INVESTIGATION OF EARLY CRACKING ON SELECTED JPCP PROJECTS

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16. Abstract

From 1995 through 2000 the department constructed 15 pavement reconstruction projects, totaling 282 lane-miles, with jointed plain concrete pavement (JPCP). Five of those projects developed mid-slab transverse cracking shortly after construction. The extent and severity of the cracking among those projects are highly variable. The primary purpose of this research study was to determine any obvious causes for the cracking that may be related to design, construction, or material related factors without conducting a complete forensic analysis. Secondary objectives were to recommend needed specification changes, follow-up research, and weigh the feasibility of using preventive maintenance treatments to extend the service life of the cracked pavements.

For three of the five distressed projects, the investigation found that poor construction practices were the primary cause for the slab cracking. For the two others with a greater extent of mid-slab cracking, the cracks initiated top-down from a loss of slab support at the transverse joints when the slab corners are upward curled. A “built-in” positive temperature differential condition was found with these projects that accentuate the time and magnitude of the slab uplift, as these two projects far exceed the temporary loss of support from normal temperature curling. In addition, concrete properties and slab length contributed to their mid-slab fatigue cracking. Several types of multi-axle trucks have axle configurations that match the joint spacing, causing simultaneous joint loading, which prevents slab rotation to relieve the impact force. Several recommendations to prevent slab cracking are given, plus needed research to better holistically understand the cracking phenomenon.

17. Key Words
jointed plain concrete pavement, transverse cracking, built-in curling, temperature gradients, loss of support

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MICHIGAN DEPARTMENT OF TRANSPORTATION
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Investigation of Early Cracking on Selected JPCP Projects

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Pavement Research Unit
Construction and Technology Division
Research Report RC-1501

Lansing, Michigan
December 2007

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This research study was conducted as a joint effort between the Michigan Department of Transportation (Department) and the University of Michigan under the direction of Professor Will Hansen in the Department of Civil and Environmental Engineering. Dr. Hansen served as principal investigator for this study. He is a recognized authority in the field of concrete pavement performance analysis. Department contribution came from many areas, including the Pavement Research Unit in the Construction & Technology Division and Region Offices, with special recognition to the Metro Region staff in Soils and Materials.

The content of this report were prepared by both the department and Dr. Hansen. David Smiley, Supervising Engineer of the Pavement Research Unit, was the primary author. Dr. Hansen provided the data analysis and figure development and text review. The conclusions and recommendations are the opinions of both authors which are solely based upon the study findings.

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EXECUTIVE SUMMARY

From 1995 through 2000 the department constructed 15 pavement reconstruction projects, totaling 282 lane-miles, with jointed plain concrete pavement (JPCP). Five of those projects developed mid-slab transverse cracking shortly after construction. Cracking was found on a sixth project four years after construction as this study was concluding. The extent and severity of the mid-slab cracking among those projects are highly variable. The 2000 distress survey data shows the percent of cracked slabs in the driving lane ranges from a few percent to about 75 percent. More importantly, the extent of cracking on some projects is increasing at a geometric rate.

The primary purpose of this research study was to determine any obvious causes for the cracking that may be related to design, construction, or material related factors without conducting a complete forensic analysis. Secondary objectives were to recommend specification changes, follow-up research, and weigh the feasibility of using preventive maintenance treatments to extend the service life of the cracked pavements.

The investigation began in August 2000 as part of the department's Pavement Research Center of Excellence at the University of Michigan. The projects with cracking involve three Regions and Michigan's major freeways: I-94, I-96 and I-75. The investigation included extensive deflection testing, surface profiling, crack surveys, and laboratory material testing. The results from past research projects were used to supplement this study's findings.

The investigation found that poor construction practices were the primary cause of the cracking for three of the five projects. The cause of the cracking of the two remaining projects on I-96 in Livingston County and on I-94 in Berrien County is a complex combination of factors. The study findings offer a hypothesis as to the cause for the cracking, which involves four factors working interactively. The factors are: (1) a significant loss of slab support at the transverse joint during upward slab curling, (2) poor load transfer efficiency (high displacements) across the joint, (3) the concrete's sensitivity to crack initiation and rapid crack propagation, and (4) simultaneous loading of each end of a slab from a fully loaded, multi-axle truck. Preliminary finite element modeling confirms these factors working interactively can cause sufficient stress to transversely crack a slab. Fatigue failure initiates when the stress buildup exceeds 50 percent of the slab's tensile strength, which is the accepted threshold to prevent fatigue cracking. This finding also agrees with national literature.

The major contributor of the four is probably loss of support, which is the emphasis of a continuing research study. JPCP is susceptible to the initiation of mid-slab cracking when a loss of support is present during an upward curled condition. The magnitude and duration of the slab curling is greater than normal for the I-94 and I-96 projects because of a positive temperature differential condition that existed when the concrete set. Unfortunately, preventive maintenance is not a suitable means to impede the initiation of new cracking. Several recommendations to prevent slab cracking are given, plus needed work and research to better holistically understand the cracking phenomenon.

1.0 INTRODUCTION

In 1995 after a long absence, the Department resumed construction of jointed plain concrete pavement (JPCP) for new and reconstruction projects to negate the occurrence of transverse cracking. However, shortly after their completion five projects began developing transverse cracking and some of those developed a limited amount of longitudinal cracking. As a result, on June 2, 2000, the Engineering Operations Committee (EOC) approved a research project to investigate the extent and likely causes of the cracking. On August 2, 2000, the Department and the University of Michigan (UM) began a joint investigation project, under the direction of Dr. Will Hansen, as principle investigator. The Department's Pavement Research Unit and various Regions were significant partners in the field studies that led to this report.

The joint Department/UM investigation study consisted of the following work tasks:

- Conduct a field investigation of the projects with cracking to gather pertinent data regarding the pavement's current distress condition, cross section, longitudinal profile, and material usage.
- Confirm from project construction records information about weather conditions, concrete placement temperatures, and quality control testing that could affect the initiation of cracking.
- Analyze the data among projects for commonality or trends to determine possible causes for the cracking.
- Compare findings with other ongoing Department field and laboratory research regarding JPCP to supplement data analysis from this study.
- Estimate ramifications of cracking on future project maintenance needs, especially any change for service life estimates.

Shortly after this research study started, a subsequent research project[1] was awarded in October 2000 to Dr. Hansen to specifically investigate the likely possibility that loss of slab support (slab-base contact) in the vicinity of the transverse joints is a primary cause for the initiation of mid-slab cracking. Excessively high joint deflections with corresponding low load transfer efficiencies were found on some JPCP projects with transverse cracking during early site investigations. This JPCP study and the slab-base investigation project then continued concurrently.

2.0 BACKGROUND

Historically, non-reinforced concrete pavement is not new to Michigan as hundreds of lane-miles of JPCP were constructed prior to WWII. Those JPCP designs consisted mainly of 20 ft. long slabs, usually eight or nine inches thick, constructed on a dense-graded aggregate base or directly on the prepared soil subgrade. Even so, jointed reinforced concrete pavement (JRCP) was the predominate pavement type constructed during the same era and became Michigan's standard rigid pavement after WWII. JRCP was used exclusively throughout the interstate construction era, except for limited use of continuously reinforced concrete pavement, mostly in metro

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[1] The project was "Qualify Transverse Cracking in PCC Pavement from Loss of Slab-Base Contact".

Detroit. JRCP was favored over JPCP for two primary reasons; a superior ride quality and the probability of less adverse consequences (less joints to replace) from potential D-cracking and other forms of material related distress.

2.1 Past Research Studies of Concrete Pavement Performance

Although this report is about the premature mid-slab cracking of JPCP, a brief summary of the Department's past research studies of JRCP performance is included for insight into possible reasons for JPCP cracking among possible design, load, material and environmental related causes.

During the late 1960s and early 1970s, the department conducted several intensive research studies regarding the performance of post WWII new pavement projects (all JRCP). The construction period studied was from the end of WWII to the early 1960s, when the Department used a standard rigid pavement design consisting of the following attributes that were seldom modified:

- 9 in. slab thickness for rural pavements and 10 in. for urban pavements.
- 99 ft. slab length between contraction joints that were sealed with hot-poured rubber.
- The joints were doweled without corrosion protection.
- A steel plate was placed on the base beneath the joint to prevent fines from intruding into the joint relief crack.
- A 4-6 in. thick dense-graded aggregate base was used over a 12-15 in. thick sand subbase.
- Concrete mix did not vary; proportions depended on the type of coarse aggregate (gravel, carbonate, blast-furnace slag) that the contractor selected.
- The coarse aggregate was a blend of two sizes (4A, 10A).
- HMA shoulders consisting of two courses over a six inch, dense-graded gravel base.

The two studies highlighted are typical of the research performed at that time. They relied heavily on a statistical approach to find any correlation between the pavement's design parameters and performance (distress and ride condition). Until the mid 1980s the condition of all freeway rigid pavements was manually surveyed the year after construction and thereafter at five-year intervals. The field distress survey consisted of detailed crack maps that described the crack's actual location and orientation, plus any associated distress features. The field crack maps were the primary data used for each research study.

"Performance of Michigan's Postwar Concrete Pavement"
Research Project 39 F-7 (15)
Research Report No. R-711
June 1970

The purpose of this study was to determine the general level of performance of postwar concrete pavements with 5-15 years of condition data (distress and ride), to find any relationship with certain design, construction, material, or environmental factors. The study reviewed 520 construction projects consisting of 1880 directional miles constructed between 1946 and 1961. The majority of the projects were performing satisfactorily, but a small number were rated poor. The poor pavements became the basis of the study.

Transverse cracking, medium 2.5 cracks per slab at 15 years, and joint spalling were the primary distress types. Joint blowups started to occur after 15 years (equates to the 1946–1954 construction era). In general, the extent and severity of distress among all projects were highly variable. Researchers were able to correlate joint blowups with the type of coarse aggregate. Their frequency was much higher when the coarse aggregate contained higher amounts ($\geq$3 percent) of soft, non-durable particles. Typically, the distress condition of the right travel lane was 65 percent worse than the passing lane. The report concluded that the key causes for the accelerated distress condition (poor project performance) were material (aggregate quality) selection, construction quality, the environment, and to a lesser extent traffic loading.

"General Evaluation of Current Concrete Pavement Performance in Michigan"
Research Project 69 F-110
Research Report No. R-905
March 1974

This intensive, lengthy study attempted to use a statistical approach (Markov chain) to find a correlation with a particular performance (distress condition) factor. At first, the study attempted to use factor analysis to match a pavement's general distress or roughness condition with a particular cause, but found that a pavement's distress condition was too highly variable, as pavements typically exhibited a different predominate distress type. A total of 128 construction projects were studied. Among those, 43 had 15 or more years of service.

Eventually, transverse cracking was chosen as the key study variable because of its frequent occurrence among pavements constructed from 1946 to 1961. A project model found an excellent relationship with cracking and joint spalling that could be used to predict the future potential for blowups, which were a major safety and maintenance concern.

Of the material, construction, and environmental factors studied, only the coarse aggregate type appeared to have a significant role on joint condition over time. Joint performance was much better when the carbonate content of the coarse aggregate was less than 10 percent or greater than 80 percent. Other single aggregate types or percentage combinations had mixed results. Neither the pavement's geographic location nor accumulated traffic was found to affect condition relative to aggregate selection.

Among all construction projects studied, the only factor that appeared to correlate with the occurrence of transverse cracking was accumulated traffic.

2.2 Clare Test Road

The findings from these research studies encouraged the department to renew its interest in evaluating shorter slab lengths (JPCP) as a means to eliminate troublesome transverse cracking and joint blowups. Research project 73F-0136[2] was created to evaluate the performance of several experimental cross sections as part of the new US-10 freeway (CS 18024 JN 06601), constructed in 1975 northwest of Clare. The 500 ft. long test sections consisted of short (12 ft. to 19 ft.), non-reinforced pavement slabs on various base types. Over time the JPCP sections were compared with a JRCP control section (department standard pavement) with 71 ft. joint spacing on only a 4 in. dense-graded (22A) aggregate base. The JPCP experimental bases were designed to contrast the benefits of higher stiffness (bituminous base) versus improved permeability, by using free-draining, asphalt stabilized, open aggregate base (PATB).

The results from the Clare study did not conclusively demonstrate which concrete pavement type is the superior performer. As expected, most JRCP slabs cracked transversely, while only some 19 ft. JPCP slabs cracked near mid-slab. For approximately the first 13 years both pavements exhibited only low severity joint spalling. Shortly thereafter, the distress rates for both types accelerated. After 17 years the JPCP section on a bituminous base had developed such severe joint distress and pronounced map-cracking over its entire surface that the slabs in the entire test section were replaced with full-depth bituminous. Today, both the remaining JPCP sections and the JRCP sections have extensive, severe joint spalling, both longitudinal and transverse, which requires continuous reactive maintenance patching. The cause of the concrete deterioration is now being investigated as part of a statewide research project on material-related distress. The entire US-10 project, including the study segments, is scheduled for major rehabilitation in the Bay Region's five-year plan. Because performance results were inconclusive, the US-10 project did not initiate any additional JPCP projects, but it did encourage the eventual standard design change to open-graded bases in the mid-1980s.

2.3 Recent Design Changes to JRCP

In 1994 a special department committee[3] was formed by EOC to recommend modifications to current design standards for JRCP and to explore the possibility of constructing some JPCP projects. Specifically, the committee was charged with determining the benefits of reducing joint spacing to reduce the occurrence of transverse cracking and the objectionable ride quality that resulted if faulting developed. The standard joint spacing in 1994 was 41 ft., which was further reduced to 27 ft., when daily commercial traffic counts exceeded 3000 vehicles per lane. Field surveys found that 41 ft. slabs usually developed two transverse cracks, while 27 ft. slabs formed one or no cracks. Joint spacing had been reduced from 71 ft. to 41 ft. in 1979. JRCP with 27 ft. joint spacing was first used in 1987 to reconstruct I-75 in northern Monroe County. The committee conducted extensive field reviews, including site visits to Illinois, Wisconsin and Minnesota (FHWA Region V states) to ascertain details regarding their current rigid pavement designs.

In January 1996, EOC approved the committee's recommendations to adopt the following standard design changes for JRCP:

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[2] The final research project report remains incomplete as a rough draft, although several interim condition reports were prepared when the study was active. Over time, project 73F-0136 continued to house other studies dealing with concrete pavement performance, particularly those relationships related to aggregate durability.

[3] The Rigid Pavement Condition Review Committee, which was chaired by John Kelsch, Executive Division.

- Eliminate 41 ft. joint spacing and use only 27 ft. spacing.
- Adopt a 14 ft. wide slab (driving lane only) for rural projects with non-tied concrete or bituminous shoulders to reduce edge deflections.
- Standardize concrete shoulder designs to eliminate “sympathy cracking” from non-reinforced tied concrete shoulders.
- Allow an option of using a dowel bar inserter in lieu of load transfer “baskets”.

The committee reported that the use of JPCP is strongly endorsed nationally by the concrete paving industry and is favored by the vast majority of other state DOTs that construct rigid pavements. Since the mid 1990s, the Michigan Concrete Paving Association (MCPA) has requested that the department only construct JPCP, especially when a conditional pavement warranty is specified.

EOC decided to allow JPCP as an option to JRCP for new and reconstructed pavements. EOC charged the Pavement Selection Review Committee[4] (PSRC) with the responsibility for deciding when JPCP should be used and to provide regular updates on its relative usage versus JRCP. A 200 lane-mile limit for constructing JPCP was set. During the mid 1990s, except for very high volume commercial routes, JPCP was usually selected for most new or reconstruction projects, as its’ life-cycle cost[5] was estimated to be lower than JRCP. The 200 lane-mile limit was reached about two years later, but then was increased by EOC to 350 lane-miles. The 350 lane-mile limit was met during the 2001 construction season. Presently, Regions recommend either JPCP or JRCP with Pavement Committee concurrence. Final pavement type approval is by EOC depending upon which pavement type has the lowest life/cycle cost.

3.0 PERFORMANCE OF RECENT JPCP PROJECTS

Table 3.0 lists all the JPCP projects constructed since 1995[6] through the 2000 construction year. The table includes only new or reconstructed projects. This study did not include JPCP utilized for rehabilitation projects, like unbonded overlays. Most unbonded concrete overlay projects constructed since 1990 are JPCP. Thus far, Department PMS data shows the unbonded JPCP projects are performing satisfactorily and are expected on average to meet service life estimates. JPCP has been selected for some inlay projects[7] where severely distressed JRCP was removed and replaced within a single lane. The inlay option was selected for these rehabilitation projects

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[4] In April 2000 PSRC was disbanded by EOC and replaced by the newly formed Pavement Committee.

[5] Since 1997, per state law, the department must select the pavement type with the lowest life cycle cost for projects exceeding one million dollars in total pavement costs. The Equivalent Uniform Annual Cost method is used to determine the pavement’s life cycle cost per annum. Costs include construction, future preventive maintenance, and user delay for the pavement’s estimated service life.

[6] The first two JPCP projects listed in Table 3.0 were originally designed as JRCP, but changed to JPCP as special demonstration projects in partnership with MCPA.

[7] The first inlays were constructed on I-75 (existing 41 ft. JRCP in Monroe County) in 1997 and on M-14 (existing 71 ft. JRCP in Washtenaw County) in 1998. These projects only replaced the pavement slab without any base or drainage modifications. Joint spacing was variable to match working cracks and joints in the adjacent lane that remained in-place with CPR performed.

Table 3.0 JPCP Projects Constructed From 1995 Through 2000

Region	Control Section	Project No.	Route	Location	Lane Miles	Design ESAL's (million)	Year Paving Completed	Paving Contractor
Metro	63191	36003*	I-96/I-275	Ramp from NB I-275 to WB I-96	4	28	1995	Carlo
Southwest	11017	32516	I-94	Friday Road E'ly to M-140 (Watervliet)	20	32	1995 EB 1996 WB	Interstate
University	47065	28215*	I-96	Chilson Road E'ly to Dorr Road	21	20	1997 EB 1998 WB**	Interstate
Metro	82061	38063*	EB US-12	RR Overpass E'ly to Howe Street (Wayne)	4	4	1997	Ajax
University	19033	33577	US-27	Chadwick Road N'ly to Price Road	16	7	1998	Ajax
University	19033	33576	US-27	Clark Road N'ly to Chadwick Road	14	5	1998	Ajax
Metro	82194	36005*	I-75	Fort Street to West Grand Blvd.	19	37	1998	Ajax
Southwest	11018, 80023	38094	EB I-94	M-140 E'ly to West of Hartford	11	37	1998	Interstate
University	81076	44603	US-23	Plank Road N'ly to Carpenter Road	2	25	1999	Interstate
Metro	82195	38100	I-75	NYC RR N'ly to Gratiot Avenue	15	57	1999	Ajax
Metro	82125, 63191	45752	I-275/I-96	5 Mile Road N'ly to I-696	50	44	1999	Carlo
Southwest	80024	45855	EB I-94	M-51 E'ly to M-40 (Paw Paw)	12	34	1999	Interstate
Southwest	80023	45859	I-94	62nd E'ly to M-51 (Hartford)	36	34	2000	Ajax
Southwest	12033	45535	I-69	Ohio Line N'ly to Lake Warren Rd.	44	21	2000	Interstate
University	81105	38009	M-14	I-94 E'ly to US-23 BR (Ann Arbor)	14	28	2000	Ajax
					282 Total			

* Project investigated for this study
** WB median lane paved in fall of 1996

because an inlay was much more cost effective than extensive patching with full-depth concrete repairs. In 1998/99 JPCP was also used to replace 17 concrete freeway ramps among I-75, I-94, and M-59 in the Metro Region.

3.1 Related Pavement Specification Changes

The primary design parameters and current condition of the JPCP projects in-service at the time of this study are summarized in Table 3.1. Several design parameters were changed during the period of JPCP construction being studied. Two parameters are deserving of some detailed explanation as background for this study report.

Open-Graded Drainage Course

From 1995 through 1997 the aggregate gradation for Open-graded Drainage Course (OGDC) was modified several times. During that period JPCP projects used a variation of a 3G OGDC. For those projects, the actual aggregate gradation selected by the Contractor, although unintended by specification, was typically a gap-graded 6A/AA, as it normally met the modified 3G gradation being specified. The Contractor, without consideration for its' comparable performance, would select a 6A/AA because it was usually readily available, whereas, a 3G was made special for a particular project. Also, because 6A was readily on-hand, its purchase cost could be lower.

If 3G was specified, the paving contractor usually claimed a stability problem existed with the 3G during paving. MCPA continually requested that the department modify or discontinue using the 3G OGDC for future projects. After numerous partnering meetings, a new series, 4G, OGDC was developed and tested in the field. In 1997, 4G became the standard gradation for all future rigid pavement projects. 4G differs from 3G in that the gradation specification requires particles to be retained on the No. 8 and No. 30 sieves to improve stability (more well-graded), but with some acceptable loss in drainability[8]. A comparison of the specification gradations for 3G, 3Gmod, and 4G along with 6A/AA follows:

Sieve Size	3G % passing	3Gmod % passing	4G % passing	6A/AA % passing
1 ½"	100	100	100	100
1"	85/100	85/100	85/100	95/100
½"	40/70	40/75	40/70	30/60
No. 4	–	–	–	0/8
No. 8	0/30	0/20	15/35	–
No. 30	0/13	0/8	5/20	–
LBW	5.0 max	5.0 max	5.0 max	1.0 max

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[8] The department does not use drainability to accept OGDC. Laboratory testing per MTM 122 has verified that the gradation for 4G will exceed 350 ft/day permeability, which the department considers a minimum desirable value.

Table 3.1 Design and Condition Data for JPCP Projects Constructed From 1995 Through 2000

PROJECT			DESIGN PARAMETERS							CONDITION				
Route	CS	Comm ADT (dir) 1999	Slab			Shoulder Type	OGDC/ Separator	PCC Mix	Coarse Aggregate Type	Average DI 1999	Average RQI 1999 (9)	Description	Slabs Cracked (%)	Remarks
			L (ft)	T (in)	Widen Y or N									
I-96 Ramp*	63191	3500	16	12	N	Tied Conc.	3GM1/Textile	35P	6A carb	<1	40	13 slabs are cracked - in sequence	1.0	Cracks in cut section
I-94**	11017	4000	Var (1)	12	Y	Bit.	3G/Textile (2)	35P	6A slag	EB 15 WB <1	EB 45 WB 38	Most TCs are spalled (3). Some slabs with 2 cracks.	EB 60 WB 10	JPCP with most cracking
I-96*	47065	3500	16	11	N	Conc.	3G/Agg (8)	35P	6A slag	EB <1 WB <1	EB 39 WB 36	Almost all TCs are on EB in clusters	EB 6.3 WB <1	% cracked for outer lane
US-27	19033	1050	14	9	N	Bit.	4G/Agg	35P	6A Gr	<1	NB 47 SB 50	No visible cracking	0	See note (4)
US-27	19033	1050	14	9	N	Bit.	4G/Agg	35P	6A Gr	<1	NB 45 SB 49	No visible cracking	0	
EB US-12*	82061	500	13 to 19	8.5	N	C&G	3G/Textile	35P	6A carb	?	47 (7)	Isolated cracking from loading & jointing flaws	< 1	Longer slabs are cracked
I-75*	82194	6000	16	12	N	Tied Conc.	4G/Textile	P1-mod (6)	4/6 carb (6)	NB <1 SB 1.1	NB 45 SB 47	Both TC & LC - mostly clustered in isolated areas	NB 0.39 SB 0.14	Warranty
EB I-94	11018 80023	5300	15	12	Y	Bit.	4G/Agg	P1	6A carb	<1 (11)	35-30	No visible cracking (11)	0 (11)	Warranty
US-23	81076	3700	15	11	Y	Tied Conc.	4G/Agg	P1	6A carb	new in 1999	new in 1999	Not surveyed for distress	unknown	Warranty?
I-75	82195	6000	15	12	N	Tied Conc.	4G/Textile	P1-mod (6)	4/6 carb (6)	new in 1999	?	No visible cracking	new	Warranty
I-275/I-96	82125 63191	7500	15	12	N	Tied Conc.	4G/Textile	P1-mod (6)	4/6 carb (6)	new in 1999	NB 40 SB 42	No visible cracking	new	Warranty
EB I-94	80024	5300	14	12	Y	ex. Conc. Rt new Conc. Lt.	4G/Textile	P1	6A carb	new in 1999	RL 41 LL 40	No visible cracking	new	Inlay design (5)
I-94	80023	5300	14	12	Y	ex. Conc. Rt. new Conc. Lt.	4G/Textile	P1	6A carb	new in 2000	new in 2000	No visible cracking	new	Inlay design (5)
I-69 (10)	12033	2750	15	10	Y	Bit.	4G/Agg	P1-mod (6)	4/6 carb (6)	new in 2000	NB 33 SB 39	Used DBI / No visible cracking. Sometime "rocking" ride on NB.	new	Grade Raise Warranty
M-14	81105	2000	15	11	Y	Bit.	4G/Textile	P1	6A carb	new in 2000	new in 2000	No visible cracking	new	Warranty Perf. Incentive

NOTES:

* Project was investigated for this study

** Project was investigated in earlier study. Findings included in this report.

(1) EB has three joint spacings of approximate equal length: random @ 15-16-17 ft., "Illinois" hinge, and 16 ft. uniform (location from west to east). WB is entirely 16 ft. uniform. I-94 was a trial project approved by EOC (January 1995).

(2) 3G OGDC had compaction requirement at 95 percent of MD

(3) Approximately 1000 ft. (portion with LC) at east end of EB was replaced in 1998 with project no. 38094. Spalling is entirely on approach (after) side of crack

(4) SB portion of project from Chadwick Road to Price Road developed extensive longitudinal cracking during construction (first 3 days of paving). Also, some TCs formed from late sawing of relief cuts - all TC panels replaced by Contractor. Before acceptance, 76 - 12 x 14 ft. panels with LC were replaced. Many slabs with very short LCs extending from TJs were left in-place.

(5) Inlay consisted of replacing pavement and reusing existing OGDC (a geotextile separator was added). Joint spacing matches existing shoulder joint spacing – shoulder is non-reinforced

(6) SP required revised concrete mix design. Coarse aggregate has lower dilation and maximum absorption requirements. Approximate blend of 40 percent 4AA and 60 percent 6AAA, by weight.

(7) Value based on only 77 percent of length (PMS survey data)

(8) A test section using a 350AA OGDC was used at east end of westbound for all lanes. Sta 759+50 to 797+50 OGDC was limestone (Pit 58-11) Sta 797+50 to Sta 837+55 was blast-furnace slag (Pit 82-19)

(9) RQI values for projects constructed after 1998 are converted from Contractor profilograph measurements.

(10) SB I-69 has test location with unsealed contraction joints between MP 3.6 (sta 315+800) and MP 4.8 (317+800). Signs mark location. Normal expansion joints were used.

(11) In Summer 2002 Southwest Region reported project has developed mid-slab cracking. Project investigated in parallel UM study.

A recent completed research project[9] that investigated the support characteristics of various OGDC gradations, including 3G and previous gradations, for JRCP bases concluded that OGDC did not contribute to slab cracking or crack spalling. Since the change to a 4G OGDC in 1997, both the department's and contractor's satisfaction with OGDC bases have greatly increased.

P1-Modified Concrete Mixture

A modification to the department's concrete mix design, known as P1-mod, was another significant change during this period. In 1997, the department began using P1-mod for some new and reconstruction projects on major statewide freeways. The objective of P1-mod was to extend the pavement's service life by resurrecting several attributes from older (pre 1970) JRCP projects that were exhibiting outstanding, long-term performance. The key adopted requirements of P1-mod are; a 40/60 blend of two coarse aggregate sizes with low dilation and absorption properties, a limit on total cementitious content that allows only Type F fly ash, the importance of timely and adequate curing, and a diligent awareness of unfavorable environmental conditions during paving. The immediate motive was to eliminate early plastic shrinkage cracking with both JPCP and JRCP and to enhance aggregate interlock across cracks and joints. Longer term, the reduced paste content and lower permeability rate were expected to provide a higher resistance to water induced, material-related distress. If transverse cracking occurred, the larger (2 ½ in. top size) 4AA coarse aggregate is expected to "reinforce" essential load transfer capability. The Appendix contains an example of a recent Special Provision for the P1-modified concrete mixture.

3.2 Preliminary Condition Reviews

Transverse cracking with JPCP was first observed in 1996 on EB I-94 (CS 11017), near Watervliet, about a year after its construction. The Southwest Region was periodically reviewing the project because longitudinal cracking had occurred during paving in 1995. At the east end of the project, pronounced longitudinal cracking developed running parallel to the centerline. The cracking was determined to be a result of late sawing of the longitudinal relief cut in combination with high (near 90°F) concrete mix temperatures and paving during unfavorable summertime conditions. The Contractor repaired the cracking at no cost with "stitching", using guidelines from the American Concrete Paving Association. In the same area the slab's surface also exhibited extensive map-like, drying shrinkage cracks. In 1997, the department began an investigation of the Watervliet project, in conjunction with the fore-mentioned, open-graded research project, to determine the extent and causes for the transverse cracking. The investigation results are summarized in Section 5 of this report.

As Table 3.1 indicates, other JPCP projects in the Metro and University Regions have also developed transverse cracking. In March 2000, a department committee of Region and central office engineers conducted field reviews of recently constructed JPCP and JRCP projects. Later

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[9] The primary objective was to determine if an open-graded base was a cause of spalling and faulting of transverse cracks with JRCP. The project was titled "Investigation of Transverse Cracking on Michigan PCC Pavements over Open-Graded Drainage Course". Principal researchers were Dr. Will Hansen, University of Michigan and Dr. Tom Van Dam, Michigan Technological University. The report is dated November 30, 1998.

in May, the Department made a similar field review of the cracked JPCP projects with MCPA representatives. The findings of both reviews were reported to EOC, which led to this study.

