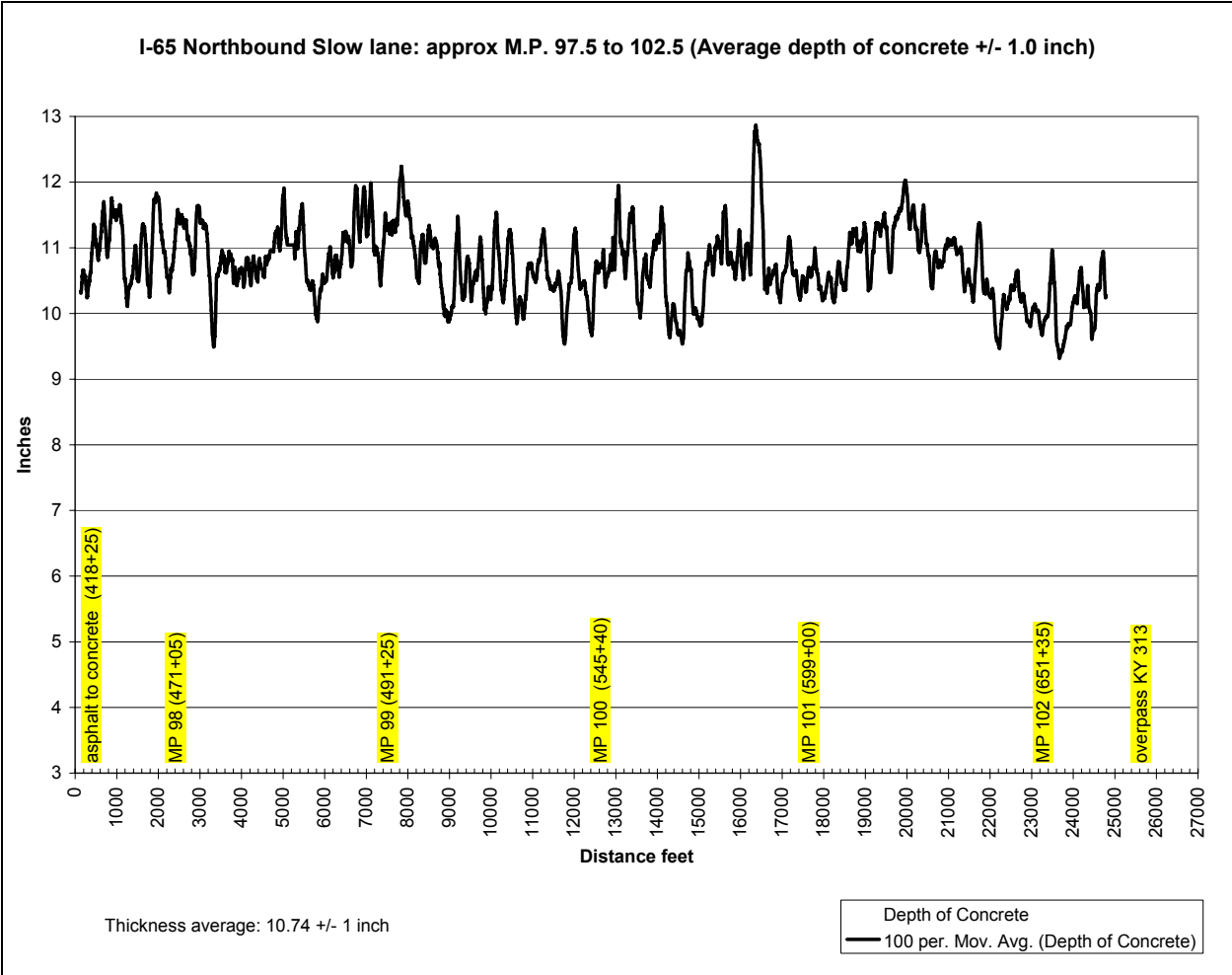


Figure 9: Concrete thickness Northbound outside lane

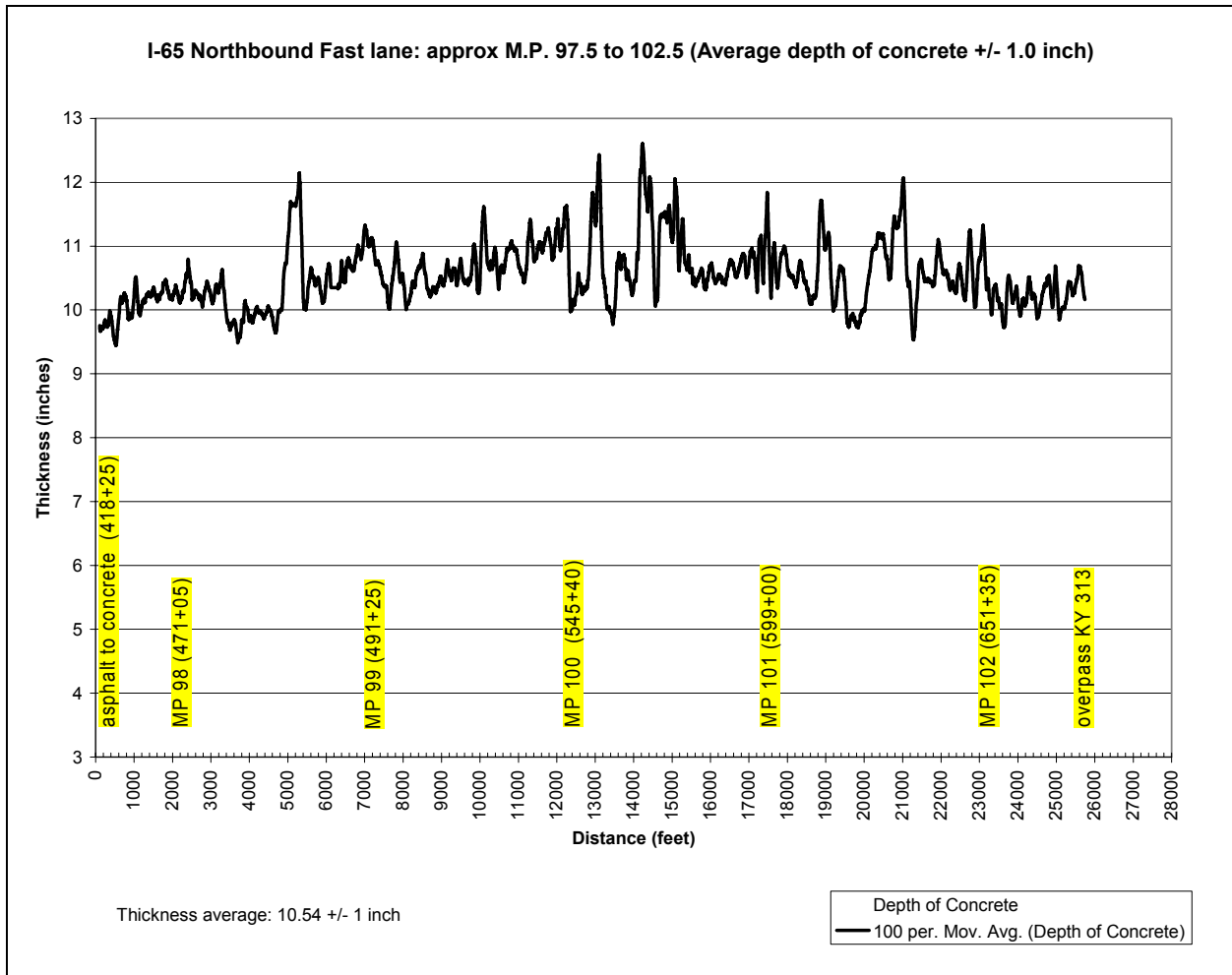


Figure 10: Concrete Thickness Northbound inside Lane

c.) Subgrade Quantities/Approximate Thicknesses

An attempt was made to approximate both the depth of the subgrade material and its material composition using GPR. Figure 11 identifies the subgrade thickness and its composition beneath the concrete layer for the southbound lanes. Figure 12 identifies the subgrade thickness and its composition beneath the concrete layer for the northbound lanes. Note, areas marked in green in Figures 11 and 12 indicate locations that had less than four inches of D.G.A. material beneath the concrete layer.

