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FINAL REPORT

CONSTRUCTION AND PERFORMANCE OF ULTRA-THIN WHITETOPPING IN KANSAS

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16 Abstract

A suburban city street in Kansas was rehabilitated with a 50 mm (2 in.) Portland Cement Concrete (PCC) thin overlay, commonly known as ultra-thin whitetopping (UTW). The construction and performance of this UTW project have been described in this report. The project, constructed in the Spring of 1995, incorporated the following design features: 0.9 m x 0.9 m (3 ft x 3 ft) panels versus 1.2 m x 1.2 m (4 ft x 4 ft) panels, plain versus fiber reinforced concrete, and sealed versus unsealed joints. The project has performed fairly well to date although some test sections needed periodic maintenance and all of the test sections except two have been overlaid as of October 2001.

Experience on this project shows that the UTW overlay can be easily built with conventional equipment and locally available materials. UTW also permits a skid-resistant finish to be applied. Excellent smoothness can also be obtained although the slab thickness is very small. Corner cracking appears to be the most dominant distress type, though it was observed that bond existed between the concrete and the asphalt layers even for the cracked panels. The bond appeared to degrade with time. Joint spacing has a significant effect on performance. The sections with smaller joint spacing appeared to perform better. The performance of the sections with fibers in concrete was inconclusive. Also, joint sealing did not appear to affect the performance.

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ABSTRACT

A suburban city street in Kansas was rehabilitated with a 50 mm (2 in.) Portland Cement Concrete thin overlay, commonly known as ultra-thin whitetopping (UTW). The construction and performance of this UTW project have been described in this report. The project, constructed in the Spring of 1995, incorporated the following design features: 0.9 m x 0.9 m (3 ft x 3 ft) panels versus 1.2 m x 1.2 m (4 ft x 4 ft) panels, plain versus fiber reinforced concrete, and sealed versus unsealed joints. The project has performed fairly well to date although some test sections needed periodic maintenance and all of the test sections except two have been overlaid as of October 2001.

Experience on this project shows that the UTW overlay can be easily built with conventional equipment and locally available materials. UTW also permits a skid-resistant finish to be applied. Excellent smoothness can also be obtained although the slab thickness is very small. Corner cracking appears to be the most dominant distress type, though it was observed that bond existed between the concrete and the asphalt layers even for the cracked panels. The bond appeared to degrade with time. Joint spacing has a significant effect on performance. The sections with smaller joint spacing appeared to perform better. The performance of the sections with fibers in concrete was inconclusive. Also, joint sealing did not appear to affect the performance.

ACKNOWLEDGEMENTS

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Mr. John Wojakowski, P.E., KDOT Concrete Research Engineer was the principal investigator of the project and project monitor on the KSU agreement.

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INTRODUCTION

Good quality Portland Cement Concrete Pavement (PCCP) generally results in a long lasting roadway. However, cities and counties often look at the initial costs when selecting pavement type, and routinely tend to choose asphalt pavements. During resurfacing, an asphalt overlay is often the choice for the same (cost) consideration. However, since asphalt roads need periodic maintenance, traffic control and disruption of traffic become very valid issues especially in the urban areas where the traffic volume increases as the area experiences a growth in population. An alternative rehabilitation technique, thin bonded PCCP overlay, commonly known as ultra-thin whitetopping (UTW), has been advocated by the concrete industry. UTW is a relatively new technique for resurfacing deteriorated asphalt pavements. It involves placing a very thin concrete slab (50 to 100 mm thick) on old asphalt pavements to form bonded composite pavements (*1*). The reduction in thickness is justified by the bond between the concrete overlay and the existing asphalt pavement, close joint spacing, and the use of high quality concrete.

In 1991, a UTW research project constructed in Louisville, Kentucky showed a great deal of potential for long term restoration of asphalt pavements on city streets (*1*). The study was an accelerated test of 50 mm (2 in) and 90 mm (3 ½ in) concrete pavement overlays on an existing asphalt pavement at a landfill access road. After almost two years, over 1,000,000 80 kN (18-kip) Equivalent Single Axle Loads (ESAL's) were applied on the inbound lane and 200,000 80 kN (18-kip) ESAL's were applied on the outbound lane. Knowledge gained from this research project was instrumental in subsequent application of this rehabilitation technique in Colorado, Iowa, Florida, Louisiana, Kansas, Missouri, Minnesota, New Jersey, Oklahoma and Tennessee. Applications include city streets and intersections, and low volume roads. An excellent summary of UTW practices has been provided by Mack et al. (*1*). Recently guidelines have been prepared for UTW pavement repair (*3*).

In 1994, the Kansas Department of Transportation (KDOT) participated in a demonstration project of UTW on a city street using funding from the Intermodal Surface Transportation Efficiency Act (ISTEA) (*4*). The City of Leawood, Kansas, had distressed nine-year-old asphalt on a well-traveled street in need of rehabilitation. The original project was 280 mm (11 inches) of full depth asphalt pavement constructed in 1987. A pre-construction survey showed low severity fatigue cracks on the driving lane with a lesser number of cracks on the passing lane. Rutting was slight, 6 mm (¼ inch) maximum. Some large spalled areas were scattered throughout the project that had four lanes, some medians, and turn lanes. This pavement was milled and resurfaced with 50 mm (two inches) of UTW. Different research features were included in the project. Variables included panel size, use of fibrillated fibers and use of sealant. Table 1 shows the research features and Figure 1 shows the layout of the test sections. Design features included 0.9 m (3-ft) panels in the eastbound lanes, 1.2 m (4-ft) panels in the westbound lanes, a section of plain concrete, and a section of pavement with a silicone joint sealant. A typical section had 1.34 kg/m^3 (3 lbs/yd^3) of fibrillated polypropylene fibers and no joint sealant. Construction was completed in late spring and summer of 1995. The project has been monitored for the last five years.

OBJECTIVE

The main objective of this report is to document the constructibility and performance of UTW in an urban area in Kansas.

PROJECT DESCRIPTION

Location and Layout

The UTW project was built on the 119^{th} street in the City of Leawood, Kansas, a suburb of Kansas City, Mo. between Mission and Roe Avenues. The street is functionally classified as a secondary arterial. The project has 800 m (half mile) of a nominal 50 mm (two-inch) inlay constructed in 1995 and three sections of about 244 m (800 feet) in length in each direction. The project starts at station 50+30.50 and ends at station 76+10.75.

Soils, Climate and Drainage

The soils of the project area are mostly silty clay (unified CL or AASHTO A-6) with low shrink-swell potential. Approximately 70 to 90% pass 0.075 mm (No. 200) sieve. The climate in this region is moderately extreme with hot humid summer (temperatures exceeding 35^0 C (100^0 F)) and cold wet winters (below –20^0 C (0^0 F)). The 30-year annual precipitation is around 810 mm (32 inches). The average number of frost-free days is 176. The referenced data was recorded at the National Weather Services station at Kansas City. Figures 2 and 3 show the monthly low and high temperatures in the project area and Table 2 presents the temperature data. Drainage on the project was provided by the side slopes, curb and gutter and with intermittent inlets to the storm sewers.

Traffic History

The traffic on this project is mostly passenger cars with a small amount of commercial vehicles and almost no heavy trucks. The average annual daily traffic (AADT) in 1994 was 22,400 and projected to be 35,600 in 2014.