3.3 Network Condition Summary

Of the 15 JPCP projects constructed through 2000, five[10] are cracked in some manner. Low severity, mid-panel transverse cracks are the norm. A typical mid-panel transverse crack is shown in Figure 3.30. Spalling exists only along cracks and is highly sporadic among projects in extent and severity. Very limited crack faulting exists. The pavement joints, regardless of the slab's level of cracking, exhibit excellent condition.

Figure 3.30 Typical Mid-Panel Transverse Crack

Except for the five projects with known cracking, PMS distress data (1999 latest year) were used to evaluate the remaining JPCP projects. Excluding the I-94 project near Watervliet, the percentage of cracked slabs for all other projects combined is less than 1.0 percent. Those projects total 262 lane-miles. The highest project percentage, excluding Watervliet, is 6.3 percent[11] for the outer lane on EB I-96 (CS 47065). When the study began the distress index (DI) and ride quality index (RQI) were well within acceptable levels[12] based on 1999 PMS data.

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[10] As this study was concluding, a sixth project had developed similar mid-slab cracking. In August 2002 the Coloma TSC reported cracking on EB I-94 (CS 11018-80023) in Van Buren County. The cracking on EB I-94 was investigated as part of the slab-base (loss of support) research study.

[11] In October 2001 another survey was performed that found 18 percent of the slabs in the outside right lane with full-width cracks and an additional 6 percent with partial width cracks.

[12] In 2002 during the study and the preparation of this report, spot visual surveys were made of I-94 (CS 11018) and EB I-96 (CS 47065). The percentage of cracked slabs and associated distress appear to be accelerating. Consequently, the Southwest Region has programmed (JN 60466) EB I-94 for reconstruction in 2007.

Of the total 282 lane-miles of JPCP, the vast majority (90 percent) has only been in-service since 1997, which is too short a time period to judge long-term performance. Of the post 1997 group only the I-75 (CS 82194) project has exhibited some cracking. In contrast, the four projects constructed prior to1998 have developed some cracking and became the focus of this study. Although the cracking on the I-75 (CS 82194) project is minimal (see Section 4.3), its' condition is still perplexing because the project specified the P1-mod concrete mixture and included a warranty with a materials/workmanship provision.

4.0 FINDINGS FROM FIELD INVESTIGATION

The four projects investigated, as part of this research study, are identified with an asterisk in Table 3.0 and Table 3.1. The primary purpose of this study was to determine an accurate accounting of the visible cracking for each project and attempt to identify the cause of the cracking regarding any obvious relationship to design, material, or construction practices. It was not the intent to isolate the "root" causes for the cracking, which requires a more lengthy and extensive forensic study. Essentially, the findings[13] from this study were to determine the Department's comfort level with using JPCP on future projects and address needed research or specification revisions.

4.1 I-96/I-275 Connecting Ramp, CS 63191 - 1995
WB I-96/NB I-275 to WB I-96 - Novi - 2.4 Miles

This reconstruction project was the first to use JPCP after EOC approved its reuse. The old pavement was a severely distress CRC pavement on a dense-graded aggregate base. The new pavement consists of two lanes with bituminous shoulders. The field investigation occurred on August 19, 2000, and consisted of a distress survey, coring, and base/subgrade verification. The Appendix contains a summary field report prepared by John Boimah, Metro Region.

Condition Summary

The ramp has thirteen, transversely cracked slabs that are clustered together at a sag (cut section) of a vertical curve (± sta 1646). The respective crack locations for each slab are shown on the field report's distress survey in the Appendix. Most cracks are spalled for more than 3 ft. of their length. The cracks are full-width, full-depth and confined to the right (driving) lane. The portion of the right lane that has cracked is approximately ¾ in. lower at centerline than the adjacent passing lane.

The Inspector Daily Report (IDR) reported that this section of cracked pavement was paved on December 3, 1995. The IDR indicated the air temperature was near freezing and the concrete temperature stayed below 60°F. To compensate for the low air temperature, the cement content was increased to eight sacks and heated sand was used to benefit the hydration process.

Cores and hand borings indicate the pavement cross section was constructed per plan. Core compressive strength is very adequate (5470 psi and 4760 psi), but the subbase does not meet the

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[13] As the investigation progressed, the Pavement Committee and EOC were given numerous briefings about key findings, which reduced the urgency to publish this report. Also, the primary author of this report, David Smiley, retired from state service in November 2002. The report was later written by Smiley.

specification for class II due to a high LBW. The subgrade soil is clay. Cores taken over cracks indicate the crack formed full-depth, passing through the coarse carbonate aggregate with some expected wander. The field review determined that deflection measurements would not be productive.

Conclusion

Assuming the lane ties were installed correctly and thus can be eliminated as a cause, the ¾ in. difference in elevation between the lanes is an important clue. It's highly probable that the subbase was frozen to some depth just prior to paving. Later, as thawing progressed top-down, the base/subbase would consolidate allowing the slab to settle under traffic loading. Slab cracking likely resulted from the high tensile stresses from the high deflections during this unstable time period.

In addition, the added cement would have greatly increased concrete temperatures during hydration/set, which probably resulted in high thermal stresses from nighttime surface cooling. Thus, these transverse cracks are uncharacteristic of JPCP designs and most likely resulted from isolated factors associated with cold weather construction.

4.2 EB US-12 CS 82061 - 1997
Railroad Structure Easterly to Howe Street - City of Wayne - 1.2 Miles

EB US-12 is a separate one-way roadway varying from two to four lanes with curb and gutter. There are numerous commercial drive openings. Slab length is variable (max. 20 ft.). Numerous drainage structures exist among the lanes without provision for isolation joints during pavement expansion. The Metro Region conducted the field distress survey on July 13, 2000. A copy is in the Appendix. Dr. Hansen and Steve Minton, Metro Region Soils Engineer, performed the site review on August 7, 2000. No coring or deflection testing were deemed necessary from the conclusions derived from that review.

Condition Summary

Both transverse and longitudinal cracking is present with low severity spalling. All traffic lanes exhibit cracking. In many cases the crack starts or terminates at a drainage structure, or a drive opening. Most of the cracked slabs are concentrated near a distribution facility for steel (roll) sheeting east of the railroad structure. Several slabs in that area have multiple cracks with varying orientation patterns, which is characteristic of inadequate support and/or excessive loading.

Conclusion

The cracking pattern is indicative of inadequate pavement joint layout and drainage structures placed without joint isolation. Heavily loaded trucks during turning movements from driveways are likely the cause of the few remaining multi-cracked slabs. The field investigators agreed that the JPCP structural design was not an underlying cause. Thus, further investigation by coring and deflection testing was not warranted.

4.3 I-75 CS 82194 - 1998
Fort Street Northerly to West Grand Blvd. - Detroit - 2.2 Miles

I-75 is mostly a depressed, eight lane freeway with a concrete median barrier. It was the first JPCP freeway reconstruction project to use the P1-mod concrete mixture specification combined with a five-year pavement (M/W) warranty. The JPCP replaced a severely distressed, 1960s era, CRC pavement with a bituminous overlay. The project was bid under an accelerated A+B construction schedule, with a completion incentive/disincentive payment clause. It was subsequently completed well within the allotted time. Paving occurred between May 18 and July 31. The northbound pavement was paved first, then the southbound pavement.

Condition Summary

The Metro Region conducted a visual distress survey of both directions (from outside shoulder), but only the outside (right) two driving lanes, on July 19, 2000. The findings showed the southbound pavement had one slab with a transverse crack (TC) and two locations with longitudinal cracking (LC). The TC had very minor spalling, while the LC was spalled between 50-100 percent of its length. The northbound pavement had four locations with TCs and five locations with LCs. Two TCs had low severity spalling, while the four LCs are spalled for almost their entire length. Cracking was equally distributed over both surveyed lanes, but somewhat clustered in two general areas. The appendix contains the field summary report prepared by Steve Minton.

Findings From the Site Investigation

A 460 ft. length (sta 4+226 to 4+366) of the northbound pavement (two outside lanes) just north of the Green Avenue overpass was selected for investigation by coring and deflection testing on August 31, 2000. A detailed distress map of the two lanes with the core locations identified is included in the field report. Each end of the 460 ft. length exhibited a different cracking pattern, so each area was selected for coring and follow up auger borings to verify the base/subbase and subgrade.

Area #1 (south end) exhibited transverse cracking across both lanes for two consecutive slabs. One mid-panel TC had a branch corner crack extending across both lanes. The borings indicated the OGDC was 1-2 in. thicker than the 4 in. plan, the sand subbase (Class IIA) was also much thicker from variable undercutting, and the subgrade contained very soft clay strata. The Class IIA sand was tested and met specification. Concrete compressive strength is okay. Standard Penetration Testing found the subgrade support to be adequate.

Area #2 (north end) has only a longitudinal crack in the outside right lane for three consecutive slabs. One end began at centerline with the other at a transverse joint. A core (TH#6) through the crack showed the coarse limestone aggregate (Aggregate Source No. 75-005) was freshly fractured. The boring through the core hole in that lane found 6 in. of OGDC placed directly over a geotextile separator, but directly on a very soft clay subgrade with no subbase material present. The subbase thickness was 13-16 in. under the adjacent (left) lane, while the OGDC thickness varied from 2-14 in. The Class IIA sand was tested and met specification. Concrete compressive strength exceeded specification.

Deflection testing and profile measurements were made over the entire 460 ft. length. In the morning there was a noticeable slab "rocking"[14] sensation at the TCs, which would indicate a loss of slab-base contact near the crack. The "rocking" sensation also was felt across a joint, but was much less apparent where there was no slab cracking. The "rocking" sensation gradually went away during the afternoon as the pavement surface warmed. The previous nighttime temperature was in the 70s following daytime highs in the high 80s. Even without a significant air temperature change, the deflection testing still indicated that early morning slab curling[15] was present, as depicted in Figure 4.30. Deflection values are higher at the joints because of the upward curled effect. The curling gradually went away as the surface temperature increased during the afternoon and the deflections reduced in magnitude. Figure 4.31 compares the variation in joint LTE for morning and afternoon. The afternoon LTEs improved significantly, as the joint deflections (d0) reduced in magnitude.

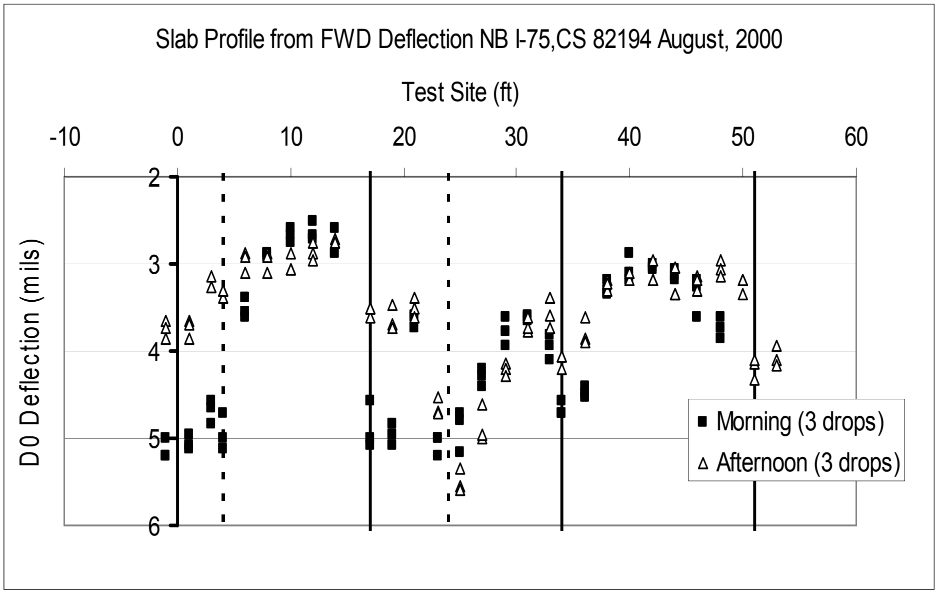

Figure 4.30 Effect of Time of Day on Slab Profile From FWD Deflection Along Outer Wheel Path

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[14] Slab "rocking" is noticeable when standing on the tied concrete shoulder near the pavement edge with one's legs straddling the transverse joint. As a truck crosses the joint, one can sense a sequential jolt in each foot/leg.

[15] In this context, the term "curling" depicts a condition where the ends (more so at the corners) of a slab are curled upward putting the slab in a concave position. Curling results from contrasts in length changes between the top and bottom of the slab depending upon varying changes in thermal gradients. Curling for JPCP slabs can also cycle through a convex position.

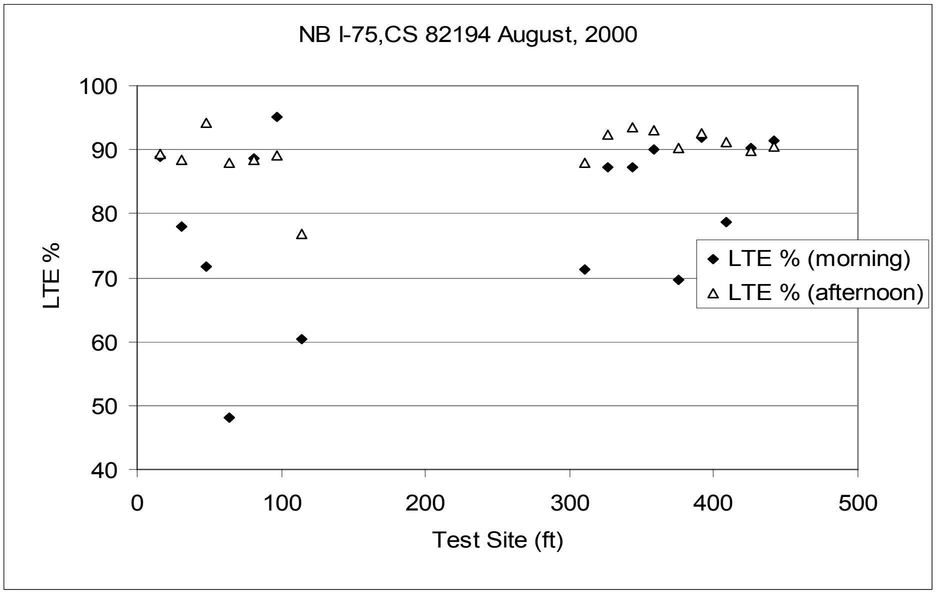

Figure 4.31 Effect of Time of Day on LTE at Joints

Figure 4.32 compares the relative joint deflections (d0) for the morning and afternoon for both area 1 and area 2. Some curling was present, but joint load transfer (BJT) remained very adequate (80-90+% LTE – PM hours) with typical joint deflection values of 3.5 to 5.0 mils at d0 (9000 lb). The afternoon values are much more consistent than the morning scatter as the effects from curling are reduced. Later analysis of the FWD data found no indication that a permanent built-in curling condition is present.

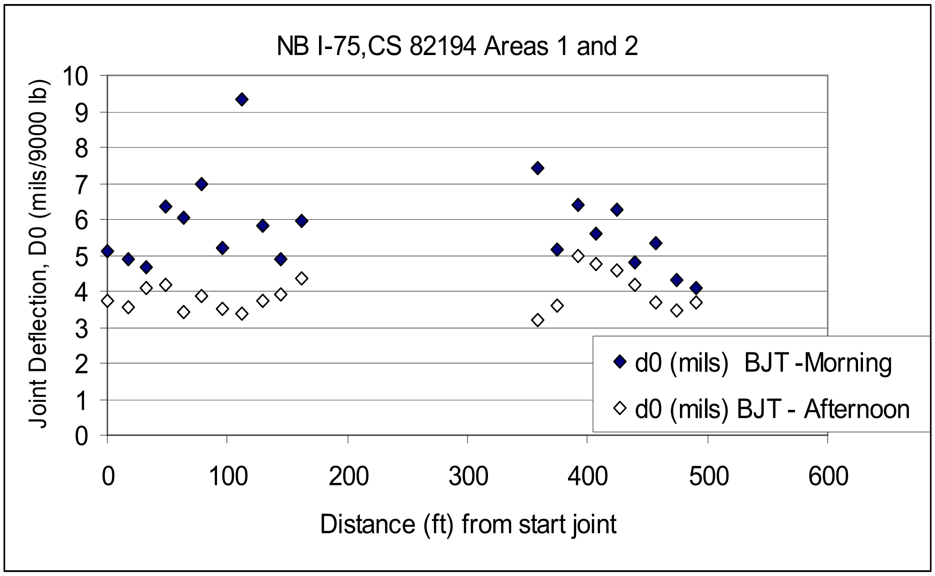

Figure 4.32 Comparison of Joint Deflections for Areas 1 and 2

IDRs reported that the concrete mix design was modified four times. The construction records verified that concrete compressive strength at 28 days (average 5270 psi.) was well above the specification.

Conclusion

The Metro Region's field report (see Appendix) concludes that the transverse cracking in area #1 is not related to any sub-surface condition. Thus, a highly probable cause is the temporary loss of support at the joints, which is explained in more depth in Section 6. For area #2 the field report confidently concludes that the longitudinal cracking resulted from the very low underlying support and likely differential winter frost heaving and subsequent spring resettlement. This conclusion is very plausible. In short, the cracking is primarily related to poor construction workmanship and not inherent with the JPCP project design parameters.

4.4 I-96 CS 47065 – 1996 to 1998
Chilson Road Easterly to Dorr Road - Livingston County - 3.6 Miles

This JPCP reconstruction project is a rural six-lane freeway. It replaced a four lane JRCP constructed in the early 1960s. The new third lane was added on the median side. Two-way traffic (two lanes per direction) was maintained on the opposite roadway, while construction occurred in each direction. Construction began in the fall of 1996 to add the new median lane along existing westbound to prepare the roadway for two-way traffic. The eastbound roadway was reconstructed in 1997. In 1998 the two remaining westbound lanes were reconstructed. The project completion date was determined by a set calendar date, which was met.

Condition Summary

On August 7, 2000, the Pavement Structures Group conducted a visual crack survey of the entire project limits from the outside shoulder. On westbound I-96 only two, mid-panel transverse cracks, near the railroad overpass, were detected. However, they both crossed all three lanes. On eastbound I-96, ninety slabs with mid-panel transverse cracks were detected from 1181 surveyed. Table 4.40 is a summary of the crack locations by lane and type.

Table 4.40 EB I-96 Crack Survey (August 2000)

Lane	Partial-Width Crack	Full-Width Crack
Outside	27	47
Center	3	10
Median	0	3
Totals	30	60

The eastbound cracked slabs are generally clustered together in five distinct areas. Spalling is mostly limited to a few open cracks. The partial-width cracks indicate that top-down cracking is likely occurring, which is discussed in more detail in section six of this report. No faulting was found, so the open-graded base is probably not contributing to the cracking.

Site Investigation (September 2000)

A 1000 ft. length of the outer right lane of EB I-96, about a mile west of Dorr Road near a median crossover, was investigated on September 27. Dr. Hansen and several members from the Pavement Structures Group were involved. The site began at Sta 782+00 and ended at Sta 792+00. The cracking is concentrated over the west half of the site, while the east half is relatively cracked free and became the control section. This general area of EB I-96 has the largest concentration of TCs. Both partial-width and full-width TCs are represented. A detailed crack map of the site is in the appendix. Besides the cracking condition, this site was also selected because deflection testing (not related to this study) had been performed in the vicinity of this location in December 1997 during construction (described later).

As with the I-75 project, group members felt a pronounced slab "rocking" sensation during the morning hours from passing trucks in the adjacent middle lane, which stopped in the afternoon. Slab curling was likely present, which was later confirmed by analysis of joint deflection values (see following discussion). The previous nighttime temperatures were cool. Air temperatures reached the 60s during testing with sunshine all day.

Coring and auger borings found the cross section to be generally constructed per plan. The six inch cores taken through full-depth cracks showed the slag coarse aggregate, as expected, to be fractured with very little protruding texture. The cores also confirmed the visual existence of top-down, partial-depth cracks that had initiated at either pavement edge. Three cores were tested for compressive strength and found to be satisfactory (average 4365 psi). The average elastic modulus was 3.5 million psi (low for slag concrete) and the split tensile strength was 571 psi. The OGDC aggregate was crushed limestone[16].

The underdrains under each shoulder were inspected with a video camera. No problems were observed. The underdrain outlets were also inspected and found to be functional.

The purpose of the deflection testing was to determine joint load transfer efficiencies (LTE) to quantify the existence of curling and any corresponding loss of slab support at the joints. LTEs were measured on both sides of the joint (location of load plate), where "before the joint" is noted, as BJT, and "after the joint", as AJT, as related to traffic flow. For this site, deflection measurements (9000 lb. load at d0) were made longitudinally at two-foot increments in the outer wheel path to determine the extent of any loss of slab support. A "walking stick" was also used to measure the surface profile to confirm the presence of upward curling found by the FWD.

Figures 4.40a and 4.40b compare morning versus afternoon LTEs (BJT and AJT) which are dependent upon the magnitude of upward curling that is present. There is considerable range in BJT and AJT afternoon values, whereas, morning BJT values are consistently less than 50 percent. As expected, the afternoon values do increase from surface heating. The AJT values are generally higher than BJT because of the permanent "slippage"[17] that has occurred along the relief crack. Figure 4.41 compares the change in maximum joint deflection at d0 from morning to afternoon. Morning deflections were about 6-13 mils, while in the afternoon they were a much better 4-6 mils. The higher joint deflections in the morning indicate a likely loss of slab support exists, while slab contact resumes with the base in the afternoon with an increase in surface temperature (negative gradient removed).

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[16] The right and middle lanes used limestone which was a mix of 6A and screenings while the median lane used limestone that was produced as 3Gmod. West of the railroad structure slag 3G mod was used under the median lane.

[17] The dowel bars have become less effective in this condition as they do not resist initial slab movement under loading.

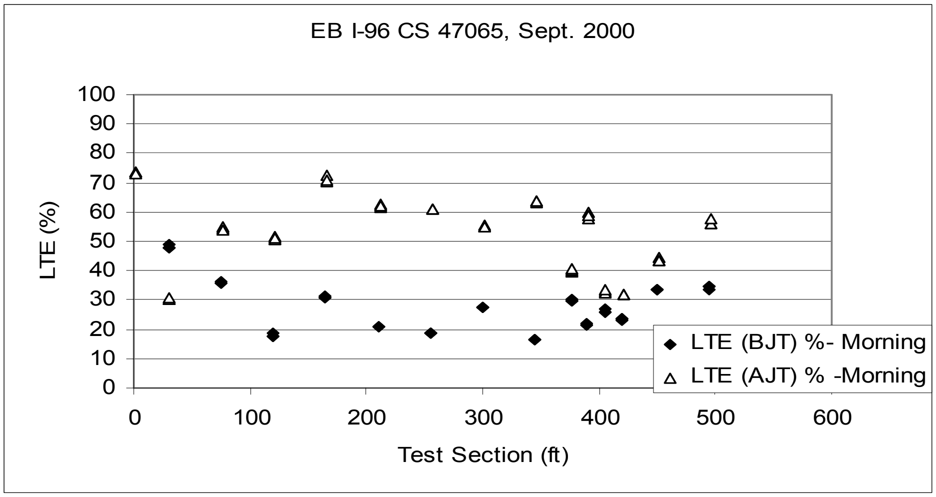

Figure 4.40a Comparison of Morning LTEs

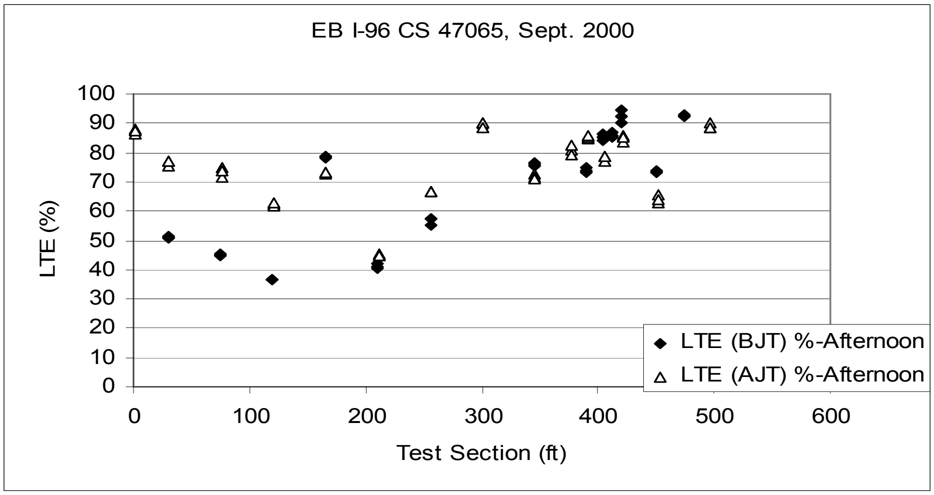

Figure 4.40b Comparison of Afternoon LTEs

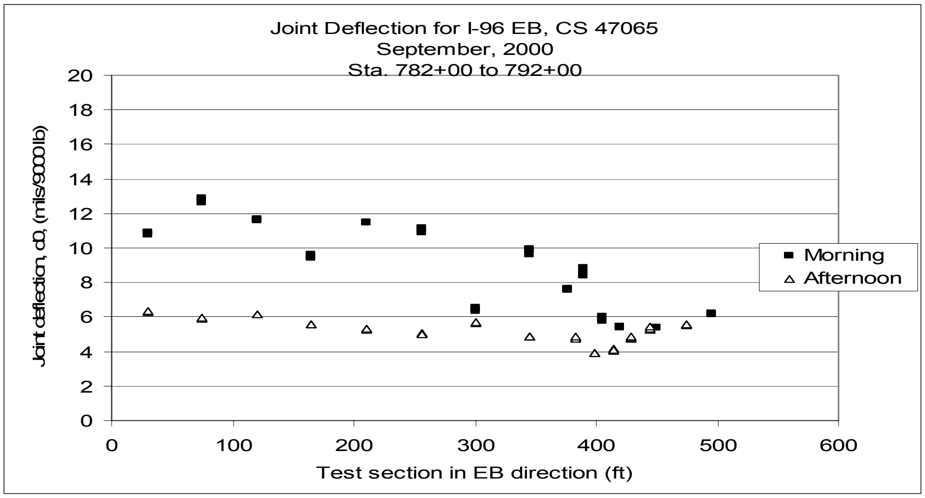

Figure 4.41 Comparison of Day Joint Deflections for EB I-96 Test Section

Deflection Testing After Paving (August 1997)
In August 1997, before the EB I-96 pavement was opened to regular traffic[18], deflection testing was performed in the outer right lane at three times during the day (morning, noon, afternoon). The 500 ft. long site (sta 728 to 733) was located directly across from the westbound rest area near the west end of the project. This testing was being done prior to this study as part of a network data gathering effort to verify joint LTEs of newly constructed pavements prior to normal traffic loading. The mid-slab cracking on EB I-94 near Watervliet was being investigated at this time. Because I-96 and I-94 both used slag coarse concrete, the I-96 data was collected to estimate a likely starting load capacity condition for EB I-94. Both pavements were constructed during hot summer temperatures, when the possibility of excessive mass shrinkage can occur. When slag concrete is used, there is probable suspicion that a wider than usual relief crack forms below the joint saw cut[19]. A wider relief crack allows higher joint deflections from decreased aggregate interlock caused by more differential slippage across the crack interface. No slab cracking of any type was observed at this site during testing.

Figure 4.42 clearly shows from joint deflections (outer wheel path) for a typical slab that concave curling exists in the morning hours, which declines significantly in magnitude (11 to 5 mils) by noon as the slab's surface temperature increased about 30 degrees. Figure 4.43 compares the high variability in LTE (BJT) values in the morning and afternoon in the outer wheel path. The majority of the LTEs are remarkably (disturbing) low (< 90 percent) for new construction which questions the effectiveness of the dowel bar installation. Infrared measurements showed the pavement's surface temperature increased to about 95°F later in the afternoon. Figure 4.44 depicts the same joint deflection values versus pavement surface temperature suggesting when the temperature increase is sufficient for the slab to resume contact with the base, which is somewhere above 70 degrees.

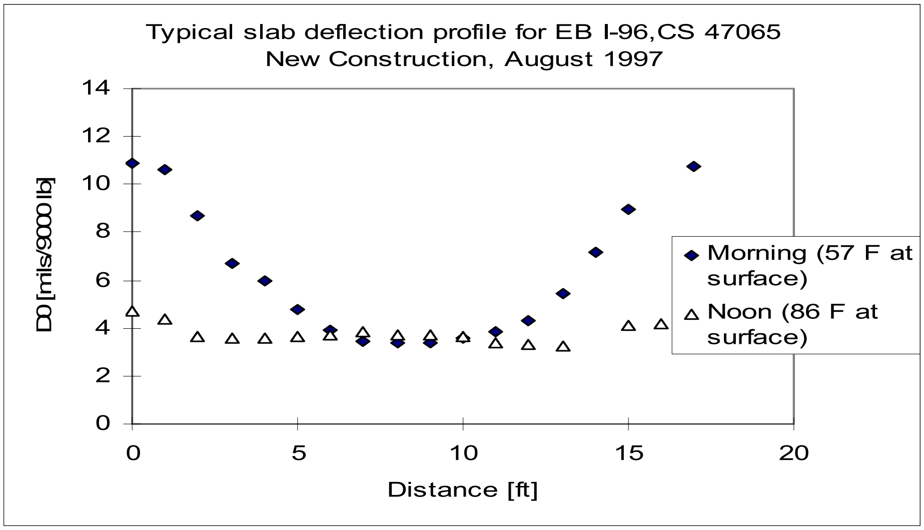

Figure 4.42 Variation in Slab Deflections Along Outer Wheel Path

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[18] However, "heavily loaded" construction vehicles had been using the pavement as a haul road.

[19] The premise depends upon the amount of mix water added at time of batching and the water content of the blast-furnace slag aggregate, which are unknown in this case. The mix design stated the absorption rate for the slag used is 3.63 percent.