I-65 Southbound: approx. subgrade thickness beneath concrete (+/- 2.0 inches)

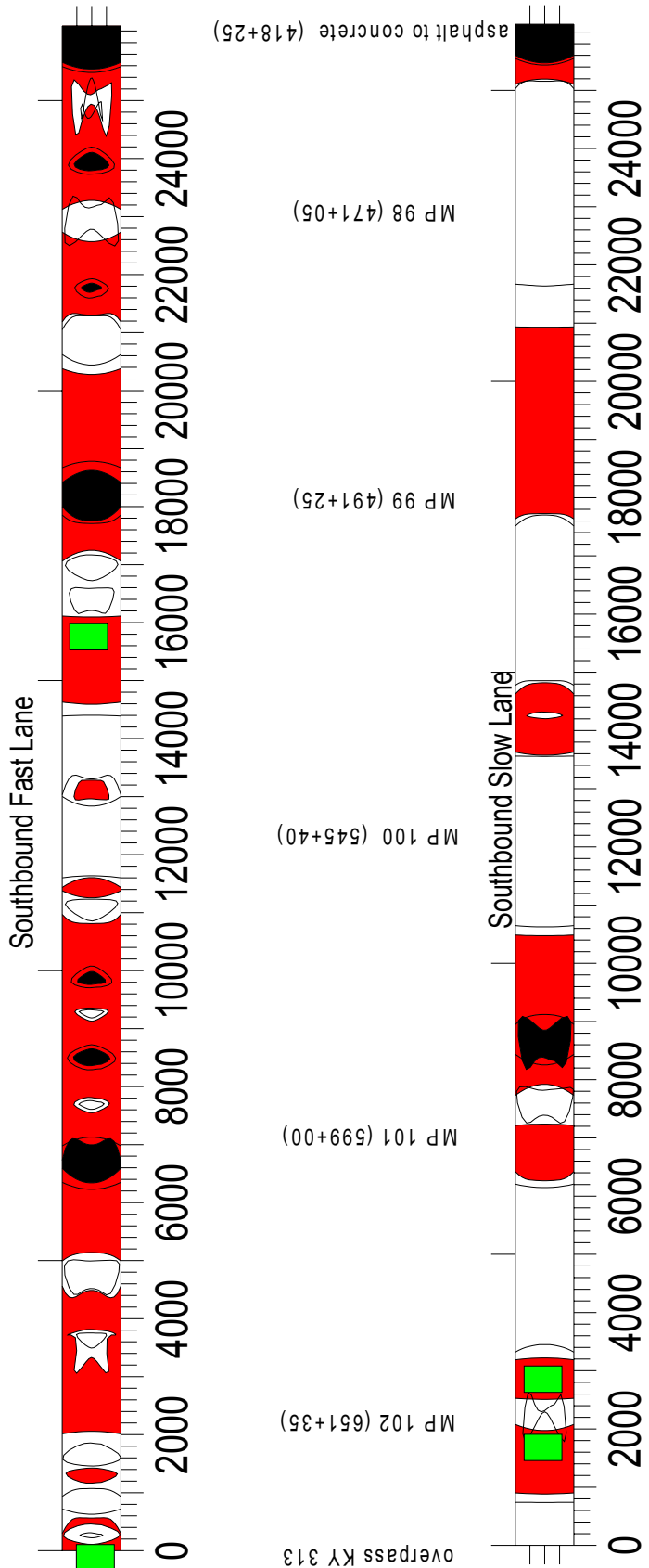
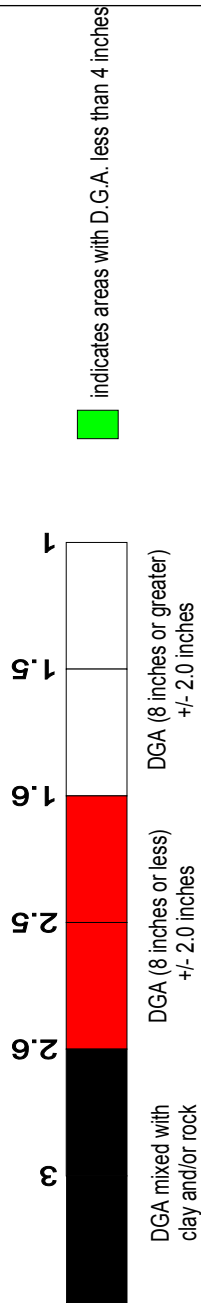