MIXTURE DESIGN

Material Sources

The materials sampled at the La Farge Corporation concrete plant were Type I/II Cement, crushed limestone coarse aggregate from Hunt Midwest in Kansas City, Kansas, and river sand from Holliday sand in Kansas City, Kansas. These sources have proven records of producing aggregates suitable for good quality Portland cement concrete. The coarse aggregate was identified as KDOT Class I durability material passing ASTM C666 Procedure B tests. Table 3 shows the gradations for both coarse and fine aggregates. A number of admixtures were used in the mixture. An air entraining agent, a high range water reducing agent, and an accelerator, produced by Master Builders Technology, were used. The material sources are identified in Table 4.

Recommended Mixture Design

For minimum disruption of traffic, access was allowed through the project on the lanes not under construction and a concrete mixture was designed for rapid strength gain. Two concrete mix designs were prepared in September 1993, to explore the potential for developing a concrete mix for use in the UTW. The project specifications required that the design mix achieve a compressive strength of 20.7 MPa (3,000 psi) in 24 hours at 22^0 C (72^0 F) with air content of 6%. A maturity curve was required for the design mix. The mix designs developed are as shown in Table 5. The recommended mixture had a cement factor of 277 kg (611 lbs), water-cement ratio of 0.42, 6 ± 1% air content, slump of 50 ± 25 mm (2 ± 1 inch), and 24-hour compressive strength greater than 20.7 MPa (3,000 psi).

Maturity Data

In addition to measuring the compressive strength of the mixture at regular intervals during the first 24 hours after mixing, maturity data was collected on a 150 mm by 300 mm (6 inch by 12-inch) cylinder cast from the mixture. The relationship between the maturity number and compressive strength for both mixtures A and B (shown in Table 5) is shown graphically in Figure 4. The maturity number is expressed as the number of degrees Celsius hours beginning at the time of mixing of the materials, using zero degrees Celsius as a base. Interpolation of the data for mixture A, recommended for use in the project, indicated that the mixture developed the desired compressive strength of 20.7 MPa (3,000 psi) in a cylinder at a maturity number of approximately 385. This condition occurred in between 15 and 16 hours after mixing the water and the cement. Additional testing with the maturity system was performed to assess the probable effect of climatic conditions on the thin 50 mm (2 in.) overlay that was used in the project. The testing program consisted of casting two 300 mm x 460 mm x 50 mm (12 in. x 18 in. x 2 in.) thick test specimens from mixture A. One specimen was cured under relatively controlled conditions, limiting the ambient air temperatures to between 17^0 C (63^0 F) and 23^0 C (74^0 F). The second specimen was cured under uncontrolled conditions, with ambient air temperatures ranging from 7^0 C (45^0 F) to 21^0 C (70^0 F). Temperature within the test slabs was monitored and recorded on an hourly basis. The results of this comparison are presented in Table 6. It is noted that while the 150 mm x 300 mm (6 in x 12 in) standard cylinder specimen, cured under relatively controlled conditions, developed the 20.7 MPa (3,000 psi) strength in 15 to 16 hours, the 50 mm (2 in) thick slab cured under similar conditions, developed that strength in approximately 17 hours, and the 50 mm (2 in) thick slab cured under uncontrolled and generally lower ambient air temperatures did not develop the 20.7 MPa (3,000 psi) strength until approximately 28 hours after mixing.

CONSTRUCTION REQUIREMENTS

Cold milling of the top 50 mm (two inches) of the existing asphalt pavement was required. Any unsound areas remaining were patched. Traffic signal loop detectors were installed before placement of the overlay. The milled surface was cleaned and then air blasted just before placing the UTW.

Construction traffic was allowed on the pavement after the compressive strength of the slab reached 13.8 MPa (2,000 psi) as determined by the maturity meter from the maturity curves of the mix. All traffic was allowed after the compressive strength reached 20.7 MPa (3,000 psi) by maturity meter and the joints were sealed.

Texturing with a metal comb was done transversely. The joints requiring sealing were dry sawed, air blasted, sand blasted, and given a final air blast before sealing with a silicone sealant. The pavement smoothness was measured with a profilograph and evaluated against the KDOT specifications for roadways with 72 km/h (45 mph) or less posted speed limit.

CONSTRUCTION OF UTW

Construction started in April 1995 in with a cool 13^0 C (56^0 F) weather and very windy condition on the first day. A CMI 350 paver was used to place the concrete 6.7 m (22 ft) wide in a single pass on the milled asphalt surface. Prior to paving, the milled surface was cleaned by a power broom and then air blasted. Texture on the UTW was provided by transverse tining, and curing followed with a curing compound. The second day was also cool with light winds, with the westbound lane being completed. On May 2, the construction began on the eastbound lanes, with somewhat milder weather conditions. Turn bays and gaps were completed in about a week and placed by hand. No particular problems occurred during construction. The project was fully opened to traffic on May 12 after 21 working days. The existing roadway and various construction operations are shown in Photographs 1 to 6.

For regular concrete, the slump averaged 120 mm (4.8 inches) with an air content of 5.5 percent. The average slump and air content for the concrete with fibers were 100 mm (3.96 inch) and 5.6 percent, respectively. Two sets of cylinders 150 mm x 300 mm (6 in. x 12 in.) were made from the regular concrete and the concrete with fibers. The 28-day strength of the regular concrete was 53.3 MPa (7,740 psi), (w/c = 0.37, air content = 4.6%) and the concrete with fibers was 42.0 MPa (6,100 psi), (w/c = 0.37, air=7.3%). The results show that the fresh and hardened concrete had all desirable properties.

Measurements on eight cores showed the UTW thickness to average 70 mm (2.8 inches) with an estimated standard deviation of 10 mm (0.41 inch). The average compressive strength of these two-inch cores was 34.4 MPa (4,990 psi) (corrected for height), with an estimated deviation of 7.2 MPa (1,047 psi) at about 80 days of age.

Construction Cost

The estimated total cost of the project was approximately $219,000 for 13,125 meter squared (15,700 sq. yd.) of concrete placement. This translates to a cost of $16.70 per meter squared ($14.00 per sq. yd). Table 7 shows a breakdown of the estimated cost. Traffic control related costs (work zone traffic control, traffic detectors and pavement markings) accounted for almost 10% of the total project cost.

SPECIAL PAVEMENT INVESTIGATION

FWD Tests

A special investigation was conducted with a Falling Weight Deflectometer (FWD). As mentioned earlier, the existing asphalt concrete pavement was milled to a nominal depth of 50 mm (two inches) and overlaid with a 50 mm (two inch) UTWT. The FWD data was collected prior to milling, after milling and after applying the thin bonded PCCP overlay. The FWD results showed that the deflections increased 0.05 mm to 0.13 mm (2 to 5 mils) after milling. After the UTW was placed deflections were up to one third less than the initial deflections, as would be expected. The FWD data was analyzed using the AASHTO "DARWin" pavement design software program which uses the 1993 AASHTO Design Guide algorithms (*5*). The program was used to obtain effective pavement modulus (E_p) values for the asphalt concrete layer in each lane prior to and after milling. A procedure for analyzing FWD data for the UTW overlaid section does not exist. Therefore, this section was analyzed as both flexible and rigid pavements and the average modulus values were computed.