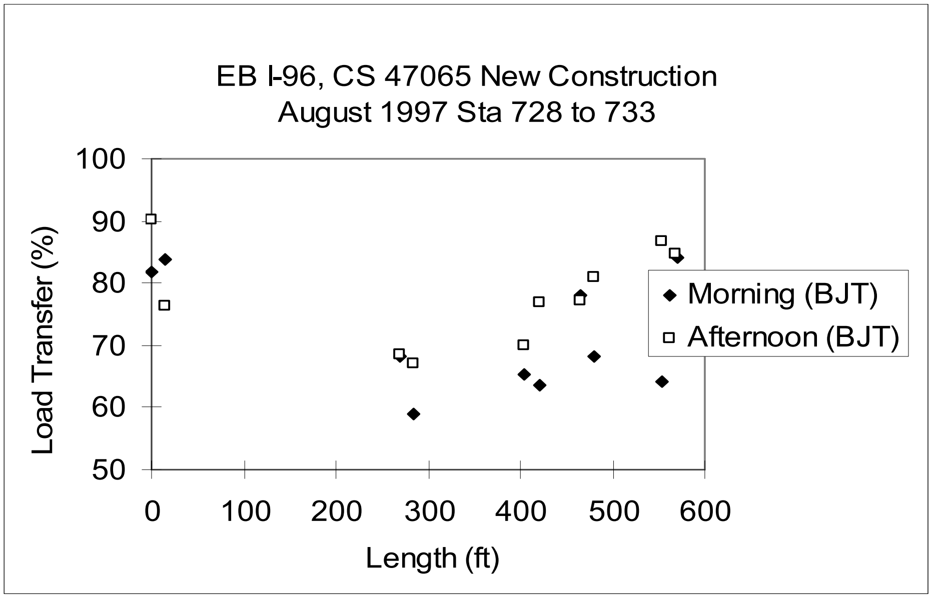

Figure 4.43 Daily Change in Joint Load Transfer Efficiency

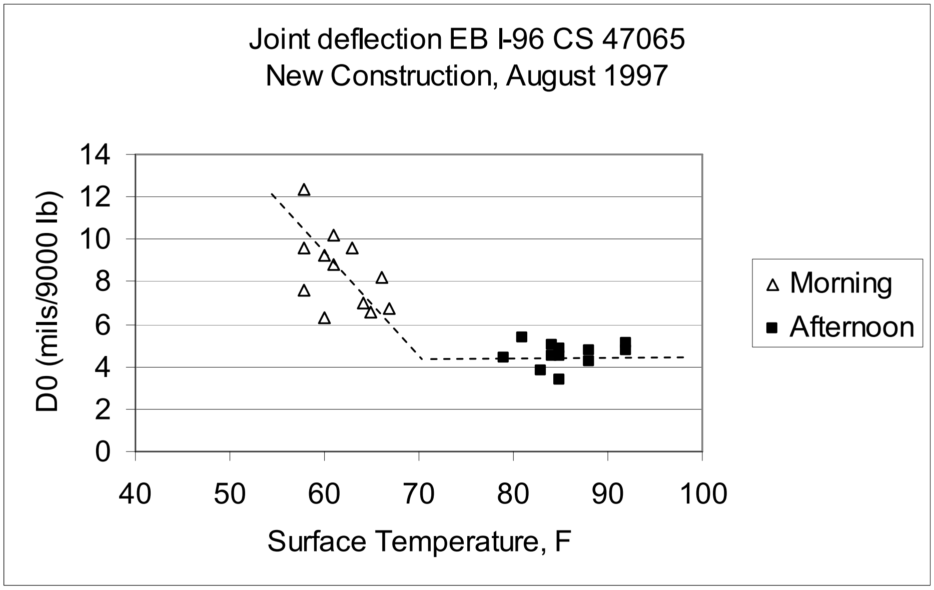

Figure 4.44 Loss of Slab Support Versus Surface Temperature

Deflection Testing After Paving (December 1997)

In mid December 1997 deflection testing was performed on EB I-96 to verify the influence of traffic exposure and temperature placement on initial LTE results. The site (sta 789 to 795) partially overlaps the September 2000 test site (sta 782 to 792). Air temperatures were in the mid 40s and the pavement surface temperature remained in the low 40s during testing. The site was selected for two reasons. One, to determine whether the short exposure to normal traffic loading had reduced the joint LTE (AJT vs. BJT) and two, to determine if any significant difference in overall LTE existed between the lanes since the right outside (truck) lane was paved during cooler fall weather, while the median lane was paved in July. The premise is that cooler air and mix temperatures should reduce the width of the relief crack (less shrinkage and contraction) and thus increase LTE.

Data analysis found BJT values lower than AJT for the right lane, while the median lane showed no disparity. However, most joint LTEs were lower for the median lane. A wider relief crack is the likely cause. Analysis also found that the right lane has experienced some permanent slippage from a loss of aggregate interlock.

Conclusion

The cause for the mid-slab cracking on EB I-96 is unlike the other three JPCP projects previously discussed. Clearly, a loss of slab support in the proximity of the joints is occurring during the morning hours, when the pavement's surface temperature is cooler than the bottom (negative temperature gradients). This factor is having a dramatic adverse result on the load transfer capability of the joint. The very noticeable slab "rocking", when one straddles a joint, as a truck passes, indicates that loss of support at the joint is present.

Extensive deflection testing has shown that upward curling is causing a temporary gap (loss of support) to form between the top of the base and the bottom of the pavement. The gap's magnitude is greatest at the joint corners. The upward curled shape can be quite pronounced, as the slab does not regain full contact with the base until near mid-slab. A greater gap (higher deflections) causes the tensile stress at the slab's mid-point to increase during truck loading, which is probably exceeding the slab's fatigue limit. Cores confirmed that mid-slab cracks are definitely originating top-down at the slab's edge, where the created stress is greatest. Upward curling combined with truck loading at the joints is undoubtedly a cause for mid slab cracking. Certain concrete properties also are contributing.

A comparison of joint deflection results at the time of construction versus three years later shows that aggregate interlock is degrading at the relief crack interface, which increases shear displacement and lowers LTE. Section 6 will further elaborate on these findings and present a plausible explanation as to how the mid slab cracks occurred and a reason for their accelerating formation. An explanation will also be offered as to why WB I-96 has only limited slab cracking at a much lower rate of formation.

5.0 I-94 INVESTIGATION NEAR WATERVLIET

Late in 1996 an investigation began to find the cause for the mid-slab cracking on the recently constructed JPCP project west of Watervliet (M-140) in Berrien County. The investigation was conducted as part of an ongoing open-graded research project (see Section 3.1). The formal investigation of I-94, as part of that research[20], was completed in 1997. As previously noted in Section 3.1, Dr. Hansen was a Co-PI for the OGDC study project and was the lead investigator for the I-94 portion. The findings from the I-94 Watervliet investigation are summarized in this report because the I-96 cracking problem is emulating over time what is occurring with I-94. The causes for the mid-slab cracking for each project appear to be similar.

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[20] The study of I-94 was added to this research which compressed the time available for the site investigations.

5.1 Construction Summary

The five mile long I-94 JPCP project (CS 11017 JN 32516A) begins at Friday Road, then extends easterly to Hennessey Road just west of M-140. The eastbound direction was paved in 1995 (August/September) and the westbound direction in 1996 (May/June). The paving contractor was Interstate Highway Construction. The pavement design parameters are summarized in Table 3.1. The pavement design for the Watervliet JPCP project utilized some atypical specifications from usual standards, which are listed below.

The eastbound pavement consists of three sections to study the performance effects of varying joint spacing. The construction limits of each section and their designation for the field investigative study were as follows:

- Section A – Regular 16 ft. joint spacing. Sta 1790+00 to 1893+00
- Section B – Illinois Hinge Joint. Sta 1709+90 to 1790+00
- Section C – Random 15 ft., 16 ft., and 17 ft. joint spacing. Sta 1630+00 to 1709+90
- Section D – Westbound pavement, entirely regular 16 ft. joint spacing.

All joints are perpendicular with centerline. Load transfer baskets were used to place the dowels.

Section B (Illinois Hinge Joint) was constructed to verify its success[21], as reported by the Illinois DOT, to mitigate post-construction cracking of long JRCP slabs. The design consists of two controlled "cracks" (hinge joints) at a 15 ft. spacing between standard 45 ft. spaced contraction joints. Thus, the slab length is considered to be 45 ft. with two hinge joints. The hinge joint is formed by the same saw cut for normal contraction joints. In lieu of dowels, the hinge joint uses 30 in. long epoxy coated #6 deformed reinforcing bars at 18 in. spacing. The bars were placed on chairs at mid-depth of the slab. The purpose of the deformed bars is to disallow slab movement at the two intermediate tied joints. The hinge saw cut reservoir is sealed with hot-poured rubber, whereas the normal contraction joints use neoprene seals.

The same open-graded base was used for the entire project. The specification was modified to require that the 3G aggregate be compacted to at least 95 percent of its maximum unit weight as established by the Michigan Cone Test. This change was intended to improve stability as compared to the standard "rolling" pattern. The 3G aggregate was a crushed limestone from source #75-005 (Port Inland), but was produced to meet a 6AA gradation, which as discussed in Section 3.1, can incur placement and paving problems. Compaction (density) was difficult to achieve. Staff from the Pavement Structures Group and the Density Support Group made several site visits to assist the TSC with project quality assurance. Acceptance test reports showed the 3G was coarsely graded, averaging 50 percent retained on the ½ in. sieve and only 5 percent passing the #8 sieve. Coarsely graded aggregate is more susceptible to segregation during placement.

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[21] The Rigid Pavement Condition Review Committee reviewed examples of the design on its' trip to the Illinois DOT. Sometime during the late 1990's the Illinois DOT stopped using the design for unknown reason.

The construction records were reviewed by the Materials Research Group to determine the as-placed concrete properties, temperature data, and paving dates/amounts. The pertinent data are listed in Tables 5.11, 5.12, and 5.13 for sections A, B, and C, respectively.

Table 5.11 EB I-94 Section A – Paving Dates, Temperatures, and Limits

Sta Limits	1893+00 to 1865+00	1865+95 to 1846+44	1846+44 to 1805+28	1805+28 to 1790+00
LFT Paved	2465 lft.	1951 lft.	4116 lft.	1528 lft.
Date	8/24/95	8/28/95	8/29/95	8/30/95
Weather	Sunny-cloudy pm	Cloudy/rain pm	Partly sunny/humid	Partly cloudy
High temp.	80	78	85	86
Low temp.	60	66	68	70
Conc. Temp (H)	88	82	88	88
Conc. Temp. (L)	82	82	82	82
Avg. Air Content	6.4%	6.8%	6.5%	6.1%
Slump (H/L)	2.25/1.75 in.	2.50/2.00 in.	2.25/1.75 in.	3.00/1.25 in.

Table 5.12 EB I-94 Section B - Paving Dates, Temperatures, and Limits

Sta Limits	1790+00 to 1766+28	1766+28 to 1747+40	1747+40 to 1709+90
LFT Paved	2372 lft.	1888 lft.	3750 lft.
Date	8/30/95	8/31/95	9/5/95
Weather	Partly cloudy	Cloudy/Windy	Sunny
High temp	86	85	87
Low temp	70	65	67
Conc. Temp (H)	88	85	86
Conc. Temp (L)	82	81	78
Avg. Air Content	6.1%	6.3%	6.8%
Slump (H/L)	3.00/1.25 in.	2.50/1.50 in.	2.50/1.25 in.

Table 5.13 EB I-94 Section C – Paving Dates, Temperatures, and Limits

Sta Limits	1709+90 to 1707+45	1707+45 to 1667+27	1667+27 to 1666+65	1666+65 to 1630+00
LFT Paved	245 lft.	4018 lft.	62 lft.	3665 lft.
Date	9/5/95	9/6/95	9/7/95	9/8/95
Weather	Sunny	Partly cloudy	Cloudy/rain	Partly sunny
High temp	87	85	70	62
Low temp	67	62	60	43
Conc. Temp (H)	86	84	*	82
Conc. Temp (L)	83	81	*	72
Avg. Air Content	6.8%	6.1%	*	6.8%
Slump (H/L)	2.50/1.25 in.	2.00/1.50 in.	*	2.50/1.25 in.

*Paver broke down, so no testing was performed

The slag coarse aggregate stockpile for the concrete mix was not kept wet (no moisture control) during batching during the entire time eastbound was paved. As such, the concrete mix was frequently stiff and difficult to place. The stockpile was occasionally wetted (sprinklers) during the paving of westbound the following year and the paving effort was improved.

5.2 Cracking Condition Summary

The Southwest Region first observed mid-panel transverse cracks on EB I-94 during the summer of 1996. In December of 1996 the Materials Research Group surveyed the entire length of the right (truck) lane of EB I-94. The eastbound pavement was opened to normal traffic on October

15, 1995. The WB pavement was finished in June and opened to normal traffic for just a few months, so only a random spot survey was made with no visible cracking detected. Tables 5.21 and 5.22 are a summary of the December survey findings for EB.

Table 5.21 December 1996 Crack Survey EB I-94

Section	Total Slabs	% Slabs Cracked	Full-Width Cracks	Partial-Width Cracks
A	644	30	106	85
B	534	29	23	294
C	500	20	31	68

Table 5.22 December 1996 Crack Survey EB I-94 – All Sections

9.5% slabs (total 1678) are cracked full-width
26% slabs are cracked partially or full-width
32% cracks are spalled
2% cracks are faulted

Figure 5.20 is a picture of a typical mid-slab, full-width crack on EB in December 1996. The partial-width cracks originated from either edge of the slab. In many cases, a single slab had more than one partial crack. The survey noted that cracking was more prevalent and clustered in areas where morning paving had occurred. Previously in June, pins had been set in three random segments of section B to monitor movement of the contraction joints and the hinge joints. December measurements found that the contraction joints were moving freely and the hinge joints were fixed, as they were designed. Pins had also been randomly set in three 200 ft. segments of WB (sta 1700 to 1767) to measure joint movement. December measurements also found those joints to be moving freely.

Figure 5.20 Typical JPCP Mid-Panel Crack

The next formal survey of both directions occurred in November 1999. During the previous three years, the Materials Research Group did occasional spot surveys to track the rate of cracking. Visible cracking (only partial-width) was first observed on WB I-94 in December 1997. In February 1999 the first full-width cracks on WB were observed. Table 5.23 is a summary of the November 1999 survey findings for each EB section.

Table 5.23 November 1999 Crack Survey EB I-94

Section	% Slabs Cracked	% Cracks Spalled	% Cracks Faulted	% Slabs > 1 Crack
A	76	29	0	1
B	27	27	3.5	0
C	72	37	4.4	1

For all three sections of eastbound combined, 58 percent of the slabs were cracked full-width, 32 percent of the cracks had spalling, and 2 percent of the cracks had faulting.

Because WB had been exhibiting much less cracking than EB, November 1999 was the first time the entire WB pavement was surveyed. Following are the results of the WB November survey.

- Total slabs with one full-width crack.................................. 140 (8.5%)
- Total slabs with more than one full-width crack 5 (0.3%)
- Total cracked slabs with spalling... 14 (10%)
- Total cracked slabs with faulting... 2 (1.2%)

The WB survey found most of the cracked slabs clustered together primarily within the eastern half of the project. During the November 1999 survey, partial-width cracks were not counted for either direction, but were readily visible.

The Materials Research Group conducted the most recent condition survey of both directions (right driving lane) in July 2002. For eastbound 74 percent of the slabs was cracked full-width. Section B continued to be the best performer of the three sections with 39 percent of slabs cracked. For westbound, 36 percent of the slabs were cracked full-width. In less than three years since the November 1999 survey, the rate of increase in cracking was 39 percent for eastbound and 76 percent for westbound. For both directions, about 90 percent of the cracks exhibited some spalling.

5.3 Site Investigation

The field site investigation of I-94, under the direction of Dr. Hansen, began in April 1997. The testing was limited to only the outer right lane. Sections A and C of EB and a WB site were investigated at that time. Testing of EB section B was not completed prior to completion of the OGDC research study. Overall, the site investigation consisted of extensive FWD testing, cross section confirmation, and material quality verification. Following is a summary of the site testing performed and the findings:

5.31 EB Section A

Section A was first to be investigated. During site testing the weather was sunny with air temperatures in the 50s. The pavement's surface temperature from infrared measurements started at 60°F and rose to the mid 70s during the day. The site was located between sta 1790+09 and 1795+08. Detailed maps were made of the visible distress condition and included locations of cores, etc. About 30 percent of the slabs within the test site had full-width cracks. Partial-width cracks, which started from the longitudinal joint, were also prevalent. Low-severity crack spalling existed. Extensive map-cracking from drying shrinkage was visible after wetting the pavement.

<u>Findings</u>

The average in-place concrete strength properties from three cores were as follows:

- Compressive..6567 psi
- Strain @ ultimate compression.................................0.00180 in/in
- Elastic Modulus ....4.6 million psi
- Split Tensile ....640 psi

The LTE of mid-slab cracks, as related to surface crack width, are shown in Figure 5.31. The results do not show an expected pattern, so other factors are affecting results. Testing was performed with the load plate (d0) on both sides of the crack. The LTEs on the leave (before) side (BCK) of a crack tend to be lower than the approach (after) side (ACK) of a crack.

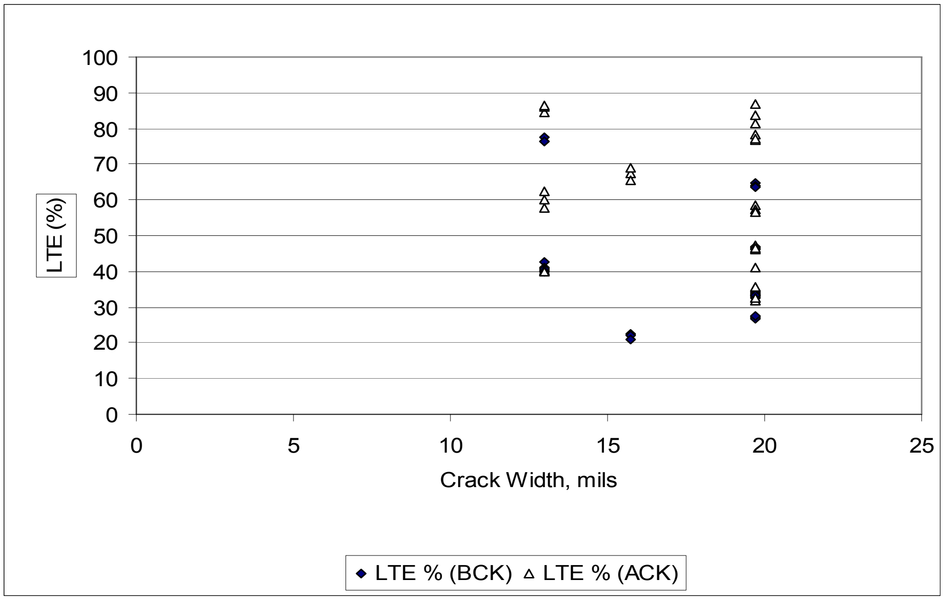


Figure 5.31 Crack Width Versus LTE – Section A EB I-94 Watervliet

The LTE of the transverse joints was measured similar to the cracks with the load plate placed on both sides of the joint. The joint LTE results are shown in Figure 5.32, which are low. The LTEs before the joint (BJT) are somewhat less than those after the joint (AJT). Data points are limited as only uncrack slabs were tested. Figure 5.33 shows LTE (BCK) for all the mid-slab cracks tested sequentially. Overall, the LTE of cracks and joints are similar.

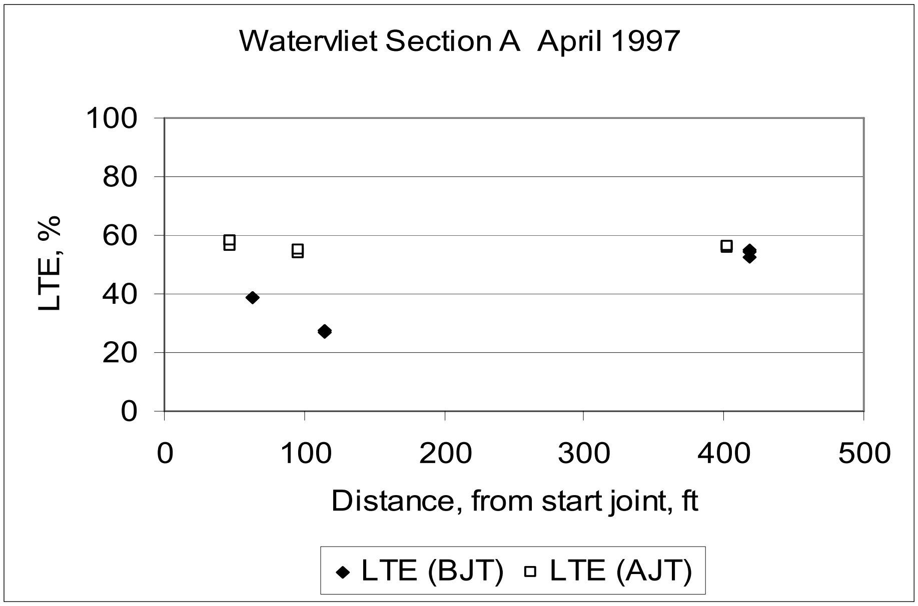


Figure 5.32 Load Transfer at Joints

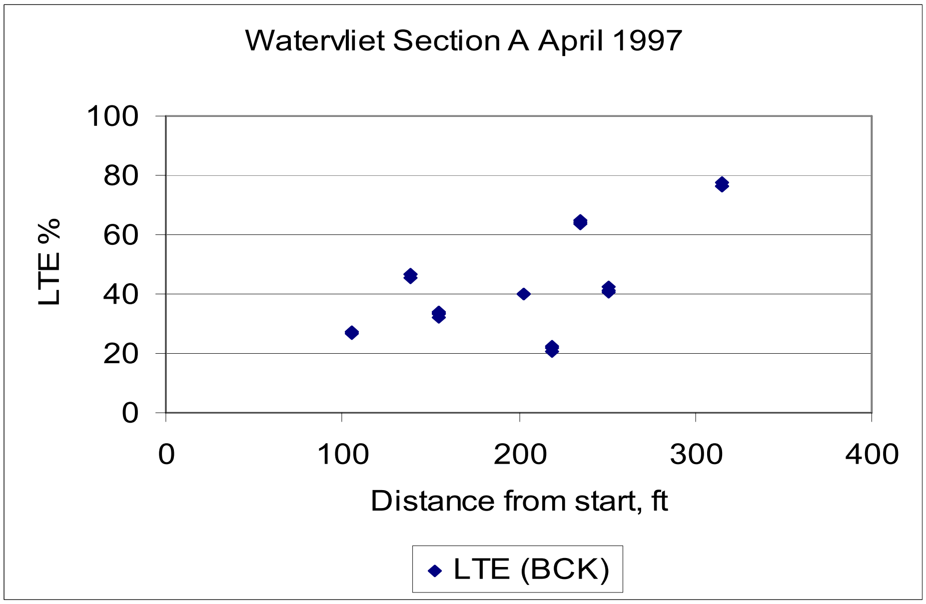


Figure 5.33 Load Transfer Efficiency of Mid-Slab Cracks

The cross section was verified by slab coring and thickness measurements made of the OGDC and subbase. The stiffness of the OGDC and subbase was determined from blow counts using a Dynamic Cone Penetrometer (DCP). Using the combined counts for the OGDC and subbase, a California Bearing Ratio (CBR) of 40-50 was estimated for the combined base/subbase. From these field measurements all pavement layers appeared to be properly constructed per plan.

5.32 EB Section C

Section C was tested about a week after Section A. The weather was partly cloudy with an air temperature of 45-50°F. The pavement's surface temperature, as measured with infrared, varied in the 60s. The site was located between sta 1682+62 and 1690+15. Detailed maps were made of the visible distress condition and included locations of cores, etc. The site selected for section C was in better overall condition than site A. Only 2 percent of the slabs had full-width cracks, but about 75 percent of the slabs exhibited partial-width cracks starting from the longitudinal joint. Only two full-width cracks exhibited spalling with minimal faulting.

Findings

The average in-place concrete strength properties from three cores were as follows:

- Compressive...7026 psi
- Strain @ ultimate compression.................................0.00180 in/in
- Elastic Modulus ....4.29 million psi
- Split Tensile...585 psi

The cross section was verified by slab coring and thickness measurements of the OGDC and subbase. The stiffness of the OGDC and subbase was determined from blow counts using the DCP. The combined CBR for the OGDC/subbase was determined to be 40-50, similar to section A. From these field measurements all pavement layers appeared to be constructed per plan.

The joint LTEs for section C are shown in Figure 5.34, which are all morning values. The BJT values are much lower than the AJT values which indicates permanent slippage at the relief crack has occurred. This indicates the dowels during initial loading have become ineffective with time.

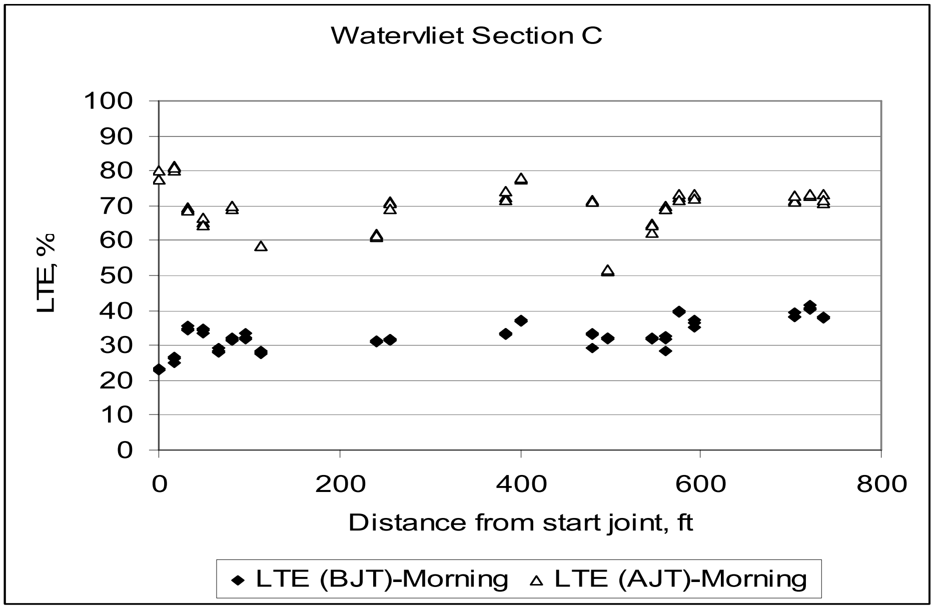


Figure 5.34 Morning Joint Load Transfer Efficiency – Section C Watervliet

5.33 WB Section D

Section D on WB I-94 was tested May 28, 1997. The weather was mostly cloudy with an air temperature of 60° to 72°F from morning (9:00am) to mid-afternoon (3:00pm). The pavement's surface temperature, as measured with infrared, ranged from 60°F to 71°F during testing. The site was located between sta 1782+98 and 1793+08. Detailed maps were made of the visible distress condition that included locations of cores, etc. Overall, the site had very little distress with only some low severity, intermittent longitudinal cracking near centerline from sta 1792+61 to 1794+50. No transverse cracking was visible.

Findings

The average in-place concrete strength properties from three cores were as follows:

- Compressive..5691 psi
- Strain @ ultimate compression.................................0.00180 in/in
- Elastic Modulus ....4.84 million psi
- Split Tensile ....556 psi

Figures 5.35 and 5.36 compare the joint LTEs for morning and afternoon, respectively. Overall, the LTEs did not significantly change from morning to afternoon. Morning BJTs averaged 30 percent, while AJTs averaged 79 percent. Afternoon BJTs averaged 41 percent and AJTs averaged 85 percent. However, there was much less variation in deflection values (d0) for the afternoon tests. AJT deflections ranged from 4-8 mils for the morning to 4-6 mils for the afternoon. BJT deflections went from 5-12 mils to 4-6 mils, respectively.

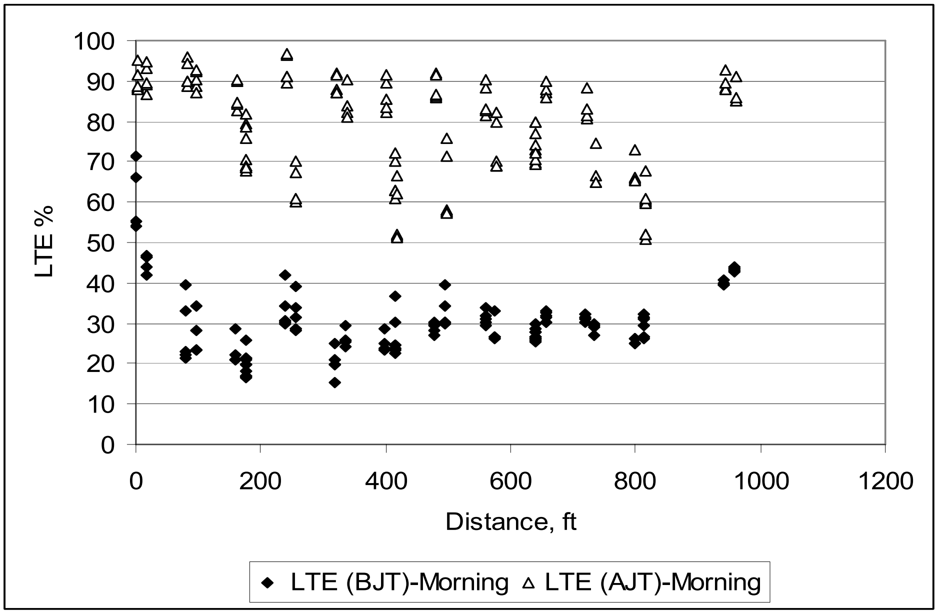


Figure 5.35 Morning Joint Load Transfer Efficiency – Section D Watervliet

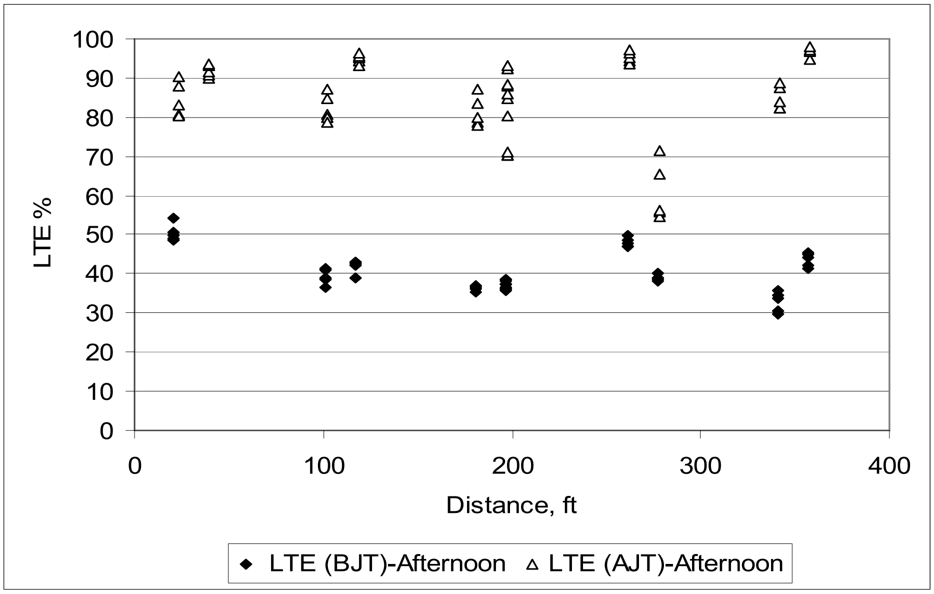


Figure 5.36 Afternoon Joint Load Transfer Efficiency – Section D Watervliet

The cross section was verified by slab coring and thickness measurements of the OGDC and subbase. The stiffness of the OGDC and subbase was estimated from blow counts using the DCP. The combined CBR for the OGDC/subbase ranged from 30-65, which is a wider distribution than with section A or C. From field measurements all pavement layers appeared to be constructed per plan.