Figure 11. Subgrade Thickness and Material Composition Beneath Concrete Layer Southbound

I-65 Northbound: approx. subgrade thickness beneath concrete (+/- 2.0 inches)

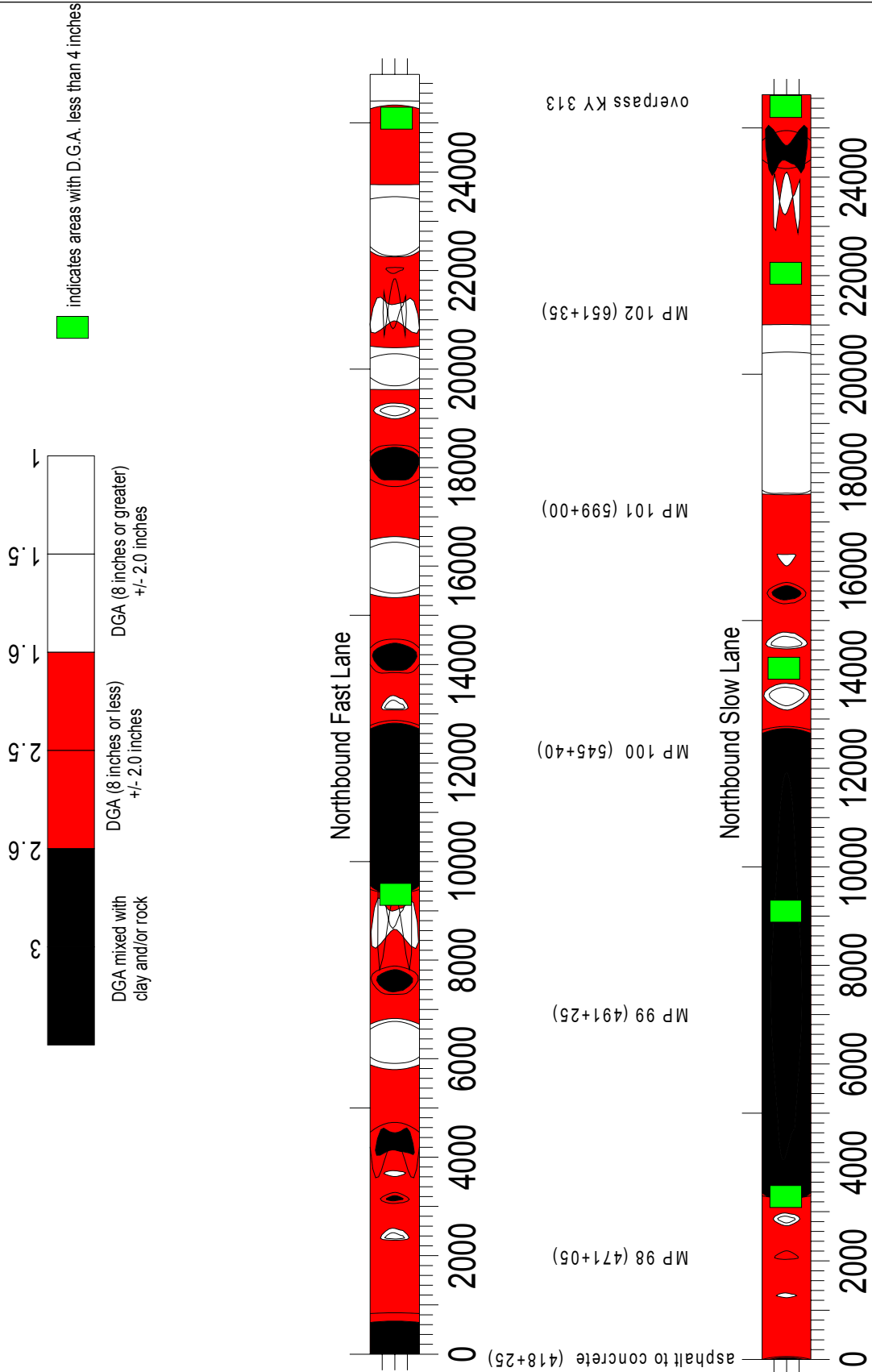


Figure 12. Subgrade Thickness and Material Composition Beneath Concrete Layer Northbound

VI. Concrete Surface Conditions/Durability

Many of the transverse joints on the project have moderate to severe deterioration, which is manifested in the form of spalling (Figure 13). In addition, a large number of the joints that currently do not display distress have hairline cracks on both sides of the joint, which is the precursor for spalling (Figure 14). Most of the spalls appear to be one to two inches in depth (Figure 15).

It is not clear what the primary causative factor is in the formation of the spalls. The research team did not test any for the cores obtained from the project. Although water has certainly exacerbated the problem of spalling, it is the opinion of the researchers that there may be an inherent problem with the concrete itself which has produced the excessive amount of spalling; however, there was no means of confirming that hypothesis.



Figure 13: Spalled Joint



Figure 14: Hairline Cracking Around Concrete Joint



Figure 15: Depth of Spalled Joint

VII. Summary

The condition assessment of the concrete paving surface on I-65 from mileposts 97.5 (asphalt to concrete transition) to 102.5 (KY 313) overpass identified several areas of concern. First, the durability of the existing concrete surface appears to be the most detrimental problem encountered in this survey. A vast majority of the pavement joints have exhibited some type of spalling between slabs. The large number of spalled joints leads the researchers to believe that there may be an inherent problem with the concrete itself. Further material testing may be necessary to accurately define this problem. Second, in several areas the subgrade drainage appears to be at a minimum. It would be expected, and has been shown in the FWD analysis, that the sub-grade strengths would be the lowest in these areas. Third, several areas were identified to have less than 4 inches of