Table 8 shows the backcalculated Ep values for each lane for all three stages of construction. The individual modulus values calculated for the flexible and concrete pavement were relatively close. As a result, the average of these values should approximate the actual effective modulus value of the composite section incorporating UTW. The UTW application resulted in effective modulus values approximately twice that of the original and milled pavements. However, the composite pavement incorporating UTW was not working as a very rigid slab - the joints appeared to soften the results.

Profilograph Tests

A California-type profilograph was used for smoothness testing before and after placing the UTW. A zero blanking band was use for analyzing the profilograms. Preconstruction smoothness averaged 1290 mm per km (81 inches per mile), which is moderately rough. Profilograph tests after UTW placement showed that all 0.16 km (0.1 mile) sections were smoother than 1030 mm per km (65 inches/mile) required for full pay by KDOT specifications for roadways with 72 km/h (45 mph) or lower posted speed limit. About 80% of the 0.16 km (0.1 mile) sections were smoother than 390 mm per km (25 inches/mile), which is the upper limit for bonus pay.

Bond Tests

Direct pull-off tests were conducted one week and three weeks after the placement of the UTW. A rather coarsely graduated dynamometer was used for the test. For several tests, failure occurred as the indicator hand began to move. Bond strengths therefore were negligible to 69 kPa (10 psi). However, tests in Iowa have showed pull off strengths from 89 kPa to 468 kPa (13 to 68 psi) (*5*).

DISTRESS SURVEY RESULTS

Visual distress surveys have been made since 1995. Interface bond was assessed by “chain dragging.” Figures 5, 6 and 7 show the annual survey results for cracked panels, hollow panels, and spalled panels, respectively. Joint faulting on each section was also assessed as shown in

Figure 8. Tests for “hollowness” were discontinued after 1999 when the test results were judged to be unreliable. At one point, all panels appeared to be “hollow”!

In the fall of 1995, the sections appeared to be performing well. Some cracking was observed on sections 5 and 6. There was minimum amount of spalling and the chain drag results appeared to be very satisfactory indictors of good bond between UTW and the existing asphalt pavement. One area near the toe of the slope on the west end of the westbound lanes had water seeping through the cracks of the inlay near the curb. To help the drainage, the city installed an edge drain for about 61 m (200 feet) at the backside of the curb, which had an outlet into a storm water sewer inlet. Conversation with a local contractor indicated that this area west of the bridge was somewhat swampy.

In the fall of 1996, section 6 had new 36 new concrete panels and section 5 had 26 new concrete panels. The panels were replaced with concrete without fibers. The panels displayed distress presumably due to subgrade problems. Additional distresses were seen in the spring survey results for all sections that were placed on a very windy spring construction day.

The pavement was wet at the time of the spring of 1997 survey making cracks easier to detect, resulting in more cracks being observed in the spring than in the fall. Some distressed panels, about 5% of section 6 around the Tomahawk Creek Bridge and west have been replaced with concrete without fibers. About 5.5% of the panels in section 5 have been replaced.

In the spring of 1998, section 6 had 29% panels cracked and section 2 had only 2% panels cracked. Survey in the fall of 1999 showed 20% of the panels in section 6 cracked. Some of the panels on this section have been replaced. Section 5 also had similar number of panels cracked. Section 2 still had the lowest number of panels cracked. That year, the cracks were classified according to type- corner, longitudinal, transverse and multiple cracks. Corner cracks appeared to be predominant on section 4 and section 6.

In the fall of 2000, some sections with replaced panels had some of those cracked again. Also, some panels were badly cracked. One hundred five (6%) cracked or broken panels had been overlaid with an asphalt patch in section No. 5. Corner cracks occurred, around the joint intersections where four panels meet and the curbs where two panels meet (Figure 9(a)). Corner cracks at the mid-slab locations were also very prominent (Figure 9(b)). Sections 4, 5 and 6 with 1.2 m (4-ft) panels had considerable more distresses than Sections 1, 2 and 3 with 0.9 m (3-ft) panels. The 2000 survey also showed that sections 5 and 6 had the highest number of replaced panels. Forty one percent of the replaced panels had cracked on section 6. Only six percent of the replaced panels had shown cracks for section 5. Sections 3, 4 and 5 appeared to have higher faulting than any other sections (Figure 8).

The final survey was made in October 2001. All sections except Sections 3 and 6 had been overlaid with an asphalt overlay. These overlaid sections were west of the bridge where subsurface water is a problem. There were some spalls, longitudinal and corner cracks on both. The replaced panels were relatively distress free.

COMPARISON OF THE STRUCTURAL PERFORMANCE WITH THE ACPA UTW CALCULATOR

The American Concrete Pavement Association (ACPA) has a website that calculates the load-carrying capacity of a UTW pavement in terms of the total number of trucks (with axle load category (A) or (B); “A” for light and “B” for medium truck categories) that can be carried during its service life (*7*). The calculations are based on a comprehensive mechanistic analysis and correlation to UTW performance data (*7*). In this study, the load carrying capacity of each test was computed using the web site calculator. The ACPA methodology needs 28-day flexural strength as an input. However, in this UTW project, no flexural strength tests were done. Thus available 28-day compressive strength data was converted in to the ASTM C78 (simple beam with third-point loading) flexural strength data using the following correlation (*8*):

$$f_r = k\sqrt{f'_c} \qquad (1)$$

where: f_r = modulus of rupture, psi

f'_c = compressive strength, psi

k = constant usually taken between 8 and 10

The subgrade modulus of reaction is another required input in the ACPA UTW calculator. The backcalculated subgrade resilient modulus values on each section were converted into k values using the AASHTO correlation (*6*):

$$k \text{ (pci)} = M_R / 19.4 \qquad (2)$$

where k is the modulus of subgrade reaction (pci) and M_R is the subgrade resilient modulus (psi).

The backcalculated k values were 105 MPa/m (387 pci), 90 MPa/m (340 pci), 100 MPa/m (366 pci) and 80 MPa/m (304 pci) for the EB inside lane, EB outside lane, WB inside lane and WB outside lane, respectively. However, the ACPA UTW calculator only accepts a maximum subgrade k value of 50 MPa/m (200 pci) (excellent subgrade). The calculated load-carrying capacities of a UTW in terms of trucks that can be carried during its service life are shown Table 9. The 119th street UTW project has carried approximately 64 million vehicles over the last six years. If two percent of this traffic is trucks, and if we take a directional distribution factor of 50% and lane distribution factor of 0.90, then the test sections have carried approximately 576,000 trucks (=64,000,000 * 0.02 * 0.50 * 0.90). If we take into account next year's traffic, the truck traffic would compare very favorably with the truck numbers calculated by the ACPA UTW calculator for the west bound lanes in the "A" traffic category. Most of the sections have been rehabilitated in 2001. That would translate the life of this UTW project into six years.