6.0 ANALYSIS AND DISCUSSION

This section will further elaborate on the analysis and similarities of the site investigations of the I-94 (Watervliet) and I-96 (Livingston County) JPCP projects. Also, the impacts on project service life and the feasibility of using preventive maintenance to correct the cause for the cracking and/or extend service life will be discussed. Finally, a plausible explanation for the mid-slab cracking will be presented.

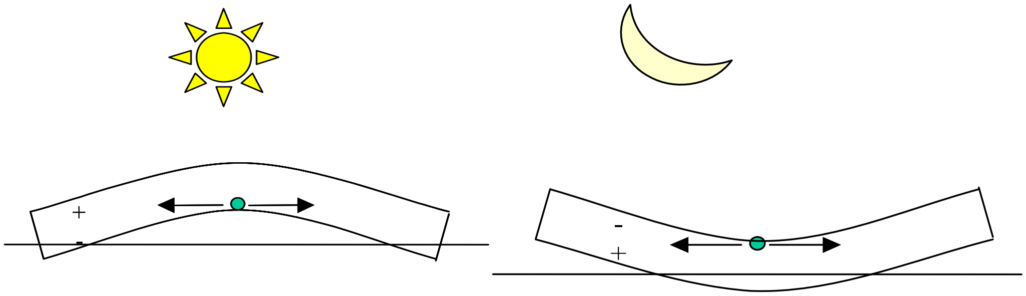
6.1 Recap Information

The preceding discussion of the JPCP project site investigations referred to the occurrence of slab curling, built-in curling and loss of slab support. Before proceeding, each term is further explained to support the discussion that follows.

Slab Curling

Curling describes the natural change in a concrete slab's shape in respect to its surface profile (slab curvature) from heating and/or cooling changes. The amount of curling present at a particular time is dependent upon the temperature gradients that exist throughout the slab's depth. These gradients change whenever the slab is heated and cooled from both internal and external environmental factors. The temperature changes are non-linear with depth.

When the slab's surface is cooler than the bottom, a negative temperature differential exists, and the slab will tend to curl upward to form a concave shape in respect to traffic. When concave, the slab's corners will rise the highest in respect to the mid slab area. When the slab's surface is warmer than the bottom, a positive temperature differential exists, and the slab will tend to curl downward to form a convex shape. In this position, the mid-portion of the slab is the highest relative to the corners. In both cases, a loss of support (gap/void space) can occur below the lifted area when the expansion/contraction force too lift the slab exceeds the resisting force generated from the slab's weight. A depiction of upward and downward slab curling is shown in Figures 6.10a and 6.10b.



(a) Downward Curling (b) Upward Curling

Figure 6.10 a & b Curling of slab due to temperature gradient (NHI, M-E Design Course)

<u>"Built-in" Construction Curling</u>

Built-in curling refers to the slab's ability to remain upward curled semi-permanently. The factors to produce built-in curling occur during paving at the time of concrete set. At the time of set the concrete slab is in a plastic state and in complete contact with the base without any curvature. Set normally occurs when the temperature differential is negative, normally at night, after the cooler air temperature has reduced the slab's higher daytime surface temperature. However, if set occurs during daylight when a positive temperature differential exists, upward curling will be more prevalent. Corner uplift wants to occur whenever the temperature gradient changes toward being negative. The conditions to produce built-in curling are prevalent during summer construction when slabs paved in the morning receive additional radiant heating from sunlight. This induces a positive temperature differential. Higher concrete mix temperatures can accentuate a positive temperature differential at time of set. In 1950 a MDOT study by Rhodes documented this phenomenon. Figure 6.11 is taken from that study which clearly shows how early morning paving affects slab temperature gradients.

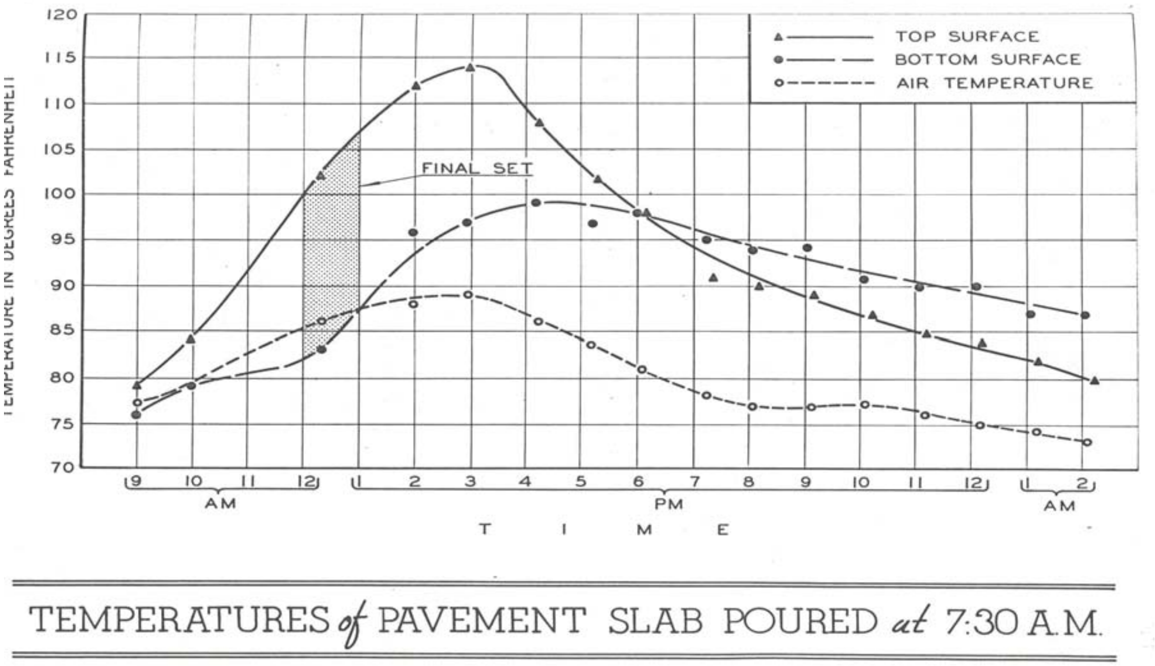


Figure
6.11 Temperature Gradients From Early Morning Paving
Reference: "Curing Concrete Pavements with Membranes", C.C. Rhodes,
Research Project 42 B-14 (2), Report No.145, August 8, 1950.

In summary, there is a tendency for upward curling to occur whenever the positive temperature differential at the time of set tends toward being negative. Thus, built-in curl increases the time when the pavement slab experiences a condition of upward curling and becomes susceptible to cracking, if the joint area loses underlying support.

Loss of Slab Support

For purposes of this study, the term "loss of support" refers to a gap or void that exists between the bottom of the slab and the top of the base layer. Generally, there are three causes for loss of support:

- Erosion of material at the base/slab interface.
- Consolidation or settlement of the supporting base/subbase layers.
- Curling from changing temperature gradients through the slab and/or warping from absorbed moisture gradients (loss or gain of water per slab depth).

This research investigation concentrated on the results from curling from temperature changes, but as previously noted in section four, localized consolidation or settlement of the base/subbase layer was found to be a cause for loss of slab support on two JPCP projects with cracking. The resulting gap from upward curling is greatest at the slab's corners and along the transverse joints. For shorter JPCP slabs it decreases in magnitude toward the slab mid area. Analysis of deflection values found that contact with the base normally resumes about 5-7 ft. from the joint depending upon the slab length. It is important to emphasize that some temporary loss of support from curling is normal and unavoidable. As discussed later, the increase in stress solely from normal curling is not sufficient to crack the slab.

Some measurable loss of support at the joint occurs when the curling stress from surface contraction exceeds the counter force from the slab's weight that tries to keep the slab in contact with the base. Several factors can contribute to the magnitude of the gap. The combined effect of the thermal and moisture gradients is the greatest contributor. Other factors are slab restraint at the joint, base stiffness, joint spacing, and the concrete's CTE. A stiffer base (higher "k" factor) will provide higher resistance, but allow a greater gap plus area of loss to occur. The friction factor for the base also influences the magnitude for loss of support. More base friction restrains the slab's movement, which determines its remaining contact area after curling initiates.

6.2 Further Discussion of I-96 and I-94 JPCP Projects

I-96 and I-94 are the most significant and revealing projects resulting from this research investigation. They have several similarities including their cross section, concrete mixture, and the same paving contractor. I-94 has about 15 percent more commercial traffic than does I-96; distributed over two lanes versus three lanes for I-96. Both projects have a similar distinct contrast in the extent and severity of cracking between travel directions, which were constructed during separate years.

These projects have the highest deflections with a significant loss of support at the transverse joints. Probably the most dramatic finding is the significant difference and variability in LTE when measured before the joint (BJT) versus after the joint (AJT). BJT measurements are much lower than AJT measurements. Sometimes by nearly 50 percent or more, which Figure 6.21 shows for EB I-96 (September 2000 site). The most probable explanation for this difference is the rapid slippage that has occurred as the approach side of the relief crack shears (depresses) against the leave side of the crack. As the coarse aggregate particles breakdown (loss of interlock), LTE decreases, which forces the dowels to solely transfer the load. Although not confirmed, some crushing of the concrete in contact with the dowel has probably occurred, which further reduces load transfer capability and increases deflection magnitudes. In addition, the width of the relief crack after thermal contraction will directly determine the endurance level of the initial aggregate interlock capacity. Other recent research[22] has shown that concrete using blast-furnace slag coarse aggregate is more susceptible to early LTE loss from wider crack widths. Without adequate base support, the joint dowels quickly become ineffective from repetitive heavy truck loading, as they cannot solely provide load transfer.

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[22] University of Michigan PRCE project – "Transverse Crack Propagation of JPCP as Related to PCC Toughness", Principal Investigator was Dr. Will Hansen, Report No. RC-1404, August 2001.

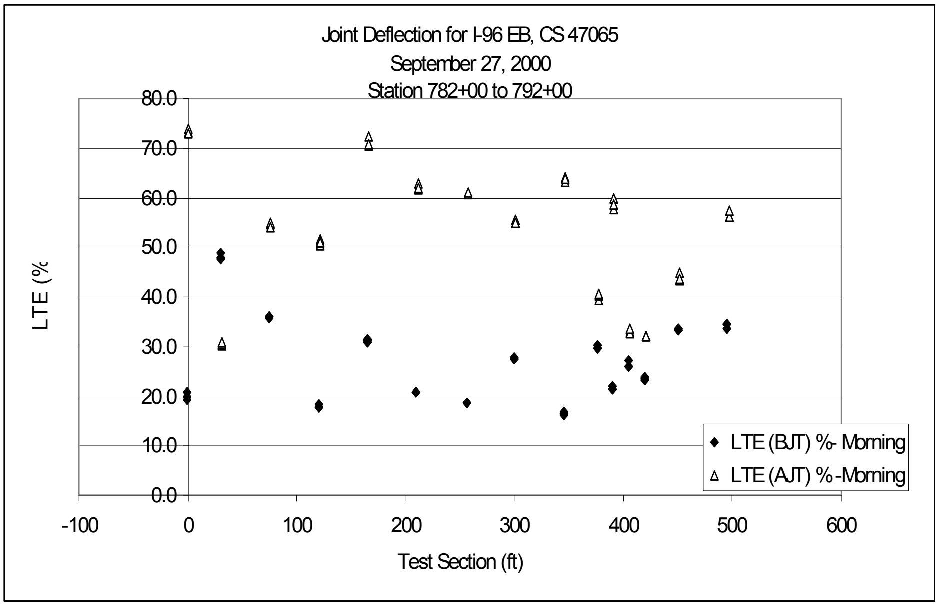

Figure 6.21 Typical Load Transfer Results for I-96 JPCP

Fortunately in October 1995 shortly after EB I-94 was paved, but not yet open to regular traffic, section B and a portion (sta 1790+00 to 1822+40) of section A were tested to determine the initial load transfer of the joints with emphasis on the experimental hinge joints. Figure 6.22 is a compilation of the LTEs (BJT) that gives a general comparison of morning values with evening. This testing was done in a continuous manner throughout the day, so each section was tested at a different time of day. Section B was morning and section A was evening.

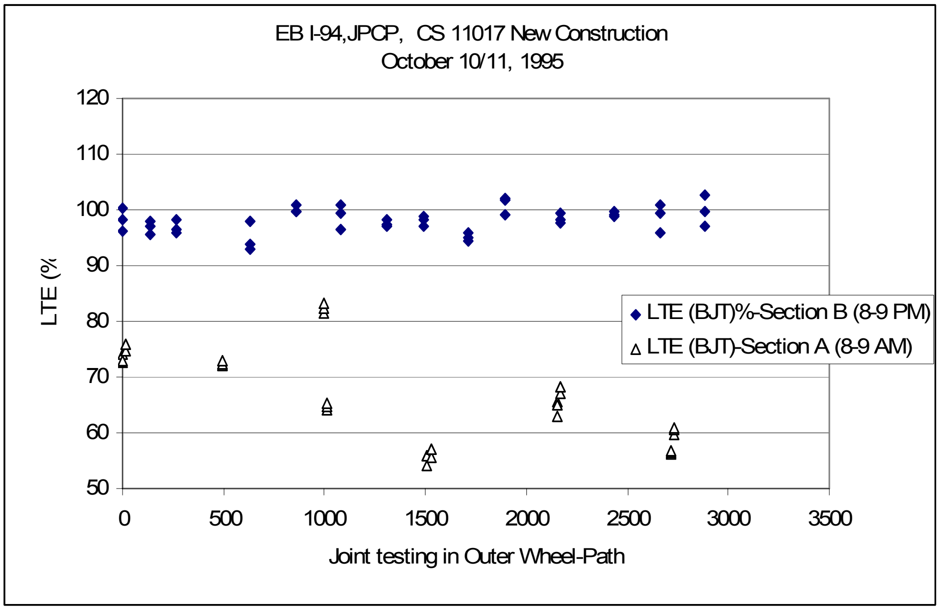

Figure 6.22 Load Transfer Efficiency for Sections A and B, EB I-94 Watervliet

Three drop loads were made at each location to determine if a loss of support was present. In the morning some loss (gap) was found after night cooling. From infrared measurements, the pavement's surface temperature changed from about 45°F in the morning to 60°F in the evening.

Unfortunately, section B was not tested during the 1997 site investigation. However, when the original LTEs for section A are compared to the 1997 test site values (Figure 6.23), overall LTE has likely reduced. Since the comparison is between BJT and AJT values, which have shown to differ for reasons previously given, whether a significant reduction has occurred is not certain. Still, a change, as shown, is dramatic for a two-year old pavement. A rapid reduction in LTE helps to explain why the cracking on EB I-94 is accelerating geometrically, both in extent and severity.

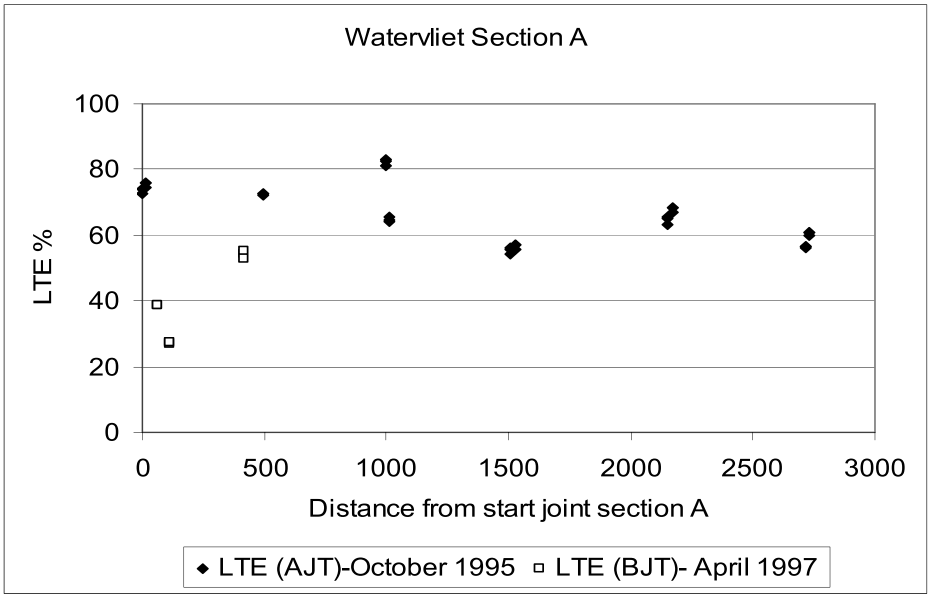

Figure 6.23 LTE After Two Years From Construction – I 94 Watervliet

During the September 2001 testing for EB I-96, the temperature gradient of a slab was measured using thermistor probes in drilled holes filled with oil. The slab's temperature was measured in ten minutes intervals at 1 in., 3 in., 5 in., and 9 in. depths during the morning, afternoon and evening. Figure 6.24 shows the representative gradients that occurred through the day, which are not linear with depth.

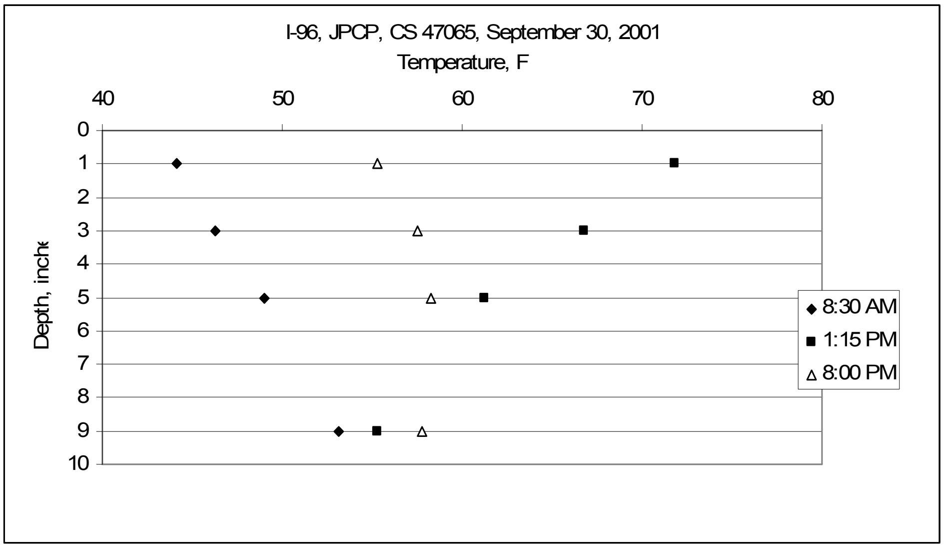

Figure 6.24 Cross Section Temperature Gradient From Morning to Evening

The deflection and temperature data from the September 2001 test site were used to determine the positive temperature differential that would need to exist to assure the slab would be in full contact with the base. A linear regression of the corner deflection values versus temperature difference between top and bottom indicates an approximate 12°F positive differential is needed for the slab to be in full contact with the base. The required temperature differential will change day to day depending upon previous temperature cycles.

The daily change in deflection for three consecutive slabs is shown in Figures 6.25a, b, and c. The figures reveal that a slab is subjected to upward curling for most of the day and a corresponding loss of joint support. As the temperature differential (ΔT) increases, the magnitude of the loss (gap) at the joint becomes greater. This analysis supports the conclusion from the previous 1997 testing that suspected a built-in curling condition formed during construction for much of EB I-96.

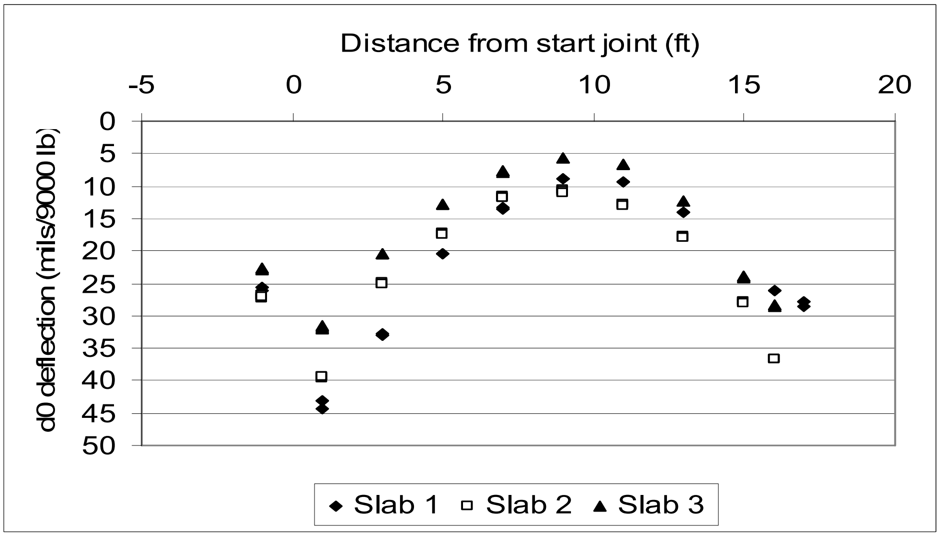

Figure 6.25a Morning Deflection Profile for EB I-96

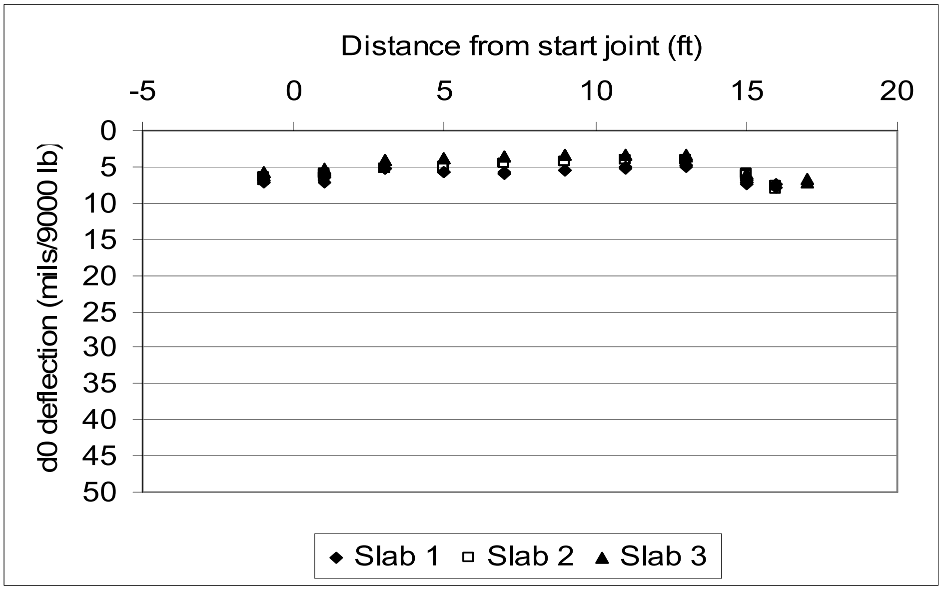

Figure 6.25b Afternoon Deflection Profile for EB I-96

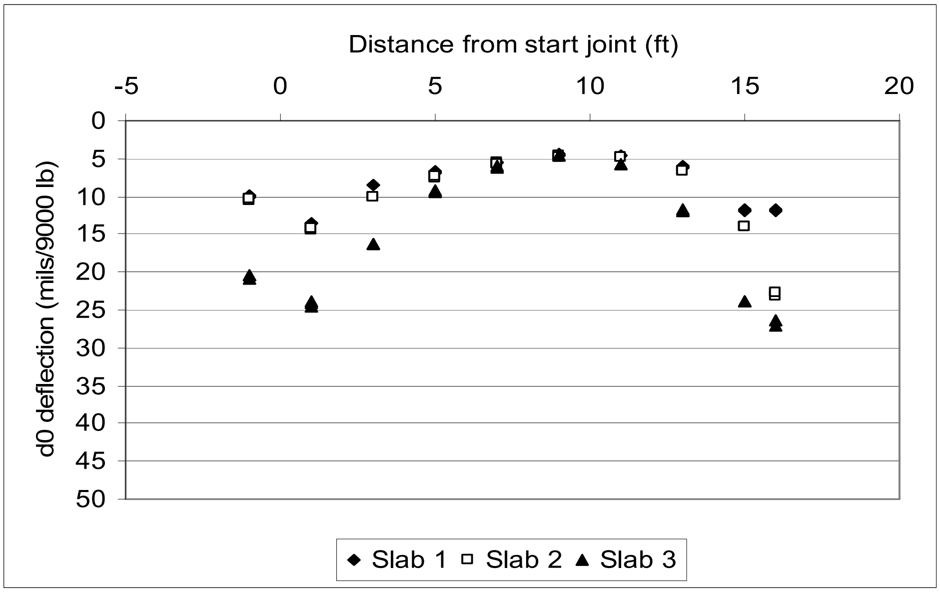

Figure 6.25c Evening Deflection Profile for EB I-96

6.3 Feasibility of Preventive Maintenance

The Department has very little experience in using preventive maintenance (PM) treatments to extend the service life of JPCP with transversely cracked slabs. These pavements have also developed extensive loss of joint support with an associate reduction in load transfer capability. The pavements that have received PM have been all pre-WWII era JPCP, where minor patching (det. #7) with thin HMA overlays was the normal treatment for all concrete pavements. None of the JPCP projects investigated for this study had received any PM treatment through the year 2002. However, as previously mentioned, the Southwest Region has decided that the condition of EB I-94 (CS 11017) west of Watervliet is too severe for PM and has scheduled the pavement for reconstruction in 2007. I-94 and some other JPCPs with cracking are now requiring considerable “reactive” hand patching to temporarily repair spalled areas.

The authors are unaware of any published guidelines for determining whether a JPCP with cracking and the aforementioned associated deficiencies is suitable for PM. So, how to judge the pavement’s suitability for PM becomes the question. Treatment types, which are generally interchangeable between JRCP and JPCP, are not an issue. The department is well versed in PM techniques, like full-depth patching, dowel bar retrofitting, and diamond grinding, so this report will not discuss those treatment details. But, the department needs guidelines to help determine the timing and suitability of PM for this condition. The following discussion regarding the merits of using PM to rehabilitate JPCP with mid slab cracking is intended to encourage the Department to eventually draft such a guide.

The first step in the guide should be data gathering and some initial analysis that would involve the following items:

- Determine the pavement’s current distress level (extent, severity, and intensity) and especially its rate of deterioration (cracking).
- Project the extent and severity of future cracking to judge when PM should be scheduled.
- Determine the extent of slab uplift, loss of support, deflection variability, LTEs across joints and cracks, and associated dowel looseness. Test and compare slabs both with and without cracking.
- Review the concrete mix design to verify the size and type of coarse aggregate used and concrete properties during construction, as to their combined influence on crack initiation.
- Verify how well the drainage system is functioning.

A primary purpose of the data-gathering process is to possibly isolate the respective causes for the cracking. As was found with this investigation, likely causes can be related to design parameters, construction practices and material selection. Another important purpose (clue) is to determine how uniformly the cracking is dispersed over the project limits. Generally, design related causes are uniformly dispersed, whereas, construction causes are isolated to a particular portion of the project. The extent of material related causes depend on how often changes in the mix design or the use of a particular material occurred.

Once the likely cause for the cracking is determined, one can better assess whether a PM treatment can correct the cause to prevent new slab cracking or significantly reduce its’ rate of occurrence. Table 6.20 is a summary of the probable causes (suspected or confirmed) discovered during this investigation for all projects with mid-slab cracking. The authors’ assessment of whether the cause can be corrected using current department PM treatments and practices is also given.

Table 6.20 Causes of Cracking and Suitability of Preventive Maintenance

Cause of Cracking	Suspected	Confirmed	Correct with PM	
			Yes	No
Design				
Missing isolation joints		X		X
Slab joint location		X		X
Joint spacing		X		X
Construction				
Cross section not per plan		X		X
Frozen base/subbase		X		X
OGDC segregated or not compacted	X			X
Subgrade disturbed – not repaired		X		X
Poor concrete consolidation	X			X
Late relief sawing at joints	X			X
Hot weather construction		X		X
Early loading by construction vehicles	X			X
Material				
High concrete CTE	X			X
Low concrete toughness	X			X
Aggregate interlock lacking - < LTE		X		X
Concrete shrinkage – wider relief crack		X		X
Non-Category				
Environmental impacts	X			X
Heavy truck loading	X			X

In all cases, using preventive maintenance would not correct the causes for the mid-slab cracking on the JPCP projects investigated. However, preventive maintenance was not intended to literally correct design, construction and material deficiencies. The primary purpose of PM is to significantly reduce the pavement's rate of deterioration to extend its service life in a cost effective manner.