D.G.A. beneath the concrete surface. In the event that these areas become saturated in the future, it is probable that differential settlement may occur between the driving lanes. Lastly, if differential settlement does occur between the different driving lanes, it is of the researchers opinion, that the existing longitudinal tie assembly between the lanes will not provide the adequate support to deter such settlement.

Although there are several areas of concern as noted above, there are several positive aspects of the existing condition of the project. First, the transverse dowel bars do not appear to be failing due to corrosion. Therefore, joint blow-ups are somewhat limited throughout the project. Second, only a few selected areas throughout the project appear to have less than 4 inches of D.G.A. as a subbase material. Most areas appear to have adequate depths of D.G.A. material. Therefore, very few areas have a clay layer right beneath the concrete paving surface. Third, load-transfers over the transverse joints appear to be in an acceptable range for the three tested sections. Lastly, the concrete thickness for the project appears to average 10.5 inches or greater (+/- 1 inch).

VIII. Recommendations

In the opinion of the researchers, the joint spalling problem appears to be the main concern along with inadequate drainage. The spalling problem may limit the options for rehabilitation of this project. To simply overlay the concrete slab with thick asphalt mat does not appear to be a viable option. The joints will continue to work with temperature and will continue to deteriorate further, causing serious reflective cracking in a few years. Also, overlaying a concrete slab with an asphalt mat without rubblelizing causes the temperature in the old concrete slab to be higher than before (the black asphalt mat transfers more heat to the old slab). This will cause more blow-ups. Therefore, it appears that the best alternative would be an unbonded concrete overlay. The second alternative would be to rubblelize the existing concrete pavement with a sonic-head breaker and overlay with asphalt. Again, drainage would need to be addressed with either alternative.