CONCLUSIONS

A suburban city street in Kansas was rehabilitated with a 50 mm (2 in.) Portland cement concrete overlay, commonly known as ultra-thin whitetopping (UTW). The UTW was constructed in the spring of 1995 and some sections have performed fairly well to date. Some test sections needed periodic maintenance and all sections except two have been overlaid in Fall of 2001. The UTW overlay can be easily built with conventional equipment and locally available materials. UTW permits a skid-resistant finish to be applied. Excellent smoothness was also obtained. The construction cost in 1995 dollars was approximately $16.7 per meter squared ($14.00/sq. yd.). Corner cracking appears to be the most dominant distress type, even though it was observed that bond existed between concrete and asphalt layers. This was also observed for the cracked panels. Joint spacing has a significant effect on performance. The sections with 0.9 m x 0.9 m (3 ft x 3 ft) panels appeared to perform better that those with 1.2 m x 1.2 m (4 ft x 4 ft) panels. The performance of the panels with fibers in concrete was inconclusive. Also, joint sealing did not appear to affect the performance.

REFERENCES

1. Mack, J. W., L. D. Hawbaker and L. W. Cole. Ultra thin White-topping; state of the practice for the thin concrete overlays of asphalt. In *Transportation Research Record 1610,* Transportation Research Board, National Research Council, Washington D.C., 1999, pp. 39 -43.
2. Risser, R. J., S.P. LaHue, G.F. Voigt and J.W. Mack. Ultra-Thin Concrete Overlays on Existing Asphalt Pavements. In *Proceedings of the Sixth International Purdue Conference on Concrete Pavement Design and Rehabilitation, Vol. 2,* Purdue University, West Lafayette, Ind., 1993, pp. 247-254.
3. ___________. UTW pavement repair demonstration. Construction Technology Laboratories, Inc., Skokie, Illinois, 2001.
4. Wojakowski, John. Thin Bonded Concrete Inlay Over Asphalt. *Construction Report.* Kansas Department of Transportation, Topeka, March 1996.
5. Grove, J.D., G. K. Harris and B. J. Skinner. Bond contribution to whitetopping performance on low volume roads. Construction Report, Iowa Highway Research Board Project No. HR-341, Iowa Department of Transportation, January 1993.
6. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, Washington, D.C.,1993.
7. http://www.pavement.com/. Accessed on July 17, 2001.
8. Mehta, P.K. and P. J. Monteiro. Co*ncrete – microstructure, properties and materials. Second edition,* McGraw-Hill, New York, 1993.

TABLE 1: Section Specifications

Section	Joint spacing, ft	3 lbs of fibers	Joint sealed
1	3	yes	no
2	3	no	no
3	3	yes	yes
4	4	yes	no
5	4	no	no
6	4	yes	yes

TABLE 2: Temperature Data

Monthly lowest and highest temperature in the project area, 1995 to 2001

COOPID: 145972
STATION NAME: OLATHE 3 E

CD	UN	YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC
ExtMin	F	1995	2	10	6	30	38	54	55	63	33	33	13	1
ExtMax	F	1995	58	76	85	81	83	90	101	98	91	88	71	68
ExtMin	F	1996	-6	-9	1	24	46	52	56	60	43	32	14	3
ExtMax	F	1996	69	74	73	86	89	91	96	92	87	81	66	67
ExtMin	F	1997	-7	20	19	24	38	55	53	59	52	26	17	19
ExtMax	F	1997	67	65	81	83	89	92	98	95	92	89	66	59
ExtMin	F	1998	8	22	0	35	49	47	63	64	51	41	30	1
ExtMax	F	1998	63	66	76	85	94	93	100	96	98	79	71	69
ExtMin	F	1999	-2	21	23	33	45	50	58	57	39	30	31	11
ExtMax	F	1999	60	73	71	80	83	89	102	97	91	85	80	67
ExtMin	F	2000	11	16	26	29	45	50	59	65	40	27	16	-1
ExtMax	F	2000	62	74	76	83	91	90	97	107	106	88	71	61
ExtMin	F	2001	0	2	18	31	46	58	61					
ExtMax	F	2001	50	59	70	86	88	89	96					

TABLE 3: Gradations for the Aggregates

Aggregate	% Passing on sieve size									
	¾ inch	½ inch	3/8 inch	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	No. 200
Coarse	100	77	48	11	3	2	2	2	2	1.7
	Percent absorption 3.1%; Specific gravity 2.570 (SSD); Dry rodded weight 94.1 pcf									
Fine	-	-	-	100	92	77	52	18	1	0.2
	Percent absorption 0.5%; Specific gravity 2.652 (SSD); Fineness modulus 2.596									

TABLE 4: Material Sources

Cement	Type I / II La Farge Corporation
Coarse Aggregate	KDOT Class I Hunt Midwest
Fine Aggregate	Natural Sand Holliday Sand Company Kansas City, Kansas
Air Entraining Agent	Pave Air Master Builders
Accelerator	Pozzutec 20 Master Builders
High Range Water Reducer	Reobuild Master Builders

TABLE 5: Mixture Design

Material Proportions	Mixture Type (one cubic yard)		
	Mixture A	Mixture B	Recommended Mixture
Cement – type I / II, lbs	611	611	**611**
Coarse Aggregate SSD Crushed Limestone, lbs	1695	1695	**1730**
Fine Aggregate SSD Natural Sand, lbs	1320	1320	**1345**
Total Water, lbs	225	225	**225**
Pave Air, oz/yd	5	5	**5**
Pozzutec, oz/yd	65	0	**65**
Rheobuild, oz/yd	75	75	**43**
Properties of Mixtures			
Fresh Unit Weight	142.5[1]	142.5[1]	**145 ± 1**
Cement Content, sacks/yd	6.5	6.5	**6.5**
Water/Cement ratio	0.37	0.37	**0.42**
Coarse Aggregate, % of total, by absolute volume	57	57	**57**
Slump, inches	3.5 ± 1[2]	3.5 ± 1[2]	**2 ± 1[2]**
Air Content, percent	6.5 ± 1.5	6.5 ± 1.5	**6 ± 1**
Compressive Strength 6’’ x 12’’ Cylinder psi @ 24 hours	3000 +	3000 +	**3000 +**

[1] – At design air content
[2] – After addition of Rheobuild

TABLE 6: Maturity Data on the 2 in. Thick Slabs

Elapsed time, hours	Slab 1		Slab 2	
	Slab maturity, ^{0}C - hr	Ambient air Temperature ^{0}C	Slab maturity, ^{0}C - hr	Ambient air Temperature ^{0}C
1	21	20	18	18
2	42	20	34	18
3	63	21	50	18
4	84	21	66	16
5	106	21	82	14
6	129	20	97	13
7	153	20	111	11
8	177	20	125	10
9	202	19	138	10
10	227	19	151	10
11	251	19	164	9
12	275	19	177	8
13	298	19	189	8
14	321	19	200	7
15	343	18	211	7
16	365	18	221	7
17	386	17	231	9
18	406	18	242	11
19	426	18	254	13
20	446	18	267	15
21	466	19	280	16
22	487	20	294	18
23	508	21	309	19
24	529	21	324	20
25	550	21	340	21
26	571	22	356	20
27	592	23	373	20
28	614	22	391	18
29	636	22	407	16
30	658	21	423	15

TABLE 7: Itemized Initial Construction Costs

Item	Unit	Bid quantity	Unit price ($)	Total Cost ($)
Traffic Safety Control	LS	1	10,000.00	10,000.00
Mobilization	LS	1	2,000.00	2,000.00
Cold Milling	SY	16,814	3.00	50,442.00
Concrete Placement	SY	16,814	4.00	67,256.00
Concrete Curb Removal	LF	89	10.00	890.00
Concrete Curb Type “D”	LF	89	12.00	1,068.00
Concrete Plain	CY	1,066	25.00	26,650.00
Concrete with Fibers	CY	1,736	25.00	43,400.00
Traffic Signal Detectors	LS	1	5,000.00	5,000.00
Pavement Markings	LS	1	5,000.00	5,000.00
TOTAL				**211,706.00**