Table 6.21 is an assessment by the authors of how effectively PM would fulfill its intended purpose for the JPCP projects investigated. The assessment is only derived from the investigation findings together with the authors working experience and intuitive knowledge regarding concrete pavement performance.

Table 6.21 Effectiveness of Preventive Maintenance to Extend Pavement Service Life

Project/Location	Primary Cause for Cracking	Effectiveness of Preventive Maintenance	
		Likely Best Fix	Achieve Extended Service Life
I-275/I-96 ramp/ Novi	Frozen subbase	Full-depth patching	Likely
US-12/City of Wayne	Jointing & heavy loading	Full-depth patching	Likely
I-75/Detroit	Poor construction workmanship	Slab replacement with grinding	Very likely
EB I-96/Livingston Co.	Loss of support at joints	Grinding	Highly unlikely
WB I-96/Livingston Co.	None found/minimal cracking	Full-depth patching with grinding	Likely
EB I-94/Watervliet	Loss of support at joints	Reconstruction	Not applicable
WB I-94/Watervliet	Loss of support at joints	Grinding	Unlikely

The pavement cracking for the I-275/I-96 ramp, US-12, and I-75 projects are isolated to specific areas of the project limits. The remaining portions of these projects appear to be performing as expected, which should make them suitable for future preventive maintenance. The fixes listed in Table 6.21 were selected assuming any pavement distress will occur at normal times and rates during the pavement's service life, similar to other JPCP projects without known cracking.

In contrast, the I-94 and I-96 projects are unlike the others, as they are not conforming to normal expectations. Because of their unique performance, each travel direction for I-94 and I-96 should be considered as a separate project for consideration for preventive maintenance.

The easiest project to judge is EB I-94. The high rate of deterioration for EB I-94 clearly makes it unsuitable for PM. Some form of reconstruction, as programmed by the Region, is the only prudent course of action.

WB I-94 is more difficult to assess. WB I-94 is cracking at a slower rate (8.5 percent third year, 36 percent sixth year), than EB I-94 did (9.5 percent first year, 58 percent fourth year, 74 percent seventh year). However, the amount and rate of cracking for WB (74 percent increase from 1999 to 2002) is cause for major concern. The LTEs for WB I-94 are low, while the maximum deflections at the joints were considerably less than EB I-94. Thus, loss of support is less in duration and magnitude. Still, there are too many unknowns about WB I-94 to be confident about the effectiveness of using PM fixes to extend its' service life.

The rate of deterioration for EB I-96 is also high, as 18 percent of the slabs in the right outside lane were cracked after four years. Assuming deterioration continues at a linear rate, which is a very conservative estimate, in ten years about 45 percent of the slabs will be cracked. Thus, the authors believe any PM fix will not be cost effective. If PM is used, the objective should be to temper the effects of truck loading. The most effective PM fix for that purpose is surface grinding to smooth the profile to reduce the dynamic loading from heavy trucks. The prognosis for EB I-96 is following the same pattern as EB I-94, but at a slower pace.

WB I-96 is the outlier of the “four” projects. Its’ current rate of deterioration (cracking) is very low and not changing. From appearances, WB I-96 should be a suitable candidate for PM[23], if indeed its performance continues to be normal. WB I-96 was not formally investigated for this research project, but was studied as part of the concurrent research project regarding loss of slab/base contact during upward curling. That project report will discuss the findings and elaborate on the likely future performance of WB I-96.

In summary, the Department should be more cautious and thorough in deciding the effectiveness of PM for JPCP projects that develop mid slab cracking. An earnest engineering effort should be made to understand the cause of the cracking before a PM treatment is utilized. Isolated distress needs to be dealt with separately from distress occurring uniformly. Although the treatment “tool box” for JPCP is very similar to JRCP, the benefits from each treatment are not likely to be equivalent. To date, the Department lacks any field experience regarding PM treatments for JPCP. Thus, monitoring the performance of any future PM project is very important to qualify the estimated benefits. In this case the use of “control sections” would be beneficial. A key study finding showed that the shorter slab length for JPCP versus JRCP plays a very significant role as to how a pavement responds to temperature and load. This factor alone will ultimately decide how effective PM becomes for JPCP projects.

6.4 Hypothesis for Mid Slab Cracking

A primary objective of this study was to determine why certain JPCP projects were cracking transversely near mid slab, as the relative short slab length of JPCP is suppose to prevent the slab from cracking in this manner. Of the four projects investigated for this study, three can be explained because of isolated factors found during site testing. These were mentioned in the previous section.

The fourth project, I-96, is the focus of this section, as it represents the greatest concern for the department because it may be a precursor of future JPCP project performance. This study was not intended to conduct a complete forensic investigation, so only a hypothesis for the cracking is presented. Other ongoing research is continuing to investigate the cracking problem in more detail.

The hypothesis for the mid-slab cracking is based upon four major factors that this report has previously mentioned. Before they are discussed below, the reader is reminded of some elementary theory regarding fracture mechanics of concrete slabs.

Concrete is neither homogeneous nor isotropic which makes its performance prediction and modeling less reliable. This fact is more applicable as the mass of concrete becomes smaller. A non-homogeneous concrete slab lying on a base of unbound gravel, while under the affects of temperature changes and loading, is a complex structure to model or analyze simply. Until the recent development of computer (finite element) modeling techniques, most practitioners, like the department, relied solely on its collective experience from many years of analyzing empirical

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[23] I-96 received a Preventive Maintenance project (47065-81402A) in the May-July period of 2005 that included mostly full-depth patches at cracks and some joints and diamond grinding of the outside, right lane. Both travel directions were involved.

performance data. The general principle followed to prevent cracking was simple: If the (tensile or flexural) strength of the concrete was not exceeded, a pavement slab should not crack. In addition, Miner's rule was applied, which says that if the tensile stress remains less than approximately 50 percent of the concrete's tensile strength, fatigue cracking should be prevented. Thus, historical pavement engineering of rigid slabs was conservative in scope and expectations. These past principles still apply today and are pertinent to explain today's mid-slab cracking. Fortunately, the investigative effort of researchers has been enhanced by the capability of modeling via computer analysis.

Low Load Transfer Efficiencies

LTEs at the joints for EB I-96 were low when the pavement was constructed in 1997 and continue to decline over time. The likely cause is the loss of aggregate interlock across the joint relief crack from permanent slippage (shear displacement) of the coarse aggregate particles. The blast-furnace slag is believed to have accentuated the problem because the relief crack was probably initially wider than normal from excessive (mass) shrinkage. The PRCE/UM research study (RC-1404) previously mentioned, found that the LTE for slag concrete drops dramatically when the crack width exceeds 0.035 inches. The benefit of using large, tough aggregates for the concrete mixture is demonstrated by the lower deflections for the I-75 project, which used a P1-mod concrete mixture. Also, the amount of slab cracking on I-75 is isolated and stable.

Loss of Support

Clearly, a loss of support (high deflection) at the joints exists during upward curling, especially during the early morning hours when the slab has a negative temperature differential. This is characterized by the very noticeable "slab rocking" phenomena. The loss of support occurs when the curling stress exceeds the counter force from the slab's weight. The magnitude of the loss (gap) at the slab-base interface for EB I-96 is accentuated by the existence of a "built-in" curl condition when concrete set occurred during a positive temperature differential. The gap's magnitude depends upon the actual temperature differential (ΔT). For EB I-96 the maximum daily ΔT is higher, because it equals the positive portion at the time of set plus any negative differential that occurs. The positive portion for EB I-96 is estimated to vary from 10-20°F.

The actual highest deflection values occur when the temperature differential is most negative which is at night (am hours) during the summer. Testing with the FWD is not practical or safe at that time.

The concrete's coefficient of thermal expansion (CTE) directly affects the slab's range of expansion and contraction during temperature changes. Thus, higher CTE values will accentuate the amount of curling and subsequent loss of support for a given ΔT. A true CTE value of slag concrete is difficult to determine due to the inconsistent physical properties of slag particles, like its porosity. Two specimens from the September 2000 test site on EB I-96 were tested using the FHWA CTE test procedure. The values were 5.3 and 5.6×10^{-6} in/in/°F, which are considered to be higher range values.

Truck Loading

There are several types of multi-axle trucks that are unique to Michigan highways. A legal multi-axle truck can have, as many as, eight consecutive axles grouped together. The spacing between individual axles within a multi-axle group is 3.5 ft., while the spacing for a single axle is 9 ft. There are several axle combinations that will allow each end of a 14-16 ft. slab to be loaded simultaneously[24]. When both slab ends are loaded during corner uplift, normal rotation of the slab to alleviate stress is prevented. When rotation is prevented, the combined stress induced at mid-slab can approach twice the amount when only one end of a slab is loaded. The magnitude of the stress is dependent upon the area of the loss of contact at the slab-base interface from curling. The portion of unsupported slab acts like a cantilever beam, where the moment arm is the distance from the joint to where the slab resumes contact with the base.

Crack Sensitivity and Propagation

The PRCE/UM research for the department (RC-1404), by Dr. Hansen and Elin Jensen, studied the toughness and fracture energy properties of concrete using different types of coarse aggregate. Toughness is the ability of concrete to resist the initiation of cracking, while fracture energy determines its' resistance to crack propagation. Both properties were found to be highly dependent upon the type of coarse aggregate. In comparison to natural gravels and crushed dolomites, concrete using blast-furnace slag and some limestone sources had the lowest toughness and resistance to propagation. The study found that fracture energy is insensitive to compressive strength, but improves with a larger aggregate size that is durable and tough.

Cores from I-96 and I-94 confirmed that the mid-slab cracks initiate at the surface at the edge of the slab. Once a partial-depth crack occurs, there is a loss in the slabs' cross sectional area to further resist the propagation of the crack. The research by Hansen and Jensen found that the resistance to propagation is not directly proportional to the remaining cross sectional area. For example, in the case of blast-furnace slag, a 25 percent area loss from cracking results in an approximate 50 percent reduction in the remaining area's ability to resist further crack propagation.

Summary

All the factors discussed are likely working interactively to cause sufficient stress to crack a slab. Preliminary modeling (ISLAB2000) shows that none of them acting independently can cause enough stress to exceed 50 percent of the slab's tensile strength. Thus, sufficient resistance to fatigue cracking is maintained. This finding also agrees with the literature. The major contributor of the four is probably loss of support, which is the emphasis of other ongoing research. Figure 6.40 summarizes mid-slab stress for different amounts of negative ΔT, 5.6 CTE, 8-12 inch thick slabs with loading by a typical 5-axle truck. This shows Miner's 50 percent rule can be exceeded and the possibility of fatigue cracking exists. Input values for the pavement cross section (slab length and k factor), are from the site testing of EB I-96, which is applicable to other Michigan projects.

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[24] The total length of eight consecutive axles is about 24.5 ft., which easily exceeds the length of any JPCP slab.

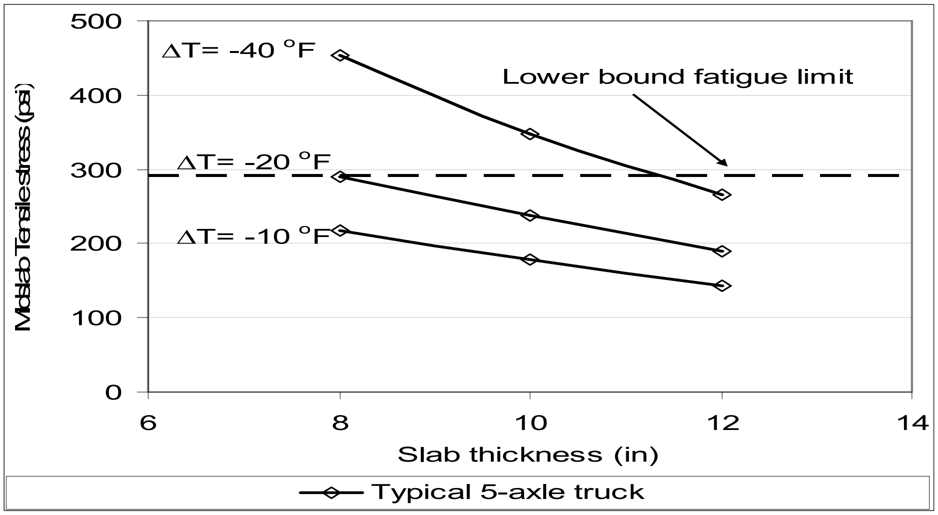

Figure 6.40 Mid-Slab Tensile Stress Prediction (ISLAB 2000) at Top of Slab 15 ft. JPCP, CTE of 5.6 x 10^-6 in/in/ F, k value of 250 psi/in

The predicted stress values indicate that very large negative gradients combined with high CTE concrete are needed to initiate fatigue cracking. These large gradients are not typical of Michigan's environmental condition. So, additional uplift from built-in curl and likely moisture warping (not confirmed) are likely contributors.

The most plausible reason for the current stable condition of WB I-96 is reduced joint deflections while under loading. Westbound was constructed during cooler weather than eastbound, which reduces mass shrinkage and will form a narrower relief crack at the joint. A narrower relief crack increases shear resistance, which alone will reduce deflection. Also, it is less likely a significant built-in curl condition exists.

This hypothesis is supported by other research. A FHWA sponsored study (FHWA-RD-95-111), by ERES Consultants (Yu, Smith, and Darter) concluded that the occurrence of top-down JPCP cracking depends on thermal and moisture conditions. These develop from factors like; time of day paving occurs, climatic conditions during paving, drying shrinkage, and long-term moisture gradients in the slab. The authors also concluded that the most damaging loading condition for a slab is when a tandem axle straddles one joint while the steering axle is over the other joint.

7.0 CONCLUSIONS

The objective of this research project involved several efforts that were intended to assess the overall condition of fifteen department JPCP projects constructed from 1995 through 2000. The emphasis of the study was to investigate and determine why four of the fifteen projects constructed during that period developed mid-slab cracking. When the study began, the department was trying to determine the relative life cycle cost of JPCP, as compared to JRCP, which historically has been the department's standard concrete pavement. Thus, the results from this study were intended to provide the department with a preliminary comfort level, as to the feasibility of using JPCP for Michigan conditions. Following is a summary of the findings/ conclusions from this study.

7.1 JPCP Designs are Adequate Structurally

The pavement (structural) design used for the four projects with cracking was adequate for the conditions. Thus, increasing the thickness of the slab would not have prevented the cracking. The use of an open-graded base versus a dense-graded base did not contribute to the cracking. Accept for the project exceptions noted in the report's text, the pavement cross section is being constructed per plan.

7.2 Construction Practices Need Improvement

Of the four projects with cracking, three can be attributed to poor construction practices and workmanship. Two of those three had obvious oversights in the construction or preparation of the base/subbase for paving. In the case of the I-275 ramp the subbase was most likely frozen when the slab was paved. With I-75, the thickness of the base/subbase was not uniform across all lanes, which caused differential support and frost heaving. The US-12 project lacked isolation joints around the drainage structures and the joint layout (slab length) should have been adjusted from standard to conform to the potential impacts from the onsite commercial development.

Paving during summertime conditions does influence the tendency of a slab to crack. Built-in curling will occur from hot weather paving at time of set, when a positive slab temperature differential exists top to bottom. Concrete mixture temperatures should stay well below the maximum allowed by specification to serve as a reserve for unavoidable radiant heating that will occur.

7.3 JPCP is Sensitive to Cracking and Rapid Crack Propagation

For some circumstances JPCP appears to be susceptible to top-down transverse cracking from a combined stress that results from a loss of support at the joint when the slab is upward curled at the corners, and when under simultaneous loading at both joints from a multi-axle truck.

Additional factors, such as a high CTE for the concrete mixture, coarse aggregate that lacks toughness, and the presence of built-in curling, will all increase the probability that mid-slab cracking will occur.

Once a crack starts at the slab's edge, it rapidly propagates across the complete slab cross section. Rapid propagation is also evident by the numerous sympathy cracks that have formed across the adjacent (passing) lane where there is much less truck loading. Spalling along the crack and minor faulting appear soon after the crack forms. Previous research found that the use of slag concrete promotes this phenomenon because of its relative low fracture energy, which is unrelated to its relative high compressive strength.

Using input values obtained from EB I-96 site testing, preliminary modeling using ISLAB2000 found fatigue failure can occur for a combination of the probable cracking factors.

7.4 High Rates of Distress Formation with JPCP

If mid-slab cracking does occur, like the EB I-96 and I-94 projects, the rate of crack initiation accelerates over time and the expected design service life will not be achieved. The acceleration of cracked slabs prevents the use of preventive maintenance to cost effectively extend project service life. The root causes for the cracking are not correctable by using preventive maintenance. Costly reconstruction becomes the only prudent option after spalling (reactive patching) and ride quality reach unacceptable levels.

7.5 Low Joint Load Transfer Efficiencies/High Deflections

FWD testing found remarkably low load transfer efficiencies (LTE) across joints with very high deflections for “newer” pavements. LTEs as low as 20 percent were common, which indicates the dowels are ineffective. The disparity between LTE when measured on both sides of a joint is most noticeable, which indicts an early slippage across the relief crack from a loss of aggregate interlock. Deflection values are highest in the morning when a negative temperature differential exists top to bottom when the slab is experiencing upward curling. “Slab rocking” in the morning, which signifies high slab deflections are occurring, was very evident when the researchers made site investigations.

Summary

The study investigation did not find sufficient cause to recommend that the department not use JPCP. However, the findings do support that JPCP and JRCP are different in terms of their response to changes in slab temperature and loading from multi-axle trucks. When there is a loss of support at the joint, the shorter slab length of JPCP appears to be more susceptible to mid slab cracking. The shorter slab length is also less forgiving when the base/subbase does not provide uniform support, which any rigid pavement design depends upon. Most importantly, a distressed JPCP is less conducive to treatment by normal preventive maintenance methods to extend its service life. Therefore, any JPCP project in a heavy truck corridor should be closely monitored during construction to ensure that factors related to crack inducement are negated within practical limitation. Because current specifications allow contractors considerable latitude in material choices and quality control practices, they also have a decisive role in the prevention solution.

8.0 RECOMMENDATIONS

The recommendations from this investigation address several needs, which are grouped under the following headings.

8.1 Enhanced Inspection and Specification Enforcement

The projects on I-275 ramp, US-12, and I-75 are instances where more diligent inspection and a minor design modification for site conditions during construction would have prevented the slab cracking for the sites investigated.

The study findings also provide cause to urge that existing specification requirements for influential work items be rigidly enforced because they significantly influence overall slab performance and the potential for cracking. The bullet items, under each work item, specifically address the objectives for ensuring the action is performed correctly.

(1) Concrete curing procedures and aggregate moisture control during batching
  - Less plastic shrinkage that reduces the width of joint relief cracks to improve LTE.
  - Eliminate surface drying shrinkage and surface crusting that reduces the amount and severity of eventual spalling.
  - Control evaporation rates that induce permanent warping stresses similar to curling.

(2) Placement of open-graded base
  - Ensure restrictions on segregation are met to reduce potential for "weak" areas of base support and the possibility of post-compaction under traffic loading.

(3) Concrete paving mixtures
  - A lower mix temperature reduces the occurrence of built-in curling from a positive temperature differential. Reducing the positive temperature differential from top to bottom will ultimately reduce stresses from upward curling.
  - Expand the use of the P1-modified mixture, but only with the use of sound, durable and tough coarse aggregate. A larger coarse aggregate size reduces the paste component of the mixture that will reduce shrinkage. A large, durable, tough aggregate will improve LTE and most importantly improve the concrete's toughness, to resist the initiation of cracking.

8.2 Field Studies

The study findings suggest that the department consider some field studies to better understand and quantify conditions present during construction that may then trigger cracking or sometime shortly after the pavement is completed.

The study found that morning paving during hot summer conditions greatly enhances the possibility of a positive temperature differential existing at time of set. The department should initiate temperature monitoring of selected JPCP projects to better understand the change in temperature gradients prior to set and the first days of strength gain. Knowing the amount of positive temperature difference at time of set will help determine the potential for a loss of support to occur when the pavement later cycles to a negative temperature differential.

The temperature data should help resolve a suspected issue regarding the EB I-96 investigation, as to when the pavement was opened to heavy construction vehicles, which might have initiated edge cracking when the slab's tensile strength was too low. Besides strength, the opening requirements for construction vehicles should depend on whether the potential exists for early loss of support to occur.

The cost/benefit of using coverings, fog sprays, and evaporation retardants should be explored for morning paving during hot summer weather to eliminate the risks of shrinkage and built-in warping/curling stresses.

8.3 Monitor Preventive Maintenance Treatments of JPCP Projects

The actual benefit of using PM to extend the service life of a cracked JPCP, like EB I-96 or WB I-94 is unknown. The cost/benefit of any such project should be thoroughly analyzed after carefully monitoring its performance. The first projects should have control sections, without treatment, to measure the original pavement's actual rate of deterioration in comparison to the treated sections. With experience, the department should draft guidelines similar to those for JRCP.

8.4 Modify JPCP for Heavy Commercial Routes

Until other ongoing research is completed and the risk factors for cracking are better understood and qualified, the following preventive steps should be taken to reduce the risk of mid slab cracking on major (truck corridors) interstate routes, like I-94 across the state and I-75 south of Bay City to the Ohio line, where high numbers of multi-axle trucks exist.

(1) Use the P1-mod concrete mixture to assure a high quality aggregate is used (benefits already mentioned).
(2) Control the time of paving by avoiding mornings (summertime conditions) to reduce the possibility of set occurring when a positive temperature differential exists. Do not exceed a ΔT of 10°F.
(3) If feasible, adjust the slab length to avoid matching the axle configuration of the most numerous multi-axle trucks using the route within the project limits.

8.5 Data Needs and Future Research

Several needs for data gathering and future research projects arose during this short investigative study to better understand the performance of JPCP and its contrasts with JRCP. No specific priority is given to them due too the department's changing resources and research commitments.

(1) Temperature monitoring[25] of both JPCP and JRCP slabs to better understand curling changes and the subsequent loss of support it causes. A variety of structural designs and construction conditions should be monitored.

(2) The early loading of concrete slabs by heavy construction vehicles may possibly be initiating top-down edge cracking if the slab had set with a "built-in" curl condition. Further research is needed to determine how plausible this phenomenon is and if there is a need for additional opening criteria besides concrete strength.

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[25] The Materials Research Group has started to monitor slab concrete temperatures with wireless data recorders.

(3) Further research can help determine the relationship of mass shrinkage and the width of joint relief cracks, which ultimately determine the shear resistance during load transfer across the joint.

(4) Additional acceptance criteria are needed for a new concrete pavement when a "built-in" curling condition has occurred. Contractors should especially be supportive if a warranty is required to quantify their participation regarding future condition results.

(5) Spalling existed only at cracks, while joints exhibited an excellent appearance. Yet, joints have high deflections similar to mid-slab cracks without dowels. Further research is needed to determine the reason.

(6) In Japan, concrete pavements are reinforced with deformed bars, placed primarily along the slab's edge. Modeling and a trial field project should test this concept as a means to retard crack initiation and especially crack propagation. The concept is best suited to longer JRCP slabs, but application to a hybrid JPCP design is also worthwhile.

(7) Extensive analysis using Finite Element Modeling is needed to better understand the affect of heavy multi-axle trucks when their axle configuration matches the pavement's joint spacing. A variety of likely support conditions should be analyzed to find the most adverse combination of factors.

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APPENDIX A.

Special Provision for P1-Mod Concrete Mixture

MICHIGAN
DEPARTMENT OF TRANSPORTATION
BUREAU OF HIGHWAYS

SPECIAL PROVISION FOR PORTLAND CEMENT CONCRETE PAVEMENT (MODIFIED)

C&T:JFS **1 of 2** **11-14-97**
CS:81103/JN:43503A
C&T:APPR:JTL:MLL 12-11-97
(1998 construction M-14 inlay project, Tony Angelo) C:\specs\p1mod.m14

a. Description.-This specification sets forth requirements for furnishing Portland cement concrete, Grade P1(Modified), for mainline, shoulder, and miscellaneous pavement applications. The Contractor shall not be granted the option of using other concrete Grades or Types in lieu of Grade P1 (Modified), unless authorized by the Engineer. The 1996 Standard Specifications for Construction and the Special Provision for Furnishing Portland Cement Concrete (Quality Assurance) shall apply except as modified herein.

b. Materials.-Coarse aggregates shall be Michigan Series 4AA and 6AAA. The absorption of the coarse aggregates sampled from the on-site stockpile to be used in the pavement concrete shall not exceed + 0.20 percent from that reported for the respective coarse aggregate sample tested for freeze-thaw durability. The bulk specific gravity of the 4 AA coarse aggregate sampled from the on-site stockpile to be used in the pavement concrete shall not be less than that reported for the respective coarse aggregate sample tested for freeze-thaw durability. The 4AA coarse aggregate shall originate geologically only from natural sources. The 6AAA coarse aggregate shall meet the requirements of Subsection 902.03. The use of a pre-blended coarse aggregate meeting the gradation requirements of the on-site blend specified in this section is not permitted. The fine aggregate shall meet the requirements for natural sand Number 2NS. Portland cement shall be Type I. All materials used in the concrete mixture shall be from Michigan Department of Transportation (MDOT) approved sources.

Handling and batching equipment shall be capable of simultaneously and separately controlling each individual coarse and fine aggregate according to NRMCA plant certification requirements. Additional aggregate storage equipment may be required to assure proper handling and batching of the coarse aggregates to prevent segregation and contamination.

Concrete Mixture Requirements.-Except as modified herein, the Contractor is responsible for determining the concrete mixture proportions (mix design) according to the Special Provision for Furnishing Portland Cement Concrete (Quality Assurance) contained in this proposal. The Contractor mix designs must be submitted for review by the Engineer a minimum of five working days prior to concrete placement; however, concrete placement shall not commence until deficiencies in the mix design are corrected by the Contractor and reviewed by the Engineer.

Proportioning of the coarse aggregates in the mix design shall be based on an on-site blend of approximately 40 percent 4AA and 60 percent 6AAA coarse aggregates, by weight. The combined bulk volume (dry, loose) of coarse aggregates per unit volume of concrete shall be greater than 0.72. The minimum cementitous material content given in Table 1 of the Special Provision for Furnishing Portland Cement Concrete (Quality Assurance) does not apply. The Portland cement content shall be between 279- and 335-kg/m^3. Strength requirements shall be as specified for Grade P1 concrete.

c. Construction Methods.-Construction of Portland cement concrete pavement shall be according to Subsection 602.03 of the 1996 Standard Specifications for Construction.

d. Measurement and Payment.-The completed work as measured for Portland cement concrete, Grade P1 (modified) will be paid for at the contract unit prices for the following contract items (pay items):

Contract Items (Pay Items)	**Pay Units**
Concrete Pavement, Non-Reinforced, Furnishing and Placing	**cubic meter**
Concrete Pavement, Non-Reinforced, Finishing and Curing	**square meter**
Concrete Pavement, Miscellaneous, Non-Reinforced, Furnishing and Placing	**cubic meter**
Concrete Pavement, Miscellaneous, Non-Reinforced, Finishing and Curing	**square meter**

Concrete Pavement, Non-Reinforced, Furnishing and Placing shall be measured and paid for by volume in cubic meters. The Engineer will determine the volume of concrete used each day, or fraction thereof, based on the number of batches placed in the work and the nominal volume of concrete per batch. This amount will be documented by the batch ticket printouts. This item shall include all materials, labor, and equipment necessary to furnish and place the concrete mixture. **Concrete Pavement, Non-Reinforced, Finishing and Curing** shall be measured in place and paid for by area in square meters. This item shall include all materials, labor, and equipment necessary to finish and cure the concrete inlay and construct the longitudinal joints. Construction of transverse joints, lane ties, and removal of the existing pavement shall be paid for separately. **Concrete Pavement, Miscellaneous, Non-Reinforced, Furnishing and Placing** and **Concrete Pavement, Miscellaneous, Non-Reinforced, Finishing and Curing** shall be measured and paid for as described above, but shall be used for areas of pavement gapping for maintaining ramp traffic.

APPENDIX B.

I-96/I-275 Field Report - Metro Region

MDOT
Michigan Department of Transportation

OFFICE MEMORANDUM

DATE: November 30, 2000

TO: Dave Smiley
Pavement Research Engineer
Lansing C & T Division

FROM: John K. Boimah
Area Soil Engineer - Metro Region

SUBJECT: I-275 JOINTED PLAIN CONCRETE PAVEMENT STUDY

On Saturday, August 19, 2000, pavement condition survey, pavement coring and subgrade investigations were performed on NB I-275 connection ramp to WB I-96. The purpose of this survey was to determine the cause of early transverse cracks in the non reinforced concrete pavements. You will find the following information attached:

- Results of our investigation
- A layout of the ramp showing the locations of transverse cracks
- Pavement cores and soil descriptions
- Typical cross sections of the ramp
- Compression test results
- Pavement Management Survey Data.

Transverse cracks were observed to have started at Station 1646+90 in the sag vertical curve of the ramp and ended at Station 1644+66.

Old plans from 1967 indicate the soil series is Conover. A profile sheet shows the location has been cut (cut section) about 12 ft. Coring data from Location Number 1, 2,3,5 and 6 show the subgrade is moderately compact moist brown sand over compact moist fine to medium sand. At Location Number 4, the subgrade is moderately compact moist brown sand over hard grayish-brown silty clay. Data from our field investigation is not indicative of a Conover. This may be because the section has been cut.

Our results from blow counts were high. The counts ranged from 28 to 59 indicating densely compacted subbase and subgrade materials. Most core locations were dry except at core number two which had a water lever at 1.29 meters.

Visually checking the cores, I observed some of the aggregates at the shearing surface of the crack cores were sheared. (Photographs are attached)

There are two compression test results for concrete cylinder cores. One core was from a pavement with no transverse cracks and another from a pavement with transverse cracks revealing the following information: The core from the location with no transverse crack broke at a compressive strength of 5,470 psi while the core from the pavement with transverse crack broke at a compressive strength of 4,760 psi. These compressive strength results are greater than the 28-day compressive strength of 3,500 psi.

I requested PMS and RSL data from the Region Pavement Management Engineer. He provided RQI reading of 40 for 1996. I was told that RSL data were not collected until 1997. The data for 1999 had a DI of 0.3, RQI of 40 and RSL of 21.

In conclusion, the subbase and subgrade were highly consolidated granular and well-drained materials. Concrete cores broke in compression tests well above the specified 28 days compressive strength required. The cracks, when viewed through our core locations, extended to the open graded drainage course.

If you have any question, I can be reached at (248) 483-5165.

METRO MATERIALS & TESTING OFFICE

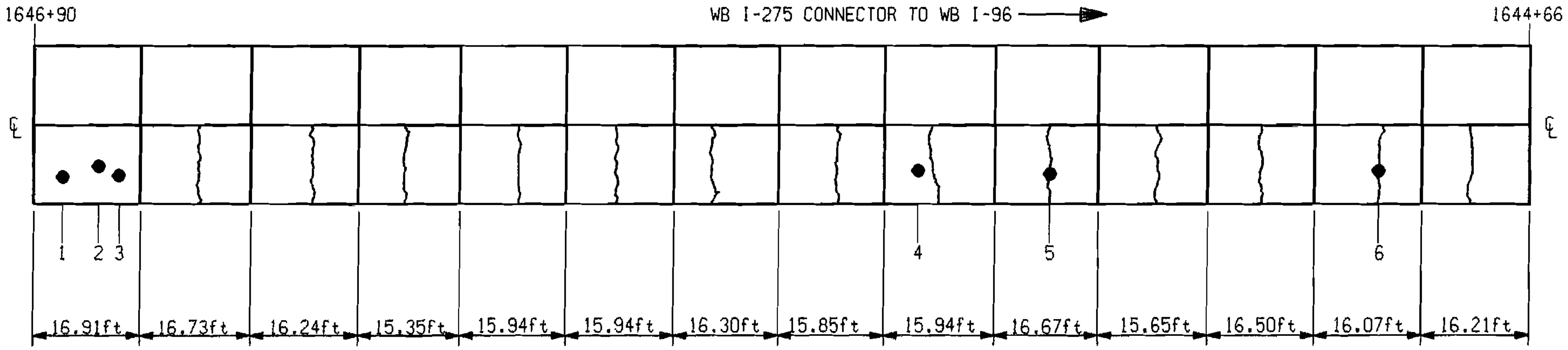
John K. Boimah

JKB:dvd

Attachment

cc: R. Ostrowski M. Grazioli Will Hansen, University of Michigan

2000
I–275 JOINTED PLAIN CONCRETE PAVEMENT STUDY

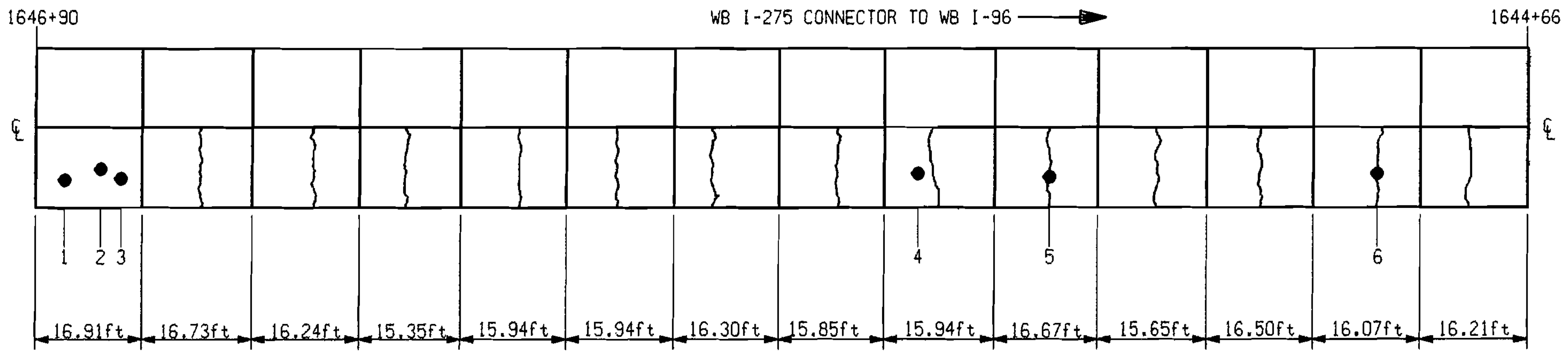


PREPARED BY: METRO REGION
DATE: 11/21/00
FILE: P:\MISC\JOHNCORE.DGN
CS: 63192

LEGEND

● CORE LOCATION
~ CRACKS DEVELOPING FULL WIDTH
— JOINT

2000
I–275 JOINTED PLAIN CONCRETE PAVEMENT STUDY



PREPARED BY: METRO REGION
DATE: 11/21/00
FILE: P:\MISC\JOHNCORE.DGN
CS: 63192
JN: 36003A

LEGEND
- ● CORE LOCATION
- 〰 CRACKS DEVELOPING FULL WIDTH
- — JOINT

TEST HOLE NO. RESEARCH CORE #1

STATION 1646+38.69 1.30m RT. OF LT. E.O.M.
NB. I-275 RAMP TO WB. I-96
OAKLAND CO.
DATE DRILLED 08-19-00

.0		CONCRETE, CRUSHED LIMESTONE AGGREGATE, NO STEEL
-.326		AGGREGATE BASE
-.460		MODERATELY COMPACT MOIST BROWN FINE SAND, TRACE FINE GRAVEL (SAMPLE #1)
-1.128		MODERATELY COMPACT SATURATED BROWN MEDIUM SAND, TRACE COARSE SAND
-1.524		E.O.C.

CORE TAKEN:
416.3m WEST OF REFRENCE LINE A
OF S05 OF 63191
6" CORE (LAB SAMPLE)
NO TRANSVERSE CRACKS

TEST HOLE NO. RESEARCH CORE #2

STATION 1646+33.61 1.90m RT. OF LT. E.O.M.
NB. I-275 RAMP TO WB. I-96
OAKLAND CO.
DATE DRILLED 08-19-00

.0		CONCRETE, CRUSHED LIMESTONE AGGREGATE, NO STEEL
-.320		AGGREGATE BASE
-.457		MODERATELY COMPACT MOIST BROWN FINE SAND, TRACE CLAY
-.762	⑧ ⑭ ⑭	
-1.524	⑨ ⑮ ⑱	
-1.981		COMPACT MOIST BROWN FINE SAND
-2.286	⑰ ㉕ ㉞	
-2.743		E.O.C.

CORE TAKEN:
418.0m WEST OF REFRENCE LINE A
OF S05 OF 63191
WATER LEVEL @ 1.29m
WATER LEVEL @ 1.26m (AFTER 73 MIN.)
8" CORE

TEST HOLE NO. RESEARCH CORE #3

STATION 1646+30.19 1.40m RT. OF LT. E.O.M.
NB. I-275 RAMP TO WB. I-96
OAKLAND CO.
DATE DRILLED 08-19-00

.0		CONCRETE, CRUSHED LIMESTONE AGGREGATE, NO STEEL
-.305		AGGREGATE BASE
-.445		MODERATELY COMPACT MOIST BROWN FINE SAND, TRACE MEDIUM SAND, TRACE FINE GRAVEL (SAMPLE #4)
-1.280		MODERATELY COMPACT SATURATED BROWN MEDIUM SAND, TRACE FINE GRAVEL
-1.524		E.O.C.

CORE TAKEN:
418.9m WEST OF REFRENCE LINE A
OF S05 OF 63191
6" CORE (U OF M SAMPLE)
NO TRANSVERSE CRACKS

TEST HOLE NO. RESEARCH CORE #4

STATION 1645+57.65 1.60m RT. OF LT. E.O.M.
NB. I-275 RAMP TO WB. I-96
OAKLAND CO.
DATE DRILLED 08-19-00

.0		CONCRETE, CRUSHED LIMESTONE AGGREGATE, NO STEEL
-.329		AGGREGATE BASE
-.475		MODERATELY COMPACT MOIST BROWN FINE SAND, TRACE FINE GRAVEL, SATURATED AT CLAY CONTACT POINT (SAMPLE #3)
-1.311		HARD GRAYISH-BROWN MOTTLED SILTY CLAY
-1.524		E.O.C.

CORE TAKEN:
441.0m WEST OF REFRENCE LINE A
OF S05 OF 63191
6" CORE (LAB SAMPLE)
TRANSVERSE CRACKS

TEST HOLE NO. RESEARCH CORE #5

STATION 1644+98.75 1.50m RT. OF LT. E.O.M.
NB. I-275 RAMP TO WB. I-96
OAKLAND CO.
DATE DRILLED 08-19-00

.0		CONCRETE, CRUSHED LIMESTONE AGGREGATE, NO STEEL (CRACK RUNS FULL LENGTH)
-.314		AGGREGATE BASE
-.472		MODERATELY COMPACT MOIST BROWN FINE SAND (SAMPLE #2)
-.762	⑩ ⑯ ⑱	
-1.524	⑨ ⑮ ⑮	
-1.676		MODERATELY COMPACT MOIST GRAYISH-BROWN MOTTLED CLAYEY FINE SAND
-1.981		COMPACT PALE GRAY MEDIUM SAND
-2.286	⑪ ⑱ ⑱	
-2.743		E.O.C.

CORE TAKEN:
462.0m WEST OF REFRENCE LINE A
OF S05 OF 63191
8" CORE (DRY HOLE)

⓪ 1st 6"
⓪ 2nd 6"
⓪ 3rd 6"

NUMBERS IN CIRCLES DENOTE NUMBER OF BLOWS REQUIRED TO DRIVE A 2.00" O.D. X 1.50" I.D. SPLIT SPOON SAMPLER 3 SUCCESSIVE 6" INCREMENTS USING A 140# HAMMER FALLING 30".

CONSISTENCY DETERMINED BY INSPECTION OF SAMPLES AND BY SOIL RESISTANCE TO PENETRATION BY JET ROD AND CASING OR AUGER.

WATER LEVELS MAY BE INFLUENCED BY RESIDUAL BORING WATER

THE SOIL BORING LOGS REPRESENT POINT INFORMATION. PRESENTATION OF THIS INFORMATION IN NO WAY IMPLIES THAT THE SUBSURFACE CONDITIONS ARE THE SAME AT LOCATIONS OTHER THAN THE EXACT LOCATION OF THE BORING.

MICHIGAN DEPARTMENT OF TRANSPORTATION

SOIL BORING DATA

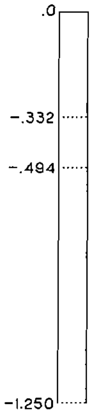
DATE	SCALE	CONT. SEC.	JOB NO.	DESIGN UNIT	SHEET NO.
11/22/2000	1 cm to .24m	63191	52277		1 of 2

DRILLER

CHECKED BY
JKB

TEST HOLE NO. RESEARCH CORE #6

STATION 1644+13.12 1.55m RT. OF LT. E.O.M.
NB. I-275 RAMP TO WB. I-96
OAKLAND CO.
DATE DRILLED 08-19-00



CONCRETE, CRUSHED LIMESTONE AGGREGATE, NO STEEL

AGGREGATE BASE

MODERATELY COMPACT MOIST BROWN FINE SAND, TRACE MEDIUM SAND, TRACE CONCRETE AND GRAVEL (SAMPLE #2)

E.O.C.

CORE TAKEN:
476.0m WEST OF REFRENCE LINE A
OF S05 OF 63191
6" CORE (U OF M SAMPLE)
TRANSVERSA CRACKS

0 1st 6"
0 2nd 6"
0 3rd 6"

NUMBERS IN CIRCLES DENOTE NUMBER OF BLOWS REQUIRED TO DRIVE A 2.00" O.D. X 1.50" I.D. SPLIT SPOON SAMPLER 3 SUCCESSIVE 6" INCREMENTS USING A 140# HAMMER FALLING 30".

CONSISTENCY DETERMINED BY INSPECTION OF SAMPLES AND BY SOIL RESISTANCE TO PENETRATION BY JET ROD AND CASING OR AUGER.

WATER LEVELS MAY BE INFLUENCED BY RESIDUAL BORING WATER

THE SOIL BORING LOGS REPRESENT POINT INFORMATION. PRESENTATION OF THIS INFORMATION IN NO WAY IMPLIES THAT THE SUBSURFACE CONDITIONS ARE THE SAME AT LOCATIONS OTHER THAN THE EXACT LOCATION OF THE BORING.

SOIL BORING DATA

DATE	SCALE	CONT. SEC.	JOB NO.	DESIGN UNIT	SHEET NO.
11/22/2000	1 cm to .24m	63191	52277		2 of 2
DRILLER				CHECKED BY JKB	

MDOT
Michigan Department of Transportation

CONCRETE CYLINDER / CORE COMPRESSION TEST RESULTS

REPORT
PAGE 1 OF 1

PROJECT NUMBER 52277C

PROJECT ENGINEER John boimah

SPECIFICATION na

CONCRETE SOURCE na

CONTRACTOR Mdot

CONSULTANT Mdot

None possible

CYL. I. D. #	DATE CAST	DATE BROKEN	AGE (DAYS)	SLUMP IN	% AIR	CONC. TEMP.	AIR TEMP.	CONC. TYPE	HGT	DIA	H/D	CF	LOAD	PSI		AVG.	P / F	TEST BY
core1	1995	8/19/00	na	na	na	na	na	na	12.00	5.94	2.02	1.00	151590	5470		2740		JAW
				na	na	na	na											
core 4	1995	8/19/00	na	na	na	na	na	na	12.00	5.94	2.02	1.00	131970	4760		2380		JAW
				na	na	na	na											

AMOUNT OF CONCRETE PLACED:

APPROVED MIX DESIGN #:

WR = WATER REDUCER:

AE = AIR ENTRAINMENT:

FA = FLY ASH:

FIELD TEST BY:

REMARKS:

LOCATION OF POUR:

(core 1) 416..3m west of refrence line (good area)

(core 4) 441.0m west of refrence line (bad area)

LAB: METRO REGION

TESTED BY:

REVIEWED BY:

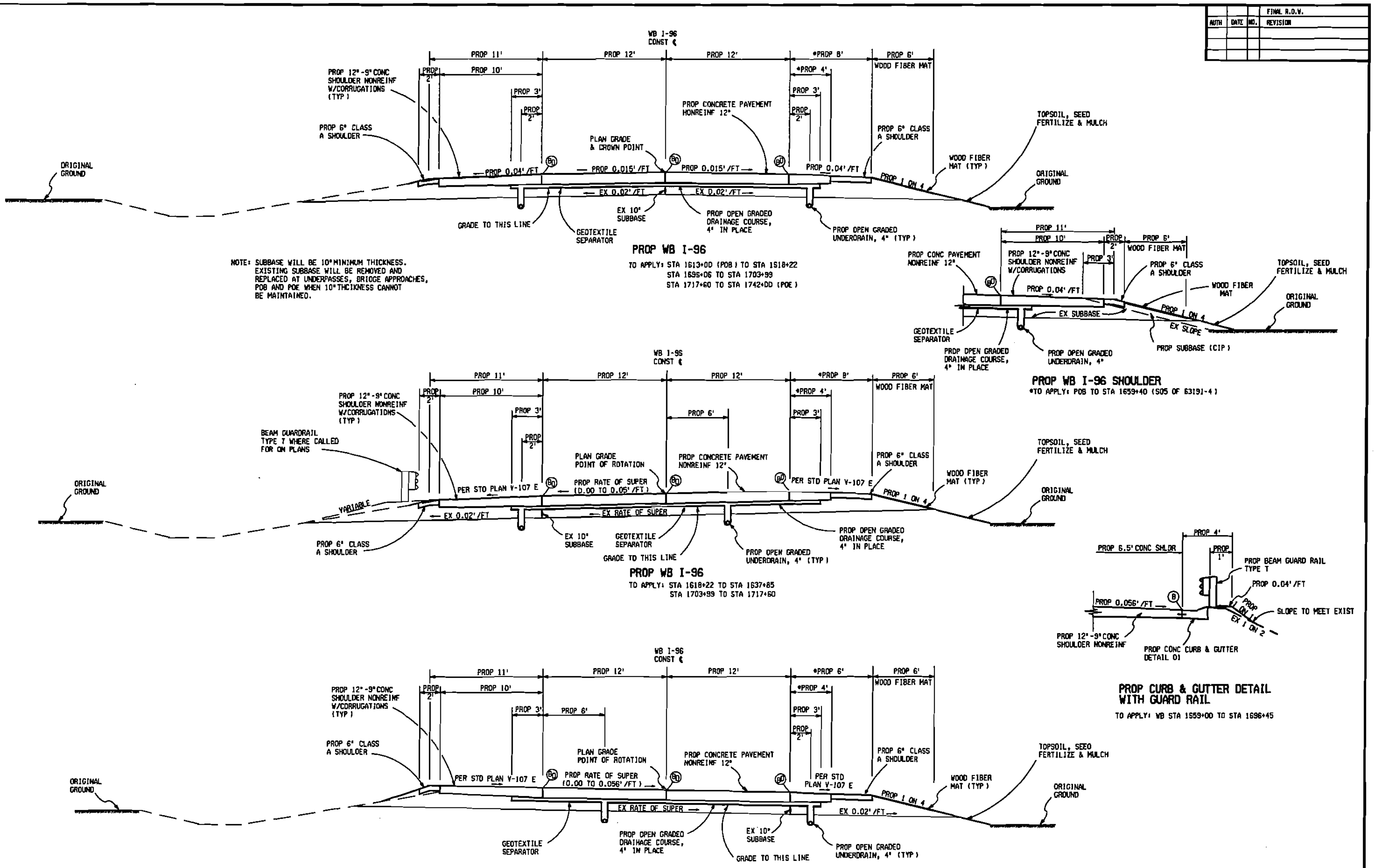
CC: DISTRICT MATERIALS & TESTING

CC: PROJECT ENGINEER

CONTROL SECTION	63191	JOB NO.	36003	SH. NO.	6

EXISTING BY: VERPLANCK DATE: 08-18-94
PROPOSED BY: DATE:
LAST CORRECTION BY: VERPLANCK DATE: 02-23-95

AUTH	DATE	NO.	REVISION
			FINAL R.O.V.



PROP WB I-96

TO APPLY: STA 1613+00 (POB) TO STA 1518+22
STA 1695+06 TO STA 1703+99
STA 1717+60 TO STA 1742+00 (POE)

NOTE: SUBBASE WILL BE 10" MINIMUM THICKNESS. EXISTING SUBBASE WILL BE REMOVED AND REPLACED AT UNDERPASSES, BRIDGE APPROACHES, POB AND POE WHEN 10" THCIKNESS CANNOT BE MAINTAINED.

PROP WB I-96 SHOULDER

*TO APPLY: POB TO STA 1659+40 (S05 OF 63191-4)

PROP WB I-96

TO APPLY: STA 1618+22 TO STA 1637+85
STA 1703+99 TO STA 1717+60

PROP CURB & GUTTER DETAIL WITH GUARD RAIL

TO APPLY: WB STA 1559+00 TO STA 1696+45

PROP WB I-96

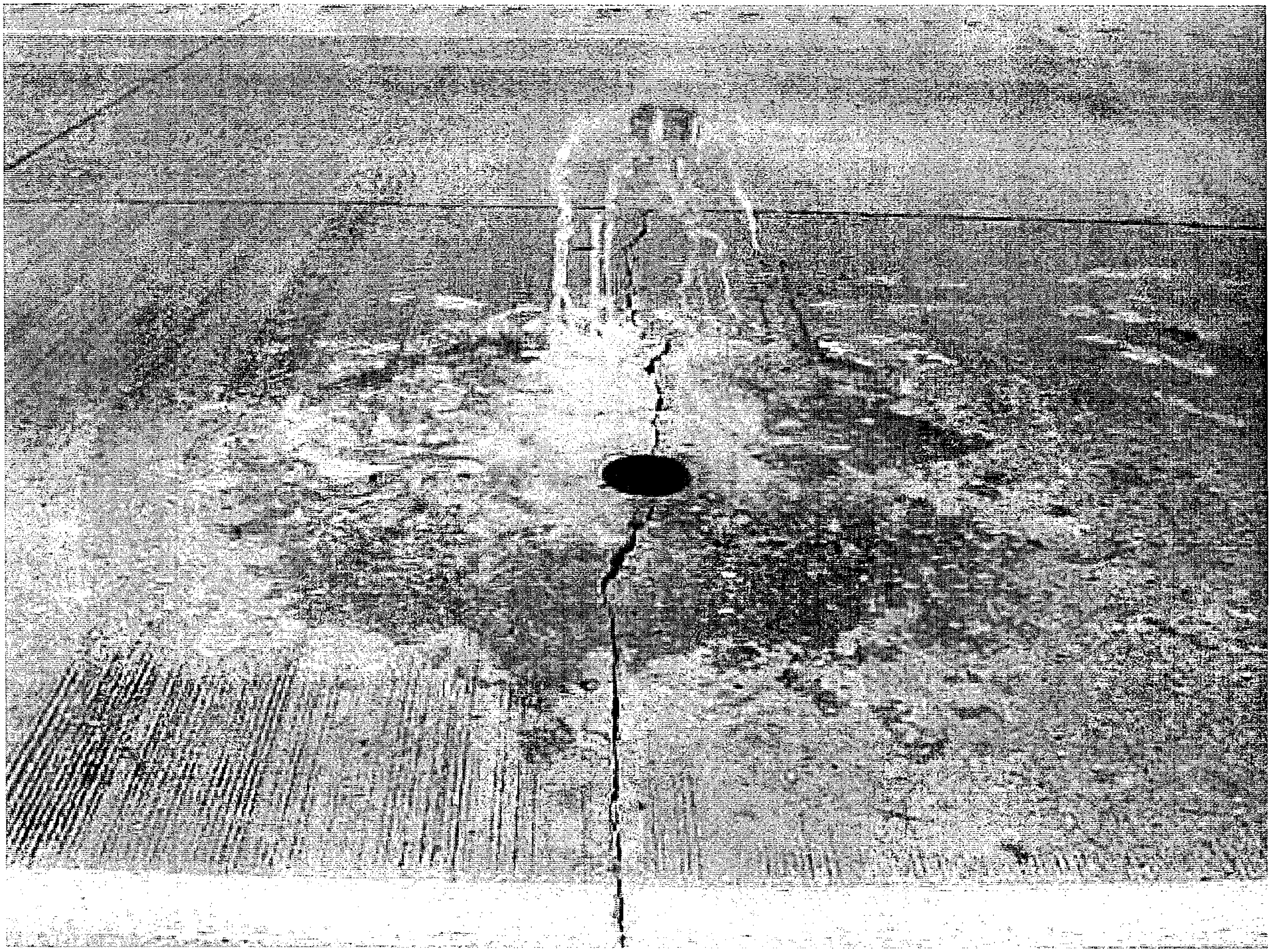
TO APPLY: STA 1637+85 TO STA 1696+06

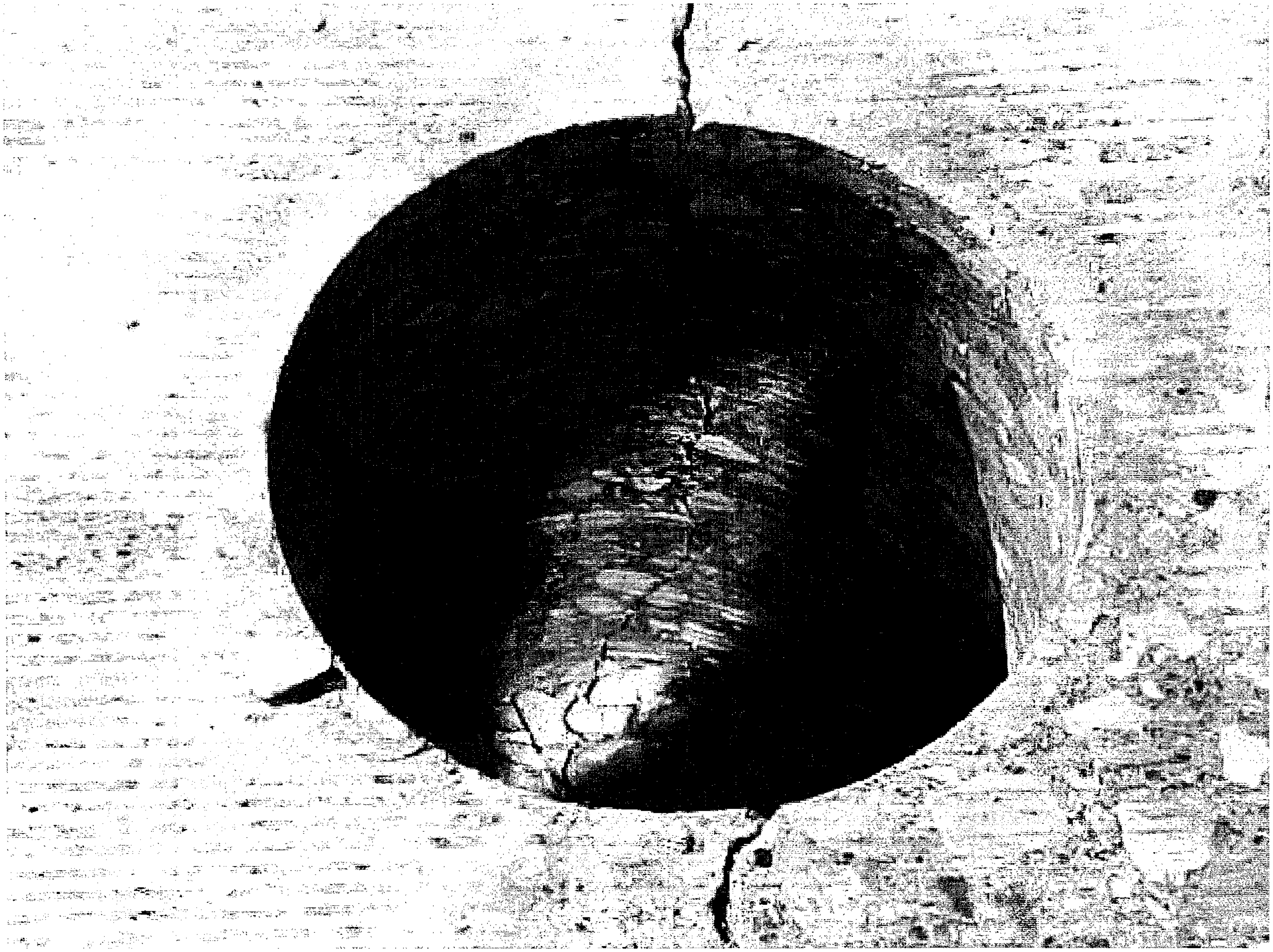
MICHIGAN DEPARTMENT OF TRANSPORTATION

PROP WB I-96 TYPICAL CROSS SECTIONS

DATE	SCALE	CONT. SEC.	JOB NO.	DESIGN UNIT	SHEET NO. R.O.W	SHEET NO. CONST.
02/23/95	1" = 4'	63191	36003	BOVER		6

FILE NAME: 36003TYP.DGN
1 2 3 4





APPENDIX C.

EB US-12 Field Report - Metro Region

JPCP Condition Survey

Roadway and Location: EB US-12 from RR to Howe Rd in the City of Wayne

CS - JN, Construction Date: NH 82061-38063, 7/22/97 - 8/17/98

Project Engineer: Dennis Cooper, Brian Scharboneau

Date: 07/13/00

By: Steven Minton MDOT Soils Engineer
John Boimah MDOT Soils Engineer

			Spalling							
			1-3'	3-6'	6-12'					
Station & Cross Rd.	Lane	Transverse	L	M	H	Longitudinal	Length	spalling	Joint fail	Comments
POB 0+765 (W. of RR bridge)										
0+892	2					X	2.4 m	NO		crack at MH
0+977	2,3	X								no spalling, at MH
1+004	2,3,4	X								no spalling, at MH
Pershing St.										
1+116	All	X	X							
1+222	4					X	300 mm	5%		
William St.										
1+423	3					X	130 mm			corner of slab
1+435	1								X	
Elizabeth St.										
Wayne Rd.										
Washington St.										
Second St.										
Fourth St.										
2+519	2					X	600 mm	NO		
2+534	4	X								no spalling
2+612	3	X	X			X	3.66 m	10%		2 cracks (transv, long)
POE 2+700 (W. of Howe Rd)										

APPENDIX D.

I-75 Detroit Field Report – Metro Region

MDOT
Michigan Department of Transportation

OFFICE MEMORANDUM

DATE: December 8, 2000

TO: Dave Smiley
Pavement Research Engineer
Lansing C & T Division

FROM: Steven Minton
Area Soils Engineer

SUBJECT: Results from Soils Investigation for I-75 in the City of Detroit
CS 82194

The investigation of I-75 in the City of Detroit was a part of a larger investigation of nonreinforced concrete pavements showing early mid panel transverse cracks. The section of I-75 that was investigated was located on NB I-75 between Green Avenue and Waterman Avenue starting at Station 4+226 to Station 4+366.

It is believed that some of the cracks in the test area are attributed to inadequate subsurface conditions. Specifically, very soft silty clay subgrade and missing subbase material. The other cracks within the test area do not appear to be the result of a subsurface problem. The details of the soils investigation are below and the following documents are attached:

- Soil boring log
- Distress map with boring locations
- Gradation test results from subbase material
- Concrete mix designs and compression test results

Results from the Soils Investigation

The soils investigation performed on August 31, 2000 examined two layers of the subsurface condition. The Metro Region Boring Crew sampled the subbase material which was tested for gradation and moisture by Lansing C&T. The boring crew also classified the subgrade material and obtained Standard Penetration Test (SPT) values in the field at the time of the investigation. The pavement cores were retained by Will Hansen for further testing.