TABLE 8: Effective Pavement Modulus for the Existing, Milled and UTW Pavements

Section	Before cold milling (psi)	After cold milling (psi)	After UTW (psi)
Westbound - Outside lane	271,349	200,517	495,577
Westbound - Inside lane	309,686	301,811	626,684
Eastbound - Outside lane	257,851	N. A.	530,744
Eastbound - Inside lane	337,326	N. A.	533,146
Effective pavement modulus for UTW Pavement			
Westbound - Outside lane	469,953*	521,200**	495,577#
Westbound - Inside lane	623,075*	628,292**	625,684#
Eastbound - Outside lane	551,016*	510,472**	530,744#
Eastbound - Inside lane	549,698*	516,593**	533,146#

* analyzed as an asphalt pavement
** analyzed as a concrete pavement
# average

TABLE 9: Allowable Number of Trucks per Lane

Section	Allowable Number of Trucks per Lane			
	WB inside lane	WB outside lane	EB inside lane	EB outside lane
Flexural Strength (psi)	753	749	702	695
Load Category A	653,000	651,000	873,000	857,000
Load Category B	345,000	339,000	513,000	506,000

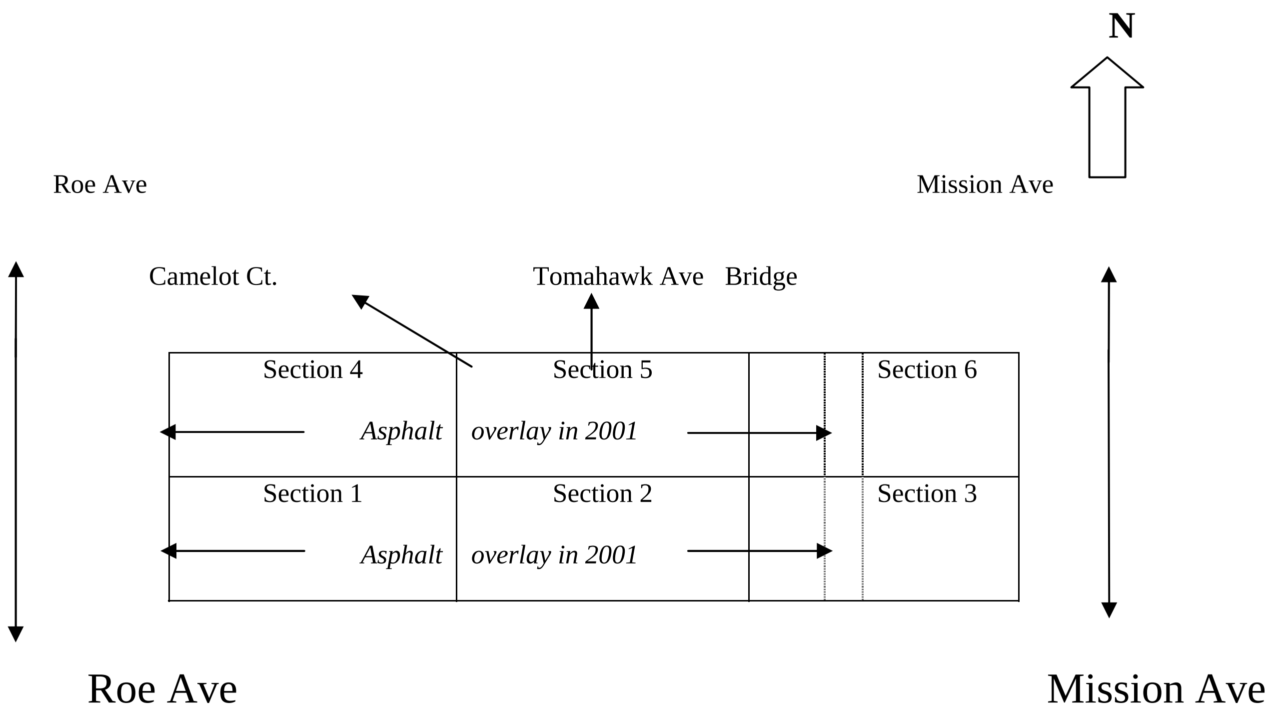


FIGURE 1: Test Section Layout

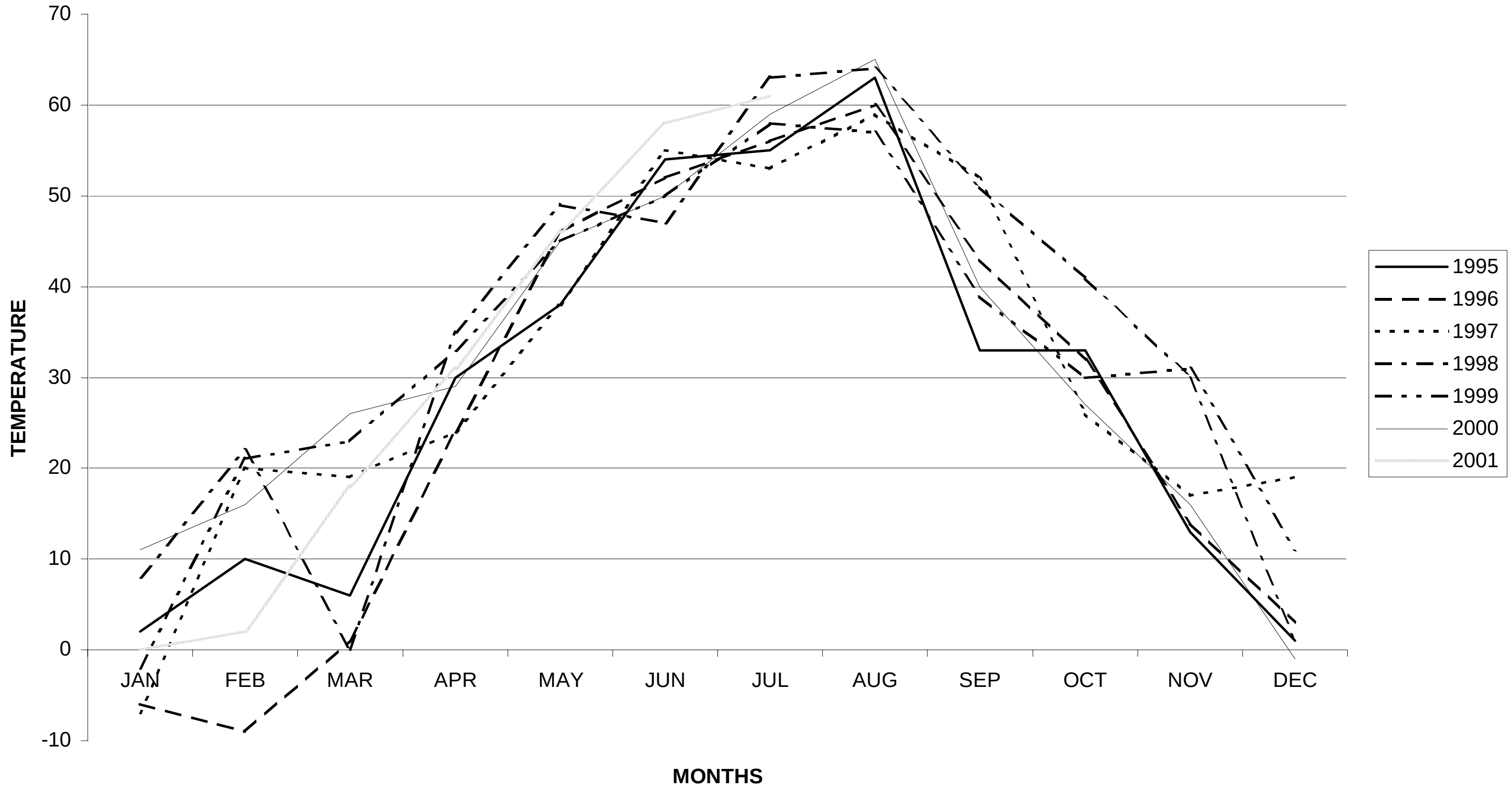


FIGURE 2: Monthly Low Temperature Since 1995

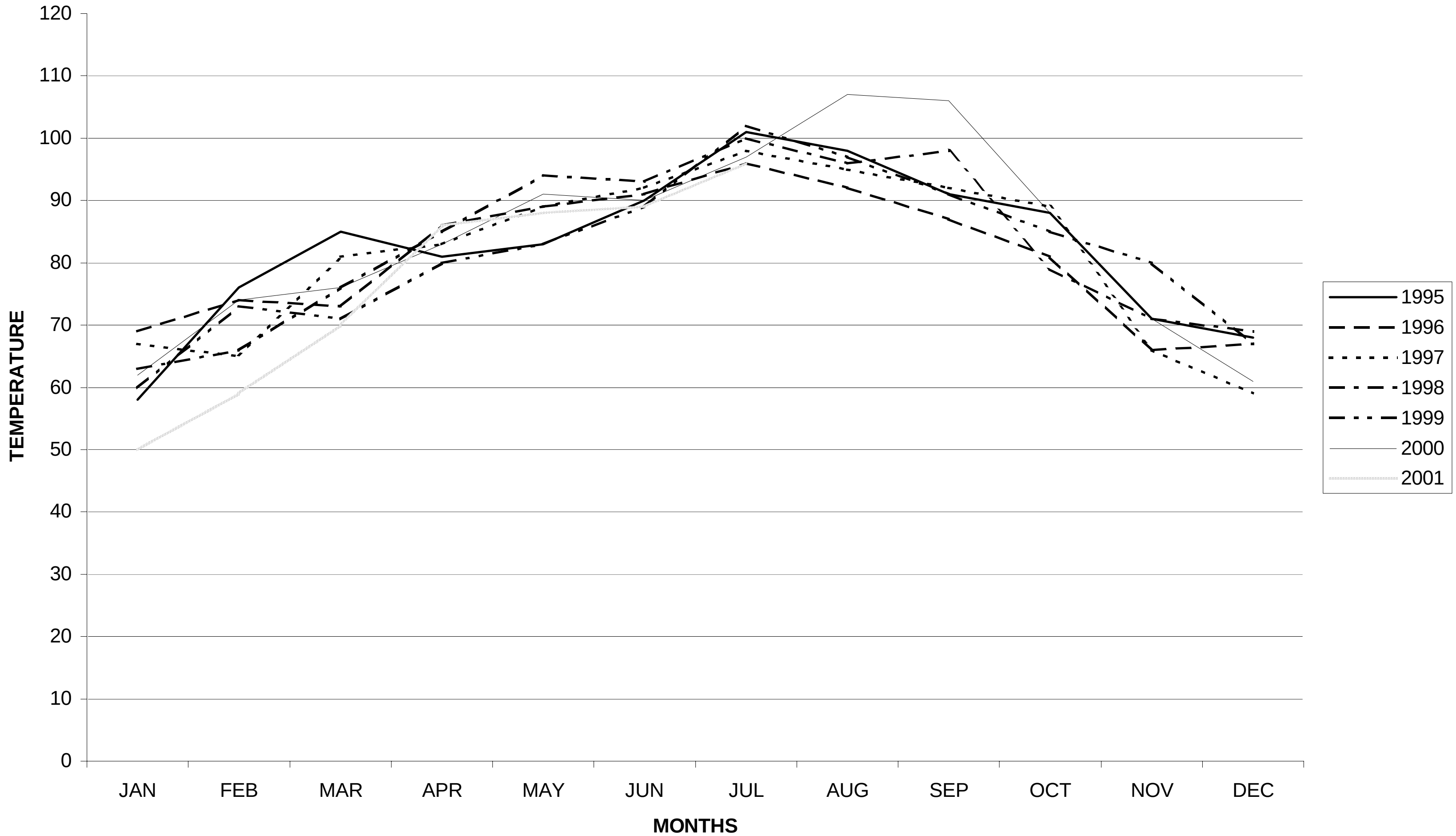


FIGURE 3: Monthly High Temperature Since 1995