There were seven locations investigated within the 140 meter (460') long study area. The borings were concentrated in two separate areas consisting of several cracked slabs at either end of the study area. The borings were done in the right two lanes of the four lane section. Three of the locations were investigated to a depth of 2.7 meters (9') while the other four were to 1.5 meters (5'). Please see the attached distress map for the boring locations.

Area #1

Four borings (1-4) were performed in three consecutive slabs in the inside right lane starting at Station 4+226.40. The first boring location was in a slab that was cracked transversly with no spalling at approximately the middle of the slab. Test Holes No. 2 and 3 were in the next slab which was also cracked transversly with moderate spalling at approximately the middle of the slab. Test Hole No. 4 was in the next slab which did not have any distress.

Test Hole No. 1 was bored to a depth of 2.7 meters (9') while obtaining SPT values. The boring revealed a 1.8 meter (6') deep granular layer beneath the 152 mm (6") open-graded aggregate base. The bottom 762 mm (2.5') of the granular layer was saturated. SPT values through the granular layer ranged from 16 to 42 indicating a moderately compact to compact condition. A very soft gray silty clay layer was below the granular layer, 2.3 meters (7.5') from the top of pavement. SPT values for this layer were zero and a pocket penetrometer did not register any reading for unconfined compressive strength. The clay did not have any field measurable strength.

Test Holes No. 2 and 3 were bored to a depth of 1.5 meters (5'). Both borings showed 460 mm to 490 mm (18" - 19") of sand subbase beneath the 143 mm (5.6") open-graded aggregate base. The subgrade consisted of firm gray very silty clay down to the end of boring at 1.5 meters (5'). The silt content of the clay was very high. The sand subbase was sampled at both of these locations. The samples met Class IIA specifications and the measured water content was 8.5% and 9.3%.

Test Hole No. 4 was also bored to a depth of 2.7 meters (9') with SPT values obtained. The open-graded aggregate base was 128 mm (5") thick and there was 753 mm (30") of sand subbase. The subgrade consisted of 335 mm (13") of the firm gray very silty clay then the very soft gray silty clay to the end of the boring at 2.7 meters (9'). Field tests for the soft clay were consistent with Test Hole No. 1 which showed no measurable strength.

The distress in this area does not appear to be the result of the very soft clay subgrade. Further research into bridge borings show that the very soft gray silty clay layer observed in borings No. 1 and 4 is consistent throughout the corridor. The shallow borings at Test Holes No. 2 and 3 did not penetrate the soft clay layer. While the clay does not have any field measurable strength, it is at such a depth that the pavement would not be influenced. The greater thickness of the sand at Test Holes No. 1 and 4 is most likely the result of undercuts.

It does not appear that the distress observed in Area #1 can be attributed to a subsurface problem. The high silt content of the clay subgrade in Test Holes No. 2 and 3 has the potential for retaining moisture through capillary action which could lead to frost heave. Also, the silty clay is within the 950 mm (3') of the top of pavement, less than the 1.22 meters (4') that is generally considered the frost line. However, the distress in the slab is not consistent with the cracking usually observed due to frost heave.

Area #2

Three borings were performed in the inside right lane and right lane starting at Station 4+346.50. Test Holes No. 5 and 7 were in two consecutive slabs that did not show any distress. Test Hole No. 6 was in a slab in the right lane adjacent to the slab with boring No. 5. The slab had a severely spalled longitudinal crack for almost it's entire length.

Test Holes No. 5 and 7 were bored to a depth of 1.5 meters (5') and showed similar characteristics except for the thickness of the open-graded aggregate base. The open-graded aggregate base thickness for Test Hole No. 5 was 350 mm (13.8") while the thickness for Test Hole No. 7 was 46 mm (1.8"). The sand subbase was 335 mm (13.2") and 396 mm (15.6") for the two locations. The sand subbase was sampled at both of these locations. The samples met Class IIA specifications and the measured water content was 10.6% and 7.4%. The subgrade consisted of low firm to firm grayish-brown silty clay, with silt partings.

The investigation for Test Hole No. 6 revealed immediately the reason for the severely spalled longtidunal cracking present in the slab. The 2.7 meter (9') boring for Test Hole No. 6 did not have any subbase present. 153 mm (6") of open-graded aggregate base was placed directly on the very soft clay subgrade described above.

The distress in this area can confidently be contributed to the absence of subbase material and the very soft clay subgrade directly beneath the aggregate base. The poorly supported concrete pavement could not withstand the loading from the traffic present on I-75.

Pavement History

The section of I-75 that was investigated was located on NB I-75 between Green Avenue and Waterman Avenue starting at station 4+226 to station 4+381. The study area was within the 1998 three mile reconstruction of I-75 from Fort Street to Grand Boulevard in the City of Detroit. The concrete was placed starting on May 18th, 1998 and finished on July 31st, 1998.

The designed cross section consisted of 300 mm of nonreinforced concrete pavement placed on 100 mm Open-graded drainage course with a geotextile separator. The existing sand subbase was retained. Some locations required extra subbase material to attain proper grade and elevation. The initial construction of I-75 in this area contained a large number of subgrade undercuts.

The construction records show that there were four concrete mix designs using the P1 Modified grade of concrete used for the I-75 reconstruction. Compressive tests done at the time of construction indicate the concrete in the test area met the strength requirement of 24 MPa (3500 psi). The average 28 day compressive strength for the concrete placed in the test area was 36.3 MPa (5270 psi). Please see the attached Concrete Mix Design Reports and Compression Test Results for more information.

In conclusion, the distress in the study area is believed to be due to two different reasons. In Area No1, it does not appear the distress is due to a subsurface problem. Frost susceptible soils are present in the subgrade, however, frost heave is not considered likely. Area No.2 revealed the absence of the sand subbase and a very soft clay subgrade which had very little field measurable strength. Construction documents show that the concrete met the strength requirements at the time of construction. The sand subbase, when present, met the gradation requirements for Class IIA and did not have a high moisture content.

METRO MATERIALS & TESTING OFFICE

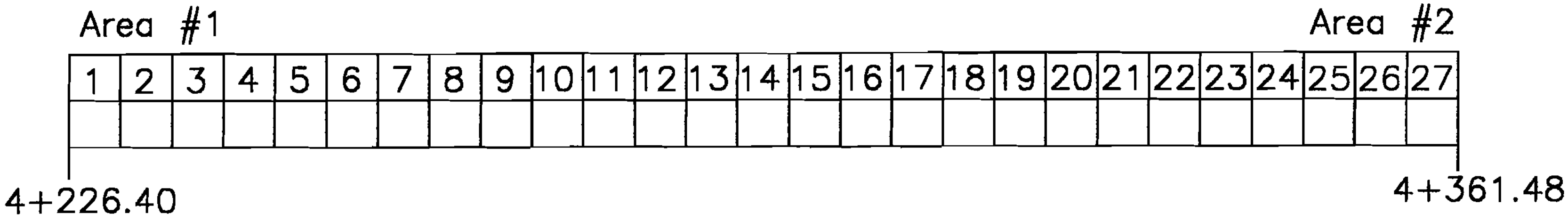
Steven D. Minton

SDM:dvd

Attachments
cc: R. Ostrowski
M. Grazioli
Will Hansen, University of Michigan

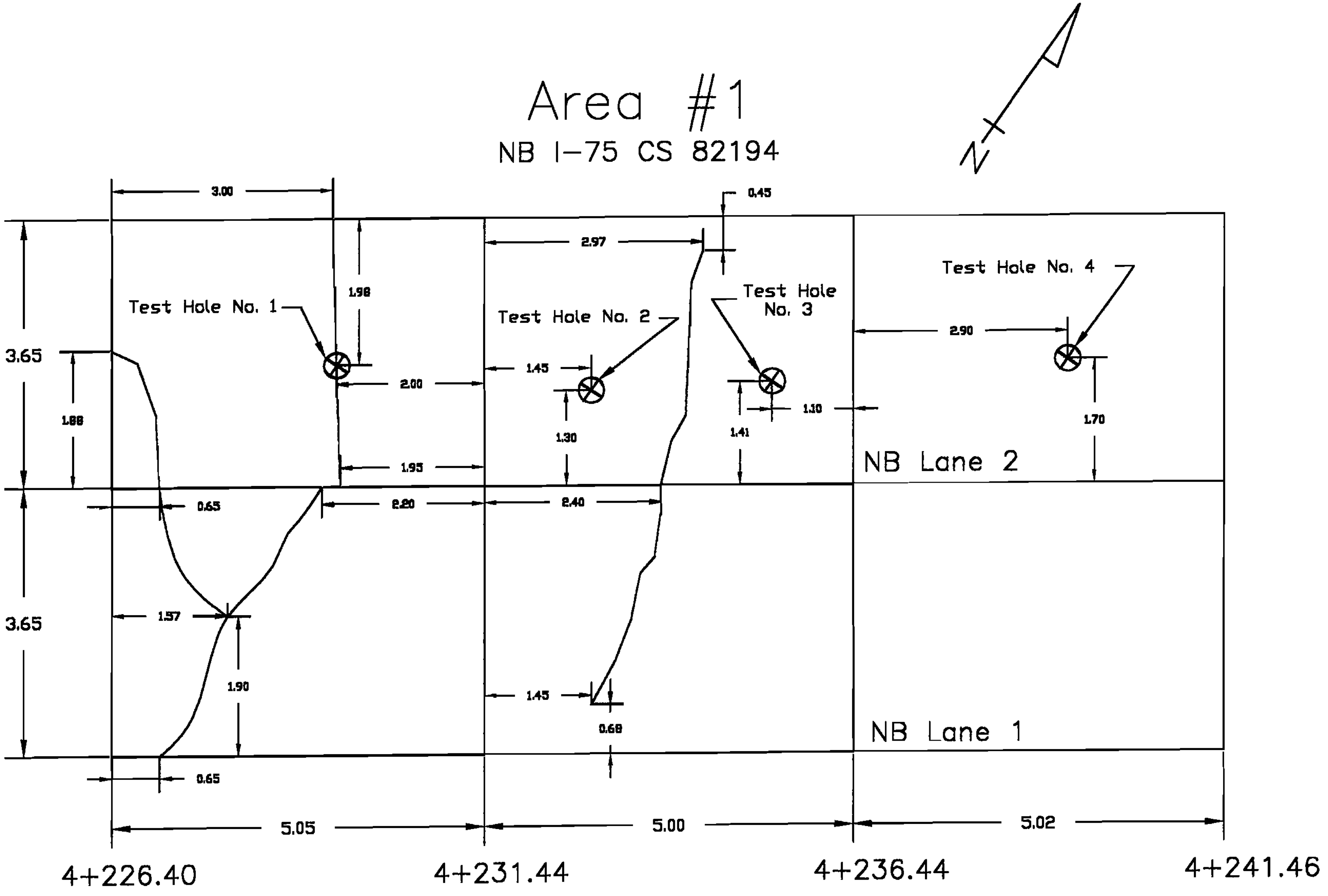
Study Area

NB I–75 CS 82194



Area #1

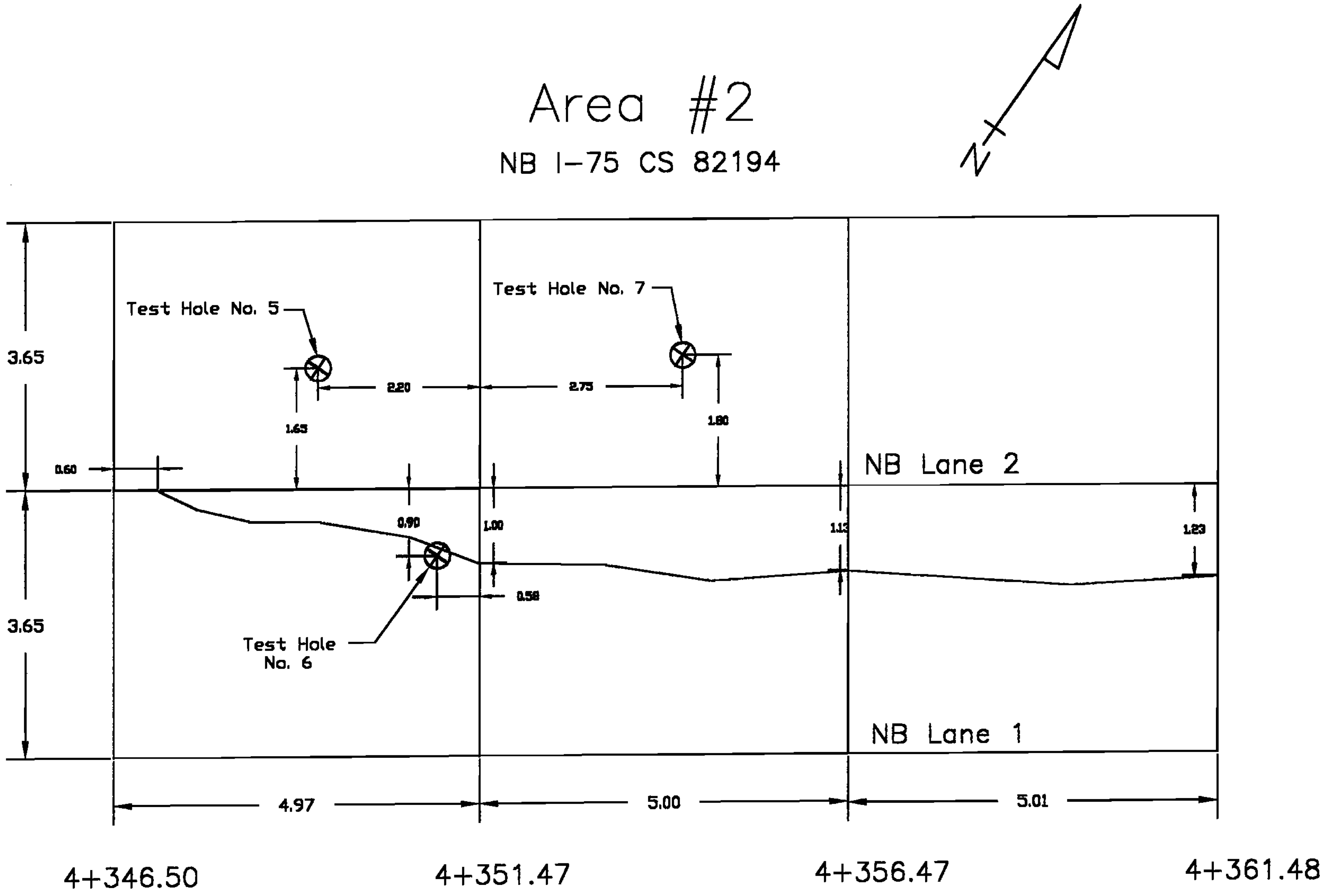
NB I-75 CS 82194



All measurements in metric, not to scale

Area #2

NB I-75 CS 82194



All measurements in metric, not to scale

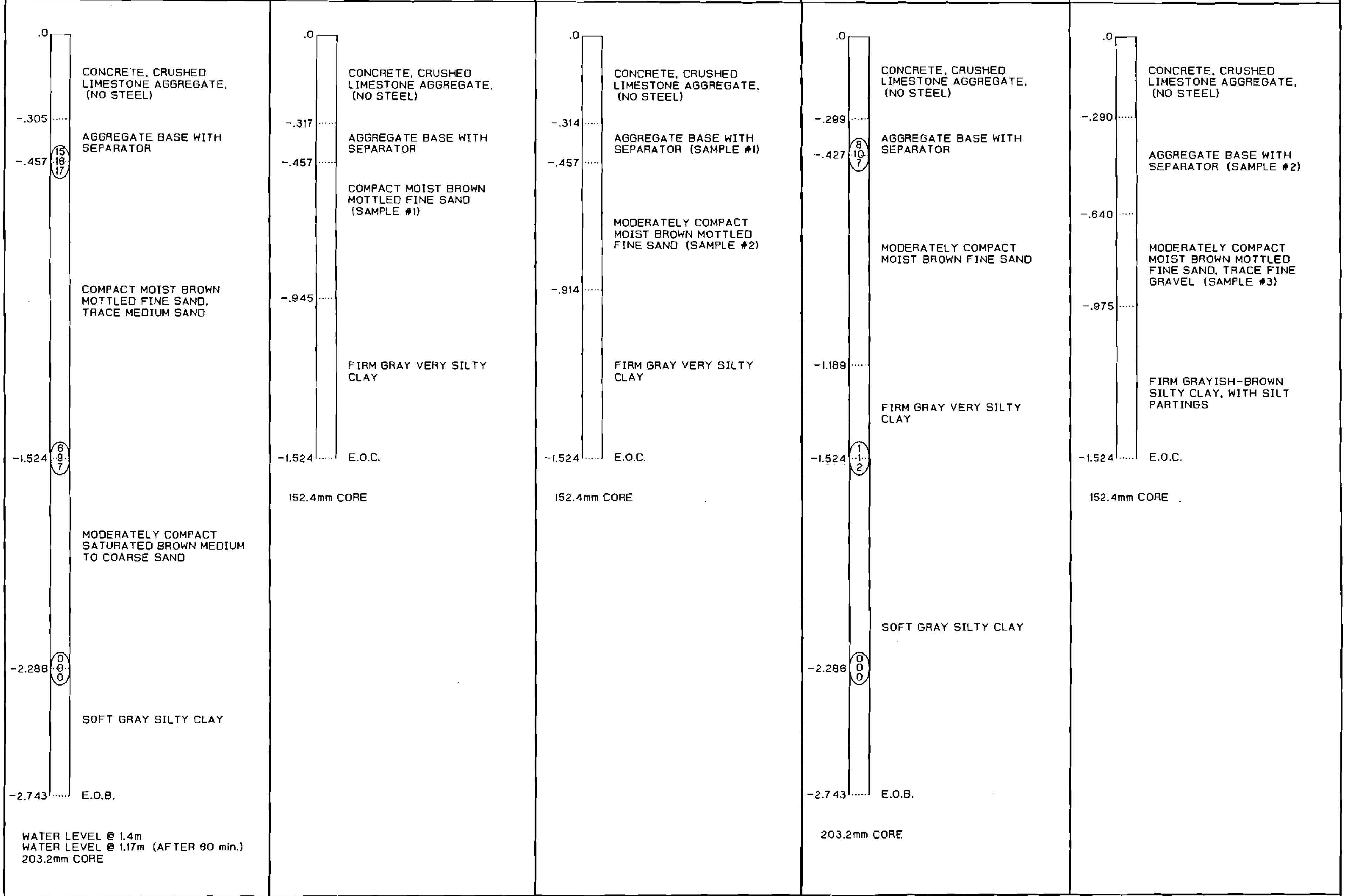
TEST HOLE NO. RESEARCH CORE #1
STATION 4+229.45 5.3m LT. OF RT. E.O.M.
I-75 NORTH BOUND (2nd SLOW LANE)
P.C. 287 CITY OF DETROIT, WAYNE CO.
DATE DRILLED 08-31-00

TEST HOLE NO. RESEARCH CORE #2
STATION 4+232.89 4.95m LT. OF RT. E.O.M.
I-75 NORTH BOUND (2nd SLOW LANE)
P.C. 287 CITY OF DETROIT, WAYNE CO.
DATE DRILLED 08-31-00

TEST HOLE NO. RESEARCH CORE #3
STATION 4+235.34 5.06m LT. OF RT. E.O.M.
I-75 NORTH BOUND (2nd SLOW LANE)
P.C. 287 CITY OF DETROIT, WAYNE CO.
DATE DRILLED 08-31-00

TEST HOLE NO. RESEARCH CORE #4
STATION 4+239.34 5.35m LT. OF RT. E.O.M.
I-75 NORTH BOUND (2nd SLOW LANE)
P.C. 287 CITY OF DETROIT, WAYNE CO.
DATE DRILLED 08-31-00

TEST HOLE NO. RESEARCH CORE #5
STATION 4+349.27 5.30m LT. OF RT. E.O.M.
I-75 NORTH BOUND (2nd SLOW LANE)
P.C. 287 CITY OF DETROIT, WAYNE CO.
DATE DRILLED 08-31-00



Core #1

CONCRETE, CRUSHED LIMESTONE AGGREGATE, (NO STEEL)

AGGREGATE BASE WITH SEPARATOR

COMPACT MOIST BROWN MOTTLED FINE SAND, TRACE MEDIUM SAND

MODERATELY COMPACT SATURATED BROWN MEDIUM TO COARSE SAND

SOFT GRAY SILTY CLAY

E.O.B.

WATER LEVEL @ 1.4m
WATER LEVEL @ 1.17m (AFTER 60 min.)
203.2mm CORE

Core #2

CONCRETE, CRUSHED LIMESTONE AGGREGATE, (NO STEEL)

AGGREGATE BASE WITH SEPARATOR

COMPACT MOIST BROWN MOTTLED FINE SAND (SAMPLE #1)

FIRM GRAY VERY SILTY CLAY

E.O.C.

152.4mm CORE

Core #3

CONCRETE, CRUSHED LIMESTONE AGGREGATE, (NO STEEL)

AGGREGATE BASE WITH SEPARATOR (SAMPLE #1)

MODERATELY COMPACT MOIST BROWN MOTTLED FINE SAND (SAMPLE #2)

FIRM GRAY VERY SILTY CLAY

E.O.C.

152.4mm CORE

Core #4

CONCRETE, CRUSHED LIMESTONE AGGREGATE, (NO STEEL)

AGGREGATE BASE WITH SEPARATOR

MODERATELY COMPACT MOIST BROWN FINE SAND

FIRM GRAY VERY SILTY CLAY

SOFT GRAY SILTY CLAY

E.O.B.

203.2mm CORE

Core #5

CONCRETE, CRUSHED LIMESTONE AGGREGATE, (NO STEEL)

AGGREGATE BASE WITH SEPARATOR (SAMPLE #2)

MODERATELY COMPACT MOIST BROWN MOTTLED FINE SAND, TRACE FINE GRAVEL (SAMPLE #3)

FIRM GRAYISH-BROWN SILTY CLAY, WITH SILT PARTINGS

E.O.C.

152.4mm CORE

NUMBERS IN CIRCLES DENOTE NUMBER OF BLOWS REQUIRED TO DRIVE A 0.051m O.D. X 0.038m I.D. SPLIT SPOON SAMPLER 3 SUCCESSIVE 0.152m INCREMENTS USING A 63.503kg HAMMER FALLING 0.762m.

0 1st 0.152m
0 2nd 0.152m
0 3rd 0.152m

CONSISTENCY DETERMINED BY INSPECTION OF SAMPLES AND BY SOIL RESISTANCE TO PENETRATION BY JET ROD AND CASING OR AUGER.

WATER LEVELS MAY BE INFLUENCED BY RESIDUAL BORING WATER

THE SOIL BORING LOGS REPRESENT POINT INFORMATION. PRESENTATION OF THIS INFORMATION IN NO WAY IMPLIES THAT THE SUBSURFACE CONDITIONS ARE THE SAME AT LOCATIONS OTHER THAN THE EXACT LOCATION OF THE BORING.

SOIL BORING DATA

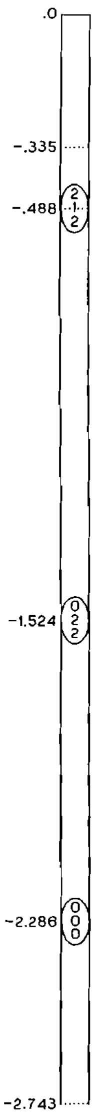
DATE	SCALE	CONT. SEC.	JOB NO.	DESIGN UNIT	SHEET NO.
09/01/2000	1 cm to .19m		52277		1 of 2

CREW CHIEF
J. DAVENPORT

CHECKED BY
SDM

TEST HOLE NO. RESEARCH CORE #6

STATION 4+350.89 2.75m LT. OF RT. E.O.M.
I-75 NORTH BOUND (SLOW LANE)
P.C. 287 CITY OF DETROIT, WAYNE CO.
DATE DRILLED 08-31-00

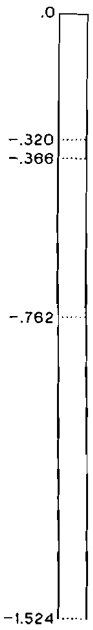


.0	CONCRETE, CRUSHED LIMESTONE AGGREGATE. CRACK RUNS THROUGH CORE, SPALLING ON SURFACE OF CORE (NO STEEL)
-.335	AGGREGATE BASE WITH SEPARATOR
-.488	
-1.524	SOFT GRAY VERY SILTY CLAY, WITH SILT PARTINGS
-2.286	
-2.743	E.O.B.

203.2mm CORE

TEST HOLE NO. RESEARCH CORE #7

STATION 4+354.22 5.45m LT. OF RT. E.O.M.
I-75 NORTH BOUND (2nd SLOW LANE)
P.C. 287 CITY OF DETROIT, WAYNE CO.
DATE DRILLED 08-31-00



.0	CONCRETE, CRUSHED LIMESTONE AGGREGATE (NO STEEL)
-.320	
-.366	AGGREGATE BASE WITH SEPARATOR
	MODERATELY COMPACT MOIST BROWN FINE SAND (SAMPLE #4)
-.762	LOW FIRM GRAYISH-BROWN SILTY CLAY, WITH SILT PARTINGS
-1.524	E.O.B.

152.4mm CORE

(0 0 0) 1st 0.152m
2nd 0.152m
3rd 0.152m

NUMBERS IN CIRCLES DENOTE NUMBER OF BLOWS REQUIRED TO DRIVE A 0.051m O.D. X 0.038m I.D. SPLIT SPOON SAMPLER 3 SUCCESSIVE 0.152m INCREMENTS USING A 63.503kg HAMMER FALLING 0.762m.

CONSISTENCY DETERMINED BY INSPECTION OF SAMPLES AND BY SOIL RESISTANCE TO PENETRATION BY JET ROD AND CASING OR AUGER.

WATER LEVELS MAY BE INFLUENCED BY RESIDUAL BORING WATER

THE SOIL BORING LOGS REPRESENT POINT INFORMATION. PRESENTATION OF THIS INFORMATION IN NO WAY IMPLIES THAT THE SUBSURFACE CONDITIONS ARE THE SAME AT LOCATIONS OTHER THAN THE EXACT LOCATION OF THE BORING.

MICHIGAN DEPARTMENT OF TRANSPORTATION

SOIL BORING DATA

DATE	SCALE	CONT. SEC.	JOB NO.	DESIGN UNIT	SHEET NO.
09/01/2000	1 cm to .19m		52277		2 of 2
CREW CHIEF	J. DAVENPORT			CHECKED BY	

MDOT
Michigan Department of Transportation
1846

REPORT OF TEST

SOIL ANALYSIS

FILE 300

Control Section Identification	
Job No.	52277C
Laboratory No.	00S-244
Date	September 13, 2000

Report of Sample of: Granular Material
Date Sampled: August 31, 2000 Date Received: September 5, 2000
Source of Material: North Bound I-75 (North of Green Avenue, 2nd slow lane)
Sampled From: Sta. 4+349.27, 5.30m Lt. of Rt. E.O.M., Depth; 0.640 to 0.975 meters.
Submitted By: J. Davenport, Engineering Technician
Intended Use: Information Specification: 1996 Standard Specifications

TEST RESULTS

Percent Passing																	**Loss By Wash. %**
Sieve Sizes																	
75 mm	**64 mm**	**50 mm**	**38 mm**	**25 mm**	**19 mm**	**12.5 mm**	**9.5 mm**	**4.75 mm**	**2.36 mm**	**2.00 mm**	**1.18 mm**	**600 µm**	**425 µm**	**300 µm**	**150 µm**	**75 µm**	
			100	95	85	69	61	53	47	47	43	38	34	27	14	10	* 9.4

REMARKS: Except where noted, laboratory testing is performed in accordance with current AASHTO procedures.
Tested for Information.
Sample tested meets specification requirements for Granular Material Class III.
*Sample tested does not meet specification requirements for Granular Material Class II.
As received moisture content = 10.6%.

Louis D. Taylor
Geotechnical Laboratory Engineer

cc: File
Soils Testing
J. Boimah
M. Grazioli
S. Minton

MDOT
Michigan Department of Transportation

1846

REPORT OF TEST

SOIL ANALYSIS

FILE 300

Control Section Identification	
Job No.	52277C
Laboratory No.	00S-245
Date	September 13, 2000

Report of Sample of: Granular Material
Date Sampled: August 31, 2000 Date Received: September 5, 2000
Source of Material: North Bound I-75 (North of Green Avenue, 2nd slow lane)
Sampled From: Sta. 4+354.22, 5.45m Lt. of Rt. E.O.M., Depth; 0.366 to 0.762 meters.
Submitted By: J. Davenport, Engineering Technician
Intended Use: Information Specification: 1996 Standard Specifications

---

TEST RESULTS

Percent Passing																	**Loss**
Sieve Sizes																	**By**
75 mm	**64 mm**	**50 mm**	**38 mm**	**25 mm**	**19 mm**	**12.5 mm**	**9.5 mm**	**4.75 mm**	**2.36 mm**	**2.00 mm**	**1.18 mm**	**600 µm**	**425 µm**	**300 µm**	**150 µm**	**75 µm**	**Wash. %**
				100	99	97	95	90	83	83	77	68	61	49	17	9	* 8.1

REMARKS: Except where noted, laboratory testing is performed in accordance with current AASHTO procedures.
Tested for Information.
Sample tested meets specification requirements for Granular Material Class III.
*Sample tested does not meet specification requirements for Granular Material Class II.
As received moisture content = 7.4%.

Louis D. Taylor
Geotechnical Laboratory Engineer

cc: File
Soils Testing
J. Boimah
M. Grazioli
S. Minton

IV

FILE 300

MDOT
Michigan Department of Transportation

1846

REPORT OF TEST

SOIL ANALYSIS

Control Section Identification	
Job No.	52277C
Laboratory No.	00S-242
Date	September 13, 2000

Report of Sample of: Granular Material

Date Sampled: August 31, 2000 Date Received: September 5, 2000

Source of Material: North Bound I-75 (North of Green Avenue, 2nd slow lane)

Sampled From: Sta. 4+232.89, 4.95m Lt. of Rt. E.O.M., Depth; 0.457 to 0.945 meters.