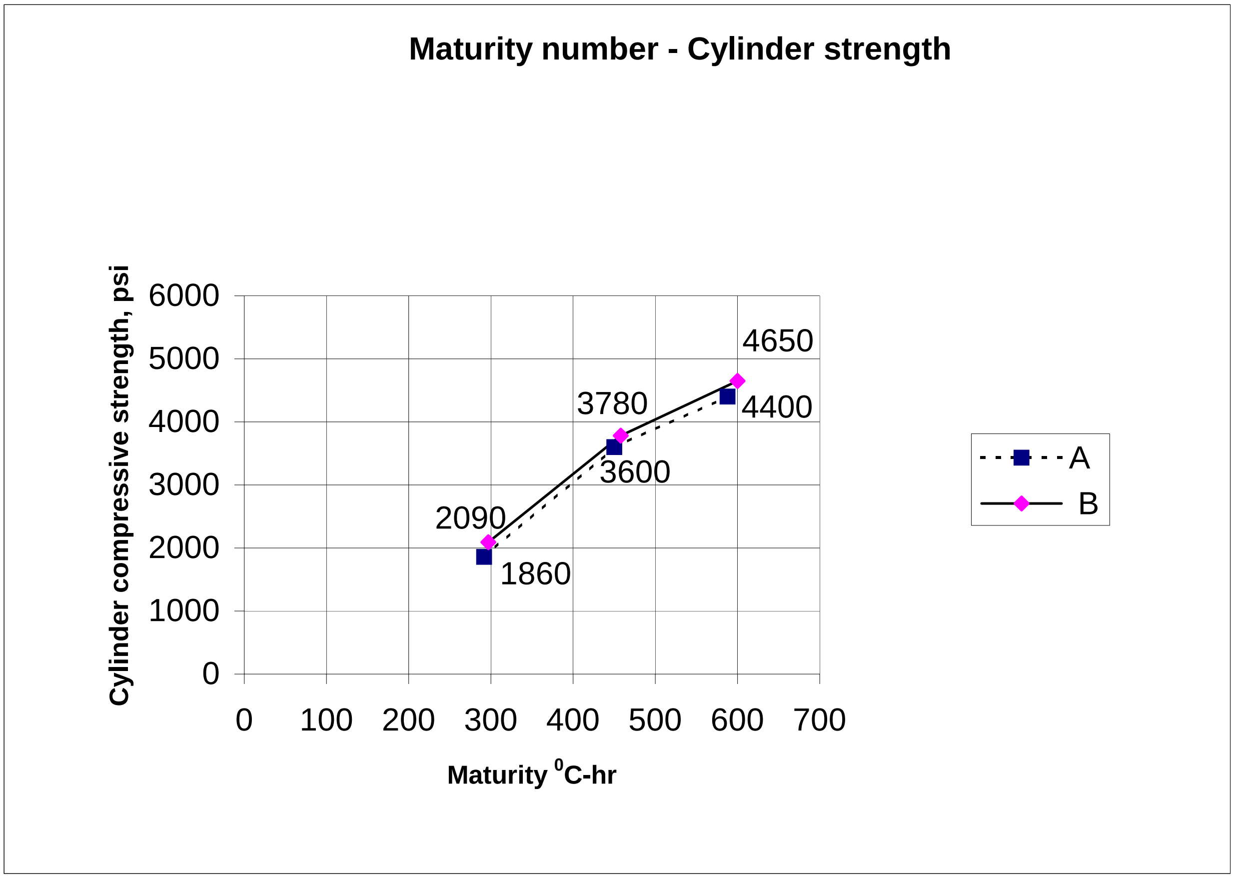


FIGURE 4: Relationship between Maturity Number and Compressive Strength

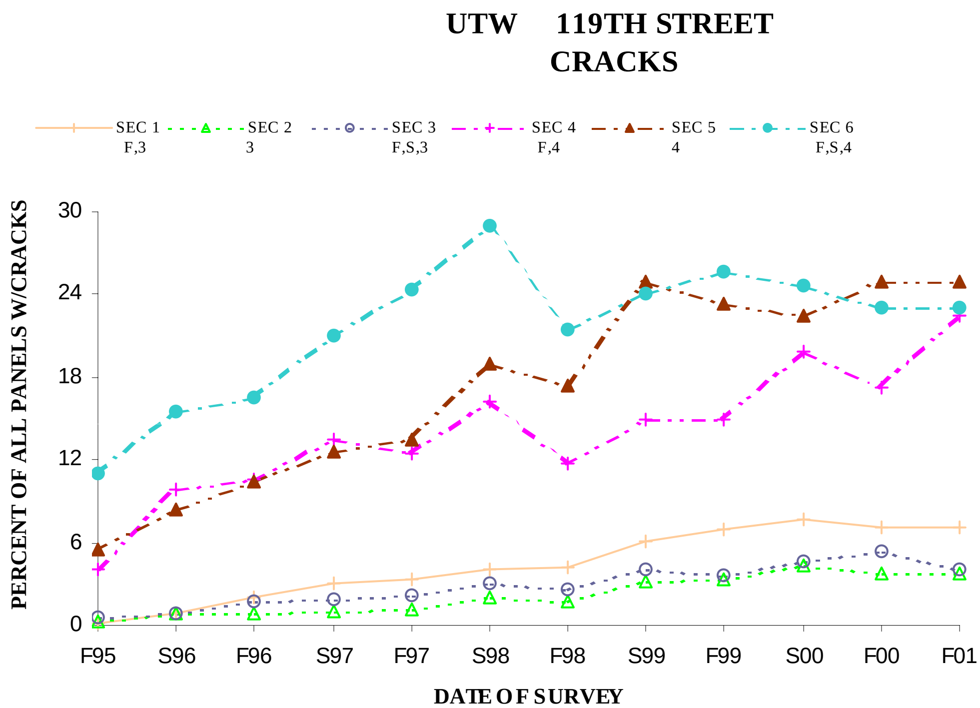


FIGURE 5: Survey Results for Cracked Panels

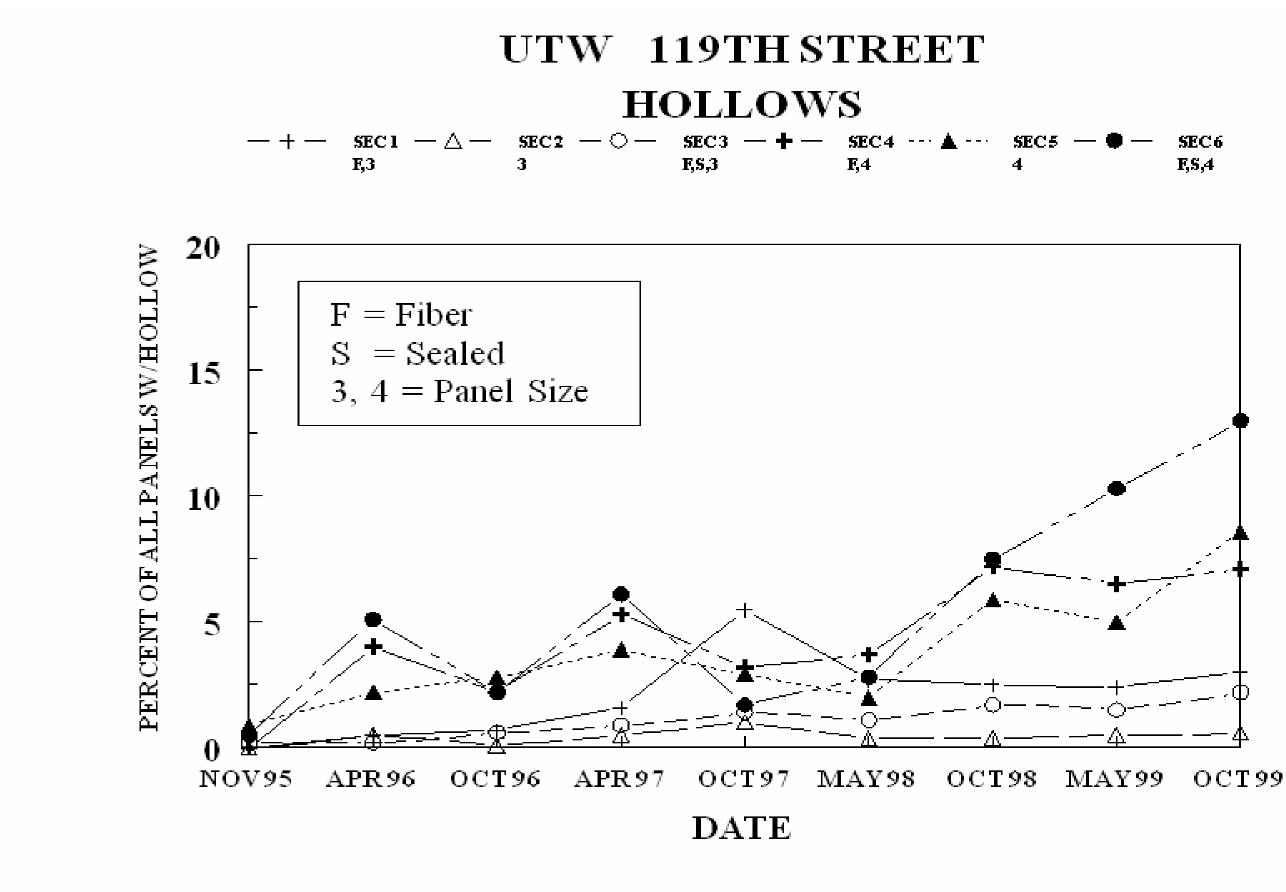


FIGURE 6: Survey Results for Hollow Panels

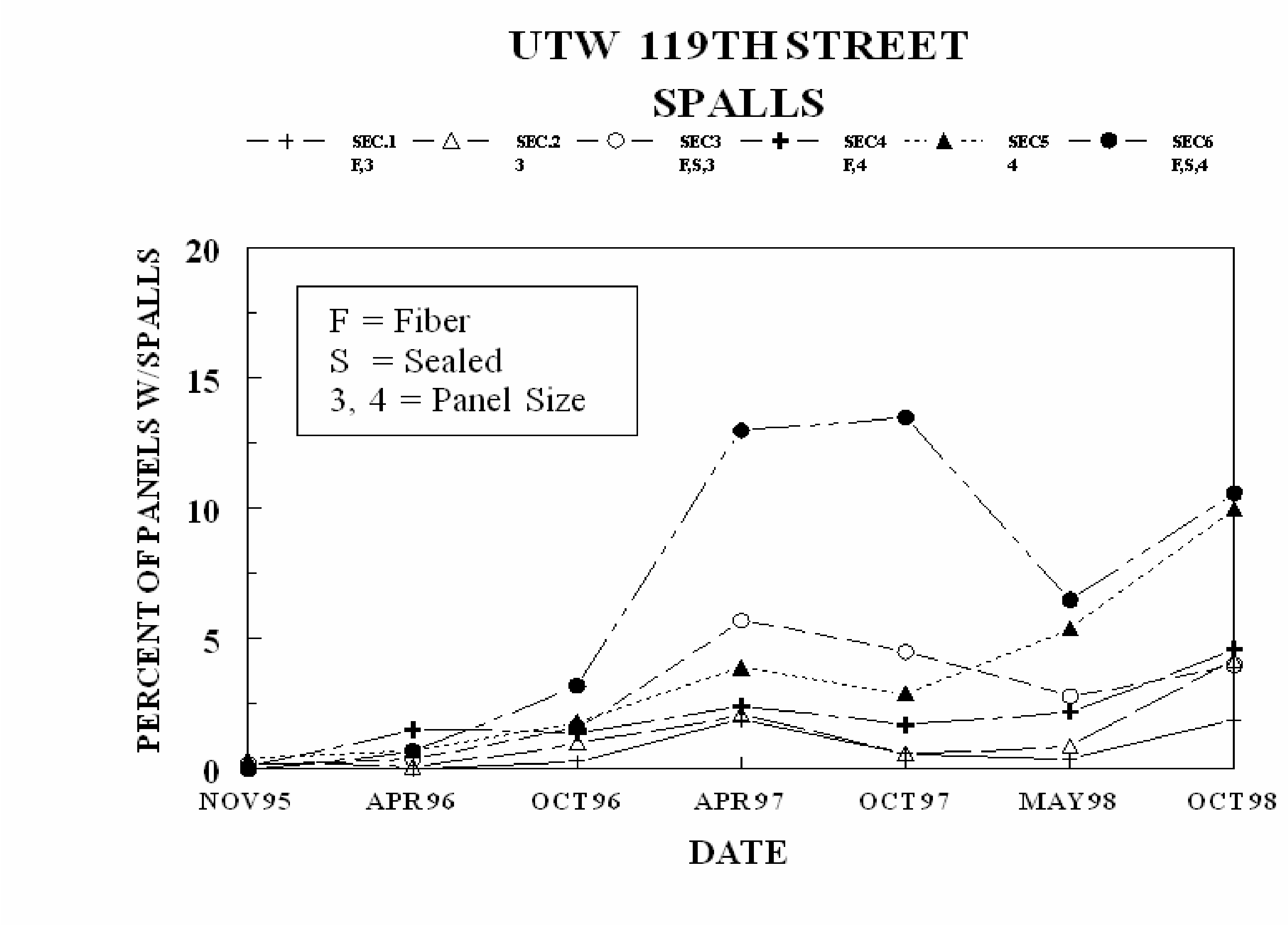


FIGURE 7: Survey Results for Spalled Panels

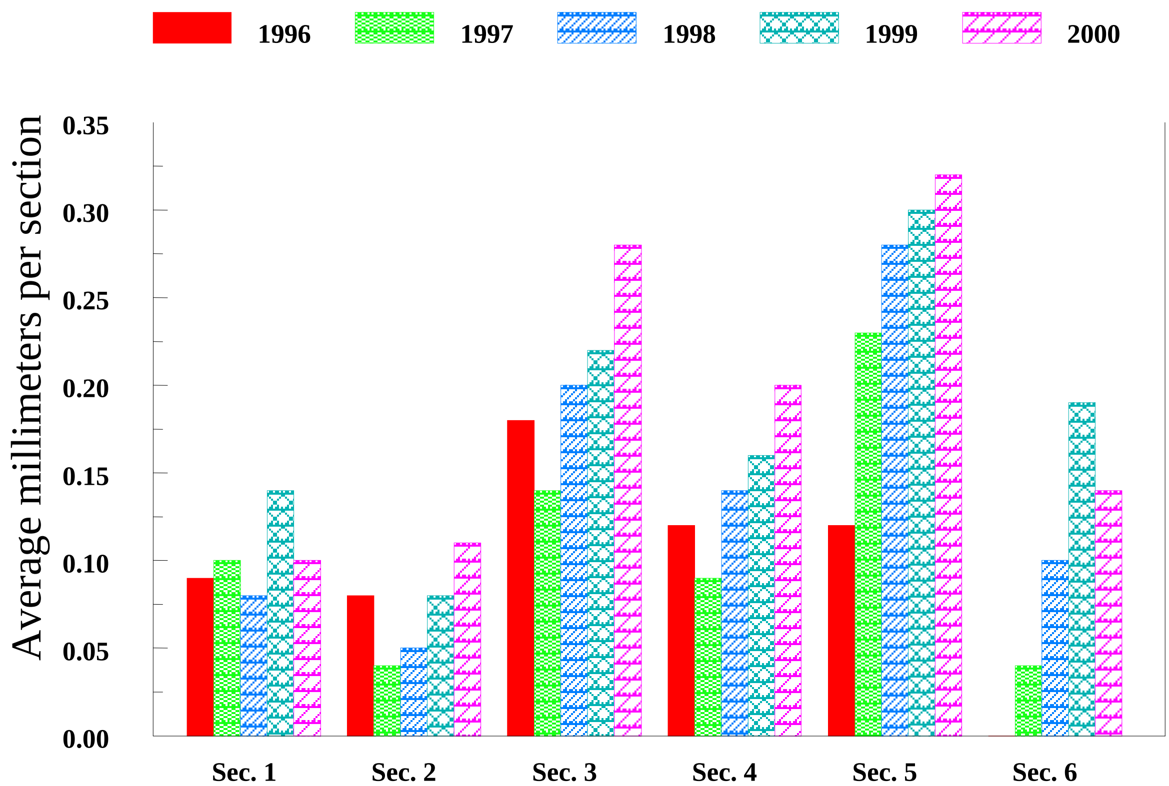