Submitted By: J. Davenport, Engineering Technician

Intended Use: Information Specification: 1996 Standard Specifications

TEST RESULTS

Percent Passing																	**Loss By Wash. %**
Sieve Sizes																	
75 mm	**64 mm**	**50 mm**	**38 mm**	**25 mm**	**19 mm**	**12.5 mm**	**9.5 mm**	**4.75 mm**	**2.36 mm**	**2.00 mm**	**1.18 mm**	**600 µm**	**425 µm**	**300 µm**	**150 µm**	**75 µm**	
				100	99	93	88	79	69	68	61	51	45	35	15	9	* 8.5

REMARKS: Except where noted, laboratory testing is performed in accordance with current AASHTO procedures.
Tested for Information.
Sample tested meets specification requirements for Granular Material Class III.
*Sample tested does not meet specification requirements for Granular Material Class II.
As received moisture content = 8.5%.

Louis D. Taylor
Geotechnical Laboratory Engineer

cc: File
Soils Testing
J. Boimah
S. Minton
M. Grazioli

FILE 300

MDOT
Michigan Department of Transportation

1846

REPORT OF TEST

SOIL ANALYSIS

Control Section Identification	
Job No.	52277C
Laboratory No.	00S-243
Date	September 13, 2000

Report of Sample of Granular Material
Date Sampled August 31, 2000 Date Received September 5, 2000
Source of Material North Bound I-75 (North of Green Avenue, 2nd slow lane)
Sampled From Sta. 4+235.34, 5.06m Lt. of Rt. E.O.M., Depth; 0.457 to 0.914 meters.
Submitted By J. Davenport, Engineering Technician
Intended Use Information Specification 1996 Standard Specifications

TEST RESULTS

Percent Passing Sieve Sizes 75 mm	64 mm	50 mm	38 mm	25 mm	19 mm	12.5 mm	9.5 mm	4.75 mm	2.36 mm	2.00 mm	1.18 mm	600 µm	425 µm	300 µm	150 µm	75 µm	Loss By Wash. %
			100	99	97	92	89	81	73	72	65	54	47	35	13	8	7.0

REMARKS: Except where noted, laboratory testing is performed in accordance with current AASHTO procedures.
Tested for Information.
Sample tested meets specification requirements for Granular Material Class II and III.
As received moisture content = 9.3%.

Louis D. Taylor
Geotechnical Laboratory Engineer

cc: File
Soils Testing
J. Boimah
M. Grazioli
S. Minton

MDOT
Michigan Department of Transportation

CONCRETE CYLINDER / CORE COMPRESSION TEST RESULTS

REPORT 5
PAGE 1 OF 1

PROJECT ENGINEER	CEDRIC DARGIN	**PROJECT NUMBER**	IM 82194 / 36005A
SPECIFICATION	1996 STANDARD SPECIFICATIONS	**CONTRACTOR**	AJAX PAVING INDUSTRIES
CONCRETE SOURCE	AJAX PAVING INDUSTRIES	**CONSULTANT**	SME (SOILS & MATERIALS ENG.)

CYL. I. D. #	DATE CAST	DATE BROKEN	AGE (DAYS)	SLUMP mm	% AIR	TEMP-CELSIUS		CONC. TYPE	HGT	DIA	H/D	CF	LOAD	PSI	MPa	AVG.	P / F	TEST BY
						CONC.	AIR											
131	05/21/98	06/18/98	28	56	5.7	22	18	P1 #2	300.70	152.80	1.97	1.00	160250	5640	38.89	37.34	P	KLS
132	05/20/98	06/18/98	28	56	5.7	22	18	P1 #2	301.20	151.50	1.99	1.00	145000	5190	35.78			KLS
133	05/20/98	06/18/98	28	69	7.6	27	32	P1 #4	303.90	152.00	2.00	1.00	165250	5880	40.54	39.85	P	KLS
134	05/20/98	06/18/98	28	69	7.6	27	32	P1 #4	302.50	151.40	2.00	1.00	158500	5680	39.16			KLS
135	05/20/98	06/18/98	28	43	7.6	23	23	P1 #2	302.00	152.00	1.99	1.00	117000	4160	28.68	30.10	P	KLS
136	05/20/98	06/18/98	28	43	7.6	23	23	P1 #2	303.20	150.50	2.01	1.00	126000	4570	31.51			KLS
137	05/20/98	06/18/98	28	63	7.9	23	23	P1 #2	303.20	152.30	1.99	1.00	160500	5680	39.16	38.89	P	KLS
138	05/20/98	06/18/98	28	63	7.9	23	23	P1 #2	304.10	151.90	2.00	1.00	157200	5600	38.61			KLS
139	05/20/98	06/18/98	28	*	*	*	*	*	303.20	151.80	2.00	1.00	141500	5040	34.75	35.44	P	KLS
140	05/20/98	06/18/98	28	*	*	*	*	*	304.10	151.00	2.01	1.00	145500	5240	36.13			KLS

LOCATION OF POUR:	**REMARKS:** **METRIC STANDARD 145.04 PSI = 1 MPa**	**LAB: METRO REGION**
131- 138: N. BOUND I-75, STA. 3+800- 4+500	WATER REDUCER= AXIM CATEXOL 1000N	TESTED BY:
	AIR ENTRAIMENT= AXIM CATEXOL 260 AE	
	CYL'S DELIVERED IN ACCORDACNE WITH ASTM C-31 & ASSHTO T-23	REVIEWED BY:
* INFORMATION IS UNAVAILABLE	FIELD TESTS BY: WALLACE HARRIS	Michael L. Cornacchia
	5.6/ 6.0 SACKS OF CEMENT	CC: DISTRICT MATERIALS & TESTING
REVISED REPORT 09/15/98	CONCRETE PLACED: 885.08 / 686.70 M3	PROJECT ENGINEER

Concrete Mix Design Report

Project ID: I-75
Location: Wayne County, Mi
Contractor: Ajax Paving Industries
Proposed Use: Pavement
Mix Number: 75-4

SME Proj. No.: 30228
Date: 3/18/98
Architect/Engineer: MDOT
Control Section: IM 82194
Job Number: 36005 A

(Metric Units)

Class of Concrete: P1
Brand and Type of Cement: La Farge Type I
Source of Sand: New Hudson 2NS MDOT Pit# 63-48 Abs. 1.1%
Source and Type of Coarse Aggregate 1: Specialty Min. Port Inland 4AA MDOT Pit# 75-5 SpGr=2.656, Abs. = 0.7%
Coarse Aggregate 2: Specialty Min. Port Inland 6AAA MDOT Pit# 75-5 SpGr=2.656, Abs.= 1.0%
Coarse Aggregate Dry Loose Unit Weight 1: 1299.1 kg/m3
Dry Loose Unit Weight 2: 1334.3 kg/m3
% Coarse Aggregate: 72.10%
Brand of A.E.A.: Axim Catexol AE 260
Type and Brand of Admixture 1: Axim Catexol 1000N
Admixture 2:
Admixture 3:
Max. Aggregate Size, mm: 51
Spec. 28 Day Strength, Mpa: 24
Min. # Sacks per m^3: 7.3
Min Slump, mm: 0
Max Slump, mm: 76
Air Content, %: 6.5 % +/- 1.50%

Design Quantities Per Cubic Meter

Trial Batch Factor: 0.085

Material	Spec. Gravity SSD	Factor	Mass, kg. SSD	Volume per cubic meter, m3	Trial Batch Weights	
Cement	3.15	3150	334.5	0.1062	28.4	kg
Sand	2.66	2660	855.1	0.3215	72.7	kg
Coarse Agg 1	2.656	2656	381.7	0.1437	32.4	kg
Coarse Agg 2	2.656	2656	579.4	0.2181	49.2	kg
Water	1	1000	145.6	0.1456	12.4	kg
A.E.A *	0	0	98.51	0.0652	28.01	ml
Admix. 1 *	0	0	131.34		37.34	ml
Admix. 2 *	0	0	0.00		0.00	ml
Admix. 3 *	0	0	0.00		0.00	ml
Total			2296.3	1.00		
	Concrete Unit Weight:		2296	kg/m3		

* mass reported as ml per 100 kg cement

30228M4.XLS

Concrete Mix Design Report

Project ID: I-75
SME Proj. No.: 30228
Location: Wayne County, Mi
Date: 3/18/98
Contractor: Ajax Paving Industries
Architect/Engineer: MDOT
Proposed Use: Pavement
Control Section: IM 82194
Mix Number: 75-3
Job Number: 36005 A

(Metric Units)

Class of Concrete: P1

Brand and Type of Cement: La Farge Type I

Source of Sand: New Hudson 2NS MDOT Pit# 63-48 Abs. 1.1%

Source and Type of
Coarse Aggregate 1: Specialty Min. Port Inland 4AA MDOT Pit# 75-5 SpGr=2.656, Abs. = 0.7%
Coarse Aggregate 2: Specialty Min. Port Inland 6AAA MDOT Pit# 75-5 SpGr=2.656, Abs.= 1.0%

Coarse Aggregate
Dry Loose Unit Weight 1: 1299.1 kg/m3
Dry Loose Unit Weight 2: 1334.3 kg/m3

% Coarse Aggregate: 72.10%

Brand of A.E.A.: Axim Catexol AE 260

Type and Brand of
Admixture 1: 0
Admixture 2:
Admixture 3:

Max. Aggregate Size, mm: 51

Spec. 28 Day Strength, Mpa: 24

Min. # Sacks per m^3: 7.3

Min Slump, mm: 0

Max Slump, mm: 76

Air Content, %: 6.5 % +/- 1.50%

Design Quantities Per Cubic Meter

Trial Batch Factor: 0.085

Material	Spec. Gravity SSD	Factor	Mass, kg. SSD	Volume per cubic meter, m3	Trial Batch Weights	
Cement	3.15	3150	334.5	0.1062	28.4	kg
Sand	2.66	2660	839.4	0.3156	71.3	kg
Coarse Agg 1	2.656	2656	381.7	0.1437	32.4	kg
Coarse Agg 2	2.656	2656	579.4	0.2181	49.2	kg
Water	1	1000	151.5	0.1515	12.9	kg
A.E.A *	0	0	98.51	0.0652	28.01	ml
Admix. 1 *	0	0	0.00		0.00	ml
Admix. 2 *	0	0	0.00		0.00	ml
Admix. 3 *	0	0	0.00		0.00	ml
Total			2286.4	1.00		

Concrete Unit Weight: 2286 kg/m3

* mass reported as ml per 100 kg cement

Concrete Mix Design Report

Project ID: I-75
SME Proj. No.: 30228
Location: Wayne County, Mi
Date: 3/18/98
Contractor: Ajax Paving Industries
Architect/Engineer: MDOT
Proposed Use: Pavement
Control Section: IM 82194
Mix Number: 75-2
Job Number: 36005 A

(Metric Units)

Class of Concrete: P1

Brand and Type of Cement: La Fargo Type I

Source of Sand: New Hudson 2NS MDOT Pit# 63-48 Abs. 1.1%

Source and Type of
Coarse Aggregate 1: Specialty Min. Port Inland 4AA MDOT Pit# 75-5 SpGr=2.656, Abs. = 0.7%
Coarse Aggregate 2: Specialty Min. Port Inland 6AAA MDOT Pit# 75-5 SpGr=2.656, Abs. = 1.0%

Coarse Aggregate
Dry Loose Unit Weight 1: 1299.1 kg/m3
Dry Loose Unit Weight 2: 1334.3 kg/m3

% Coarse Aggregate: 72.10%

Brand of A.E.A.: Axim Catexol AE 260

Type and Brand of
Admixture 1: Axim Catexol 1000N
Admixture 2:
Admixture 3:

Max. Aggregate Size, mm: 51

Spec. 28 Day Strength, Mpa: 24

Min. # Sacks per m^3: 7.3

Min Slump, mm: 0

Max Slump, mm: 76

Air Content, %: 6.5 % +/- 1.50%

Design Quantities Per Cubic Meter

Trial Batch Factor: 0.085

Material	Spec. Gravity SSD	Factor	Mass, kg. SSD	Volume per cubic meter, m3	Trial Batch Weights	
Cement	3.15	3150	310.3	0.0985	26.4	kg
Sand	2.66	2660	894.5	0.3363	76.0	kg
Coarse Agg 1	2.656	2656	381.7	0.1437	32.4	kg
Coarse Agg 2	2.656	2656	579.4	0.2181	49.2	kg
Water	1	1000	138.1	0.1381	11.7	kg
A.E.A *	0	0	98.51	0.0652	25.98	ml
Admix. 1 *	0	0	131.34		34.65	ml
Admix. 2 *	0	0	0.00		0.00	ml
Admix. 3 *	0	0	0.00		0.00	ml
Total			2304.1	1.00		

Concrete Unit Weight: 2304 kg/m3

* mass reported as ml per 100 kg cement

30228M2.XLS

4247- MIX DESIGNS

Concrete Mix Design Report

Project ID: I-75
SME Proj. No.: 30228
Location: Wayne County, Mi
Date: 3/18/98
Contractor: Ajax Paving Industries
Architect/Engineer: MDOT
Proposed Use: Pavement
Control Section: IM 82194
Mix Number: 75-1
Job Number: 36005 A

(Metric Units)

Class of Concrete: P1
Brand and Type of Cement: La Farge Type I
Fly Ash/Special Content: US Ash Type F Avon Lake Plant
Source of Sand: New Hudson 2NS MDOT Pit# 63-48 Abs. 1.1%
Source and Type of Course Aggregate 1: Specialty Min. Port Inland 4AA MDOT Pit# 75-5 SpGr=2.656, Abs. - 0.7%
Coarse Aggregate 2: Specialty Min. Port Inland 6AAA MDOT Pit# 75-5 SpGr=2.656, Abs.= 1.0%
Coarse Aggregate Dry Loose Unit Weight 1: 1299.1 kg/m3
Dry Loose Unit Weight 2: 1334.3 kg/m3
% Course Aggregate: 72.10%
Brand of A.E.A.: Axim Catexol AE 260
Type and Brand of Admixture 1: Axim Catexol 1000N
Admixture 2:
Admixture 3:
Max. Aggregate Size, mm: 51
Spec. 28 Day Strength, Mpa: 24
Min. # Sacks per m^3: 7.3
Min Slump, mm: 0
Max Slump, mm: 76
Air Content, %: 6.5 % +/- 1.50%

Design Quantities Per Cubic Meter

Trial Batch Factor: 0.085

Material	Spec. Gravity SSD	Factor	Mass, kg. SSD	Volume per cubic meter, m3	Trial Batch Weights	
Cement	3.15	3150	279.1	0.0886	23.7	kg
Fly Ash	2.213	2213	36.9	0.0167	3.1	kg
Sand	2.66	2660	880.8	0.3311	74.9	kg
Coarse Agg 1	2.656	2656	381.7	0.1437	32.4	kg
Coarse Agg 2	2.656	2656	579.4	0.2181	49.2	kg
Water	1	1000	138.1	0.1381	11.7	kg
A.E.A *	0	0	98.51	0.0652	23.37	ml
Admix. 1 *	0	0	131.34		31.16	ml
Admix. 2 *	0	0	0.00		0.00	ml
Admix. 3 *	0	0	0.00		0.00	ml
Total			2295.9	1.00		
		Concrete Unit Weight:	2292	kg/m3		

* mass reported as ml per 100 kg cement

30228113.XLS

APPENDIX E.

I-96 Livingston County Crack Map

STATE OF MICHIGAN
DEPARTMENT OF STATE HIGHWAYS
TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION

Form 1048

PAVEMENT CONDITION SURVEY

Road Project No. E.B. I-96 NEAR HOWELL

Research Project

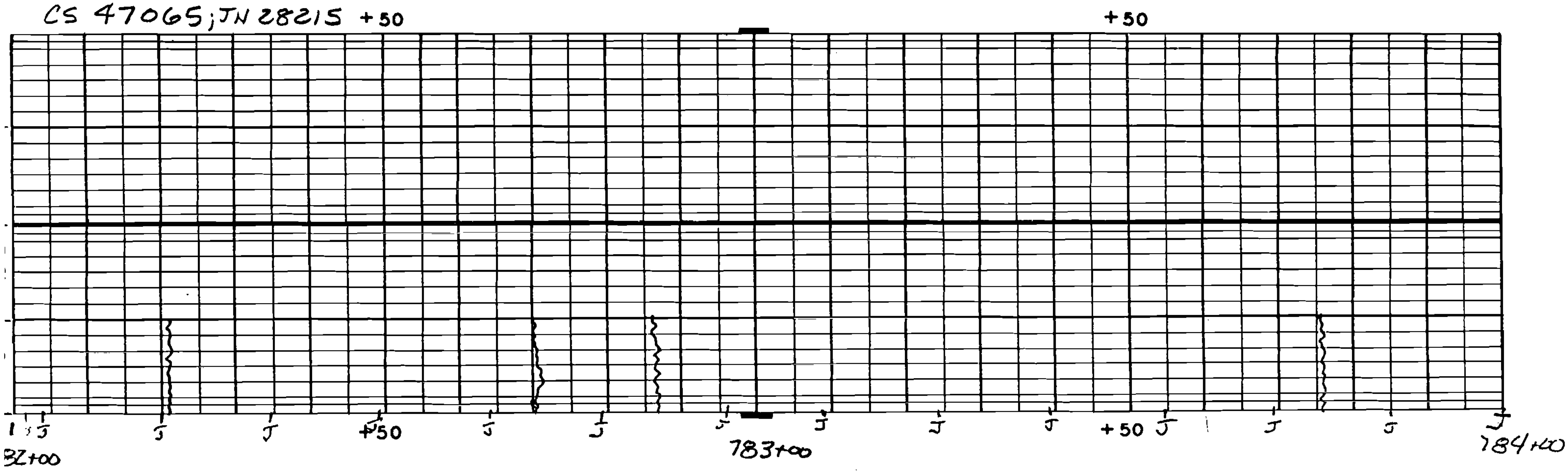
Project Description and Location.

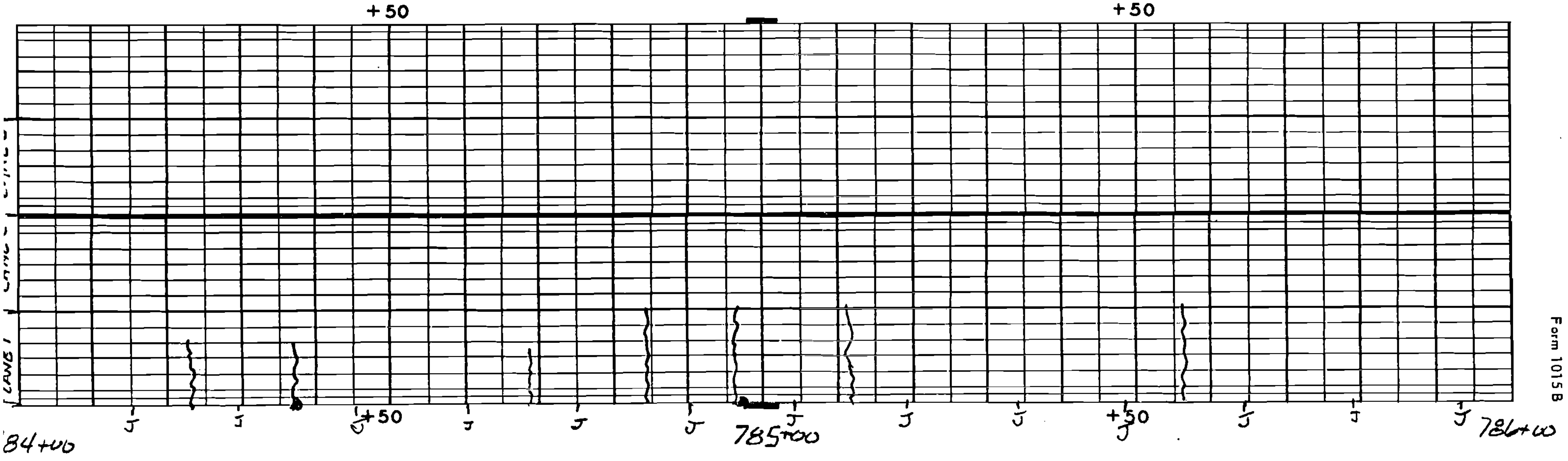
C.S. 47065
JN 28215
POB STA. 782+00
POE STA. 792+00

Survey No.	Survey Date	Surveyed By	Remarks
1	9-27-00	J. SWEENEY	

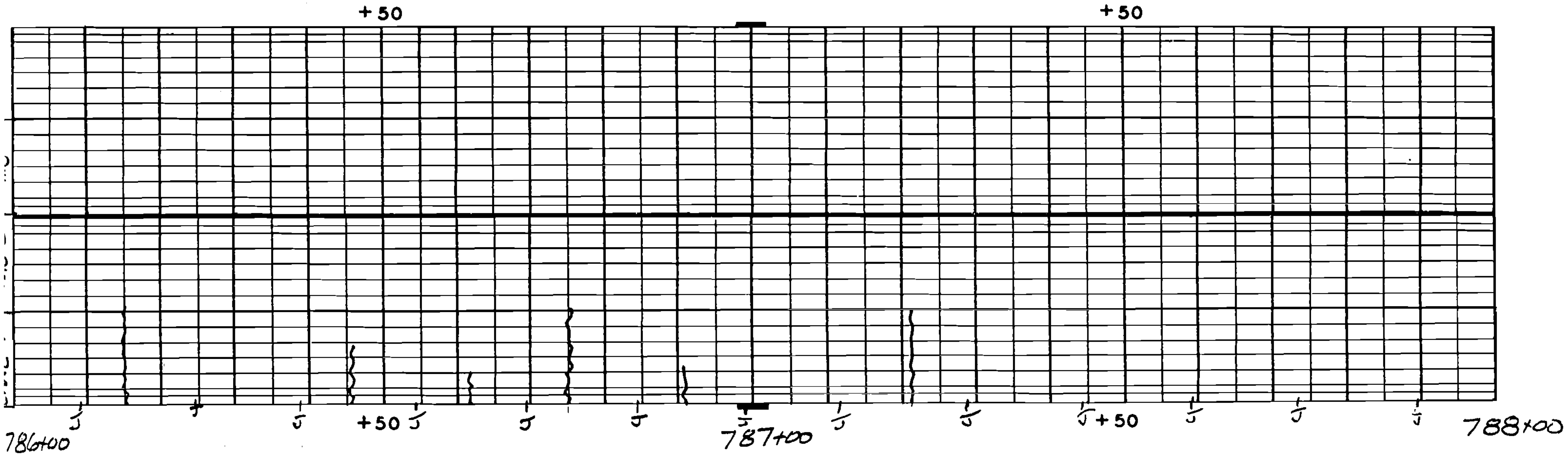
PROJECT NO. EB I-96 SURVEY DATE 9-27-00 SURVEYED BY J. Sweeney PAVEMENT WIDTH 36'

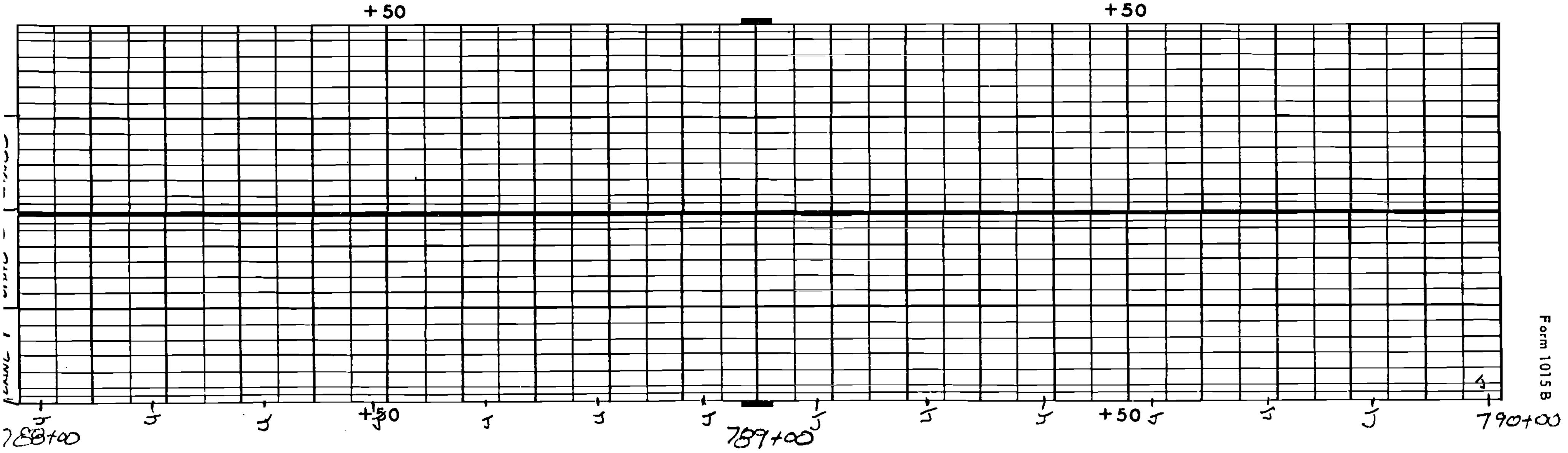
CS 47065; JN 28215





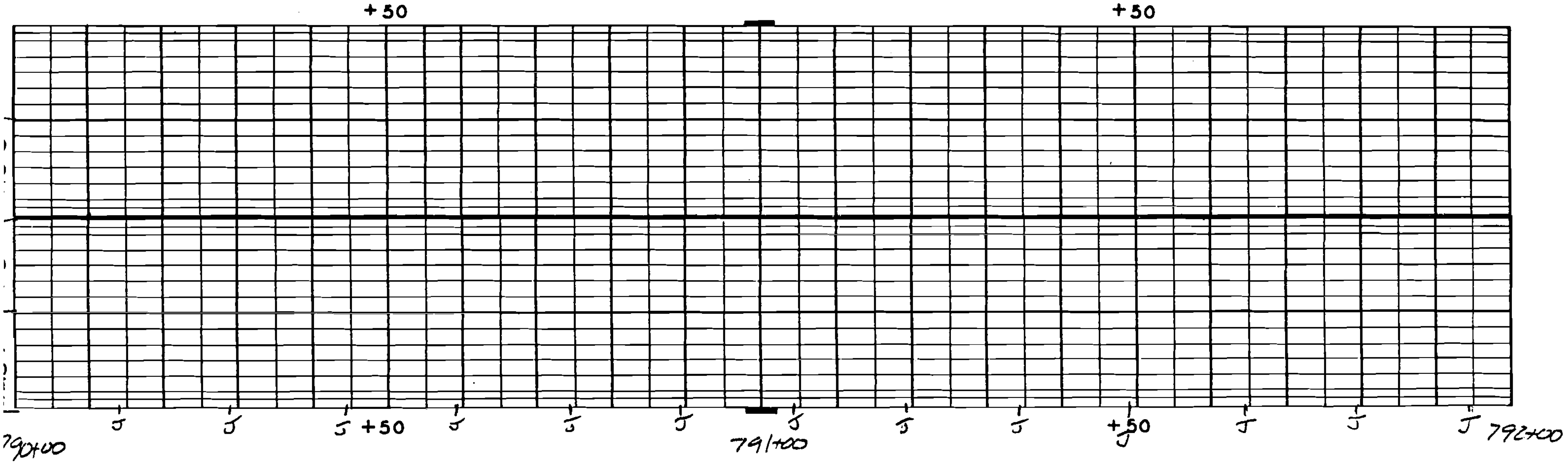
PROJECT NO. ____________ SURVEY DATE ________ SURVEYED BY ____________________ PAVEMENT WIDTH ________

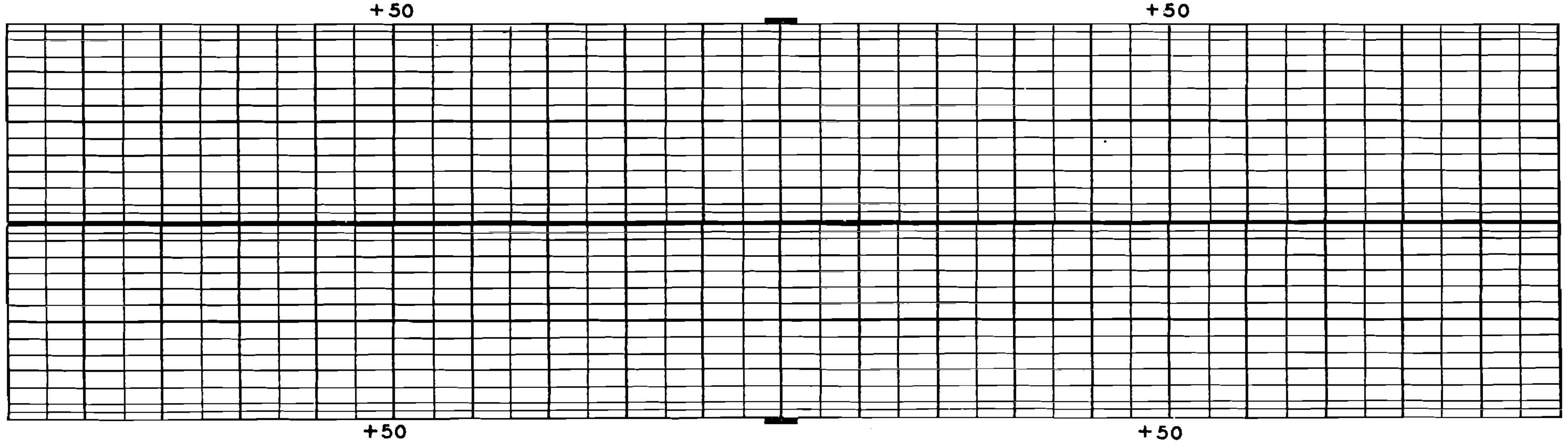




Form 1015 B

PROJECT NO. ____________ SURVEY DATE ________ SURVEYED BY ____________ PAVEMENT WIDTH ________





Form 1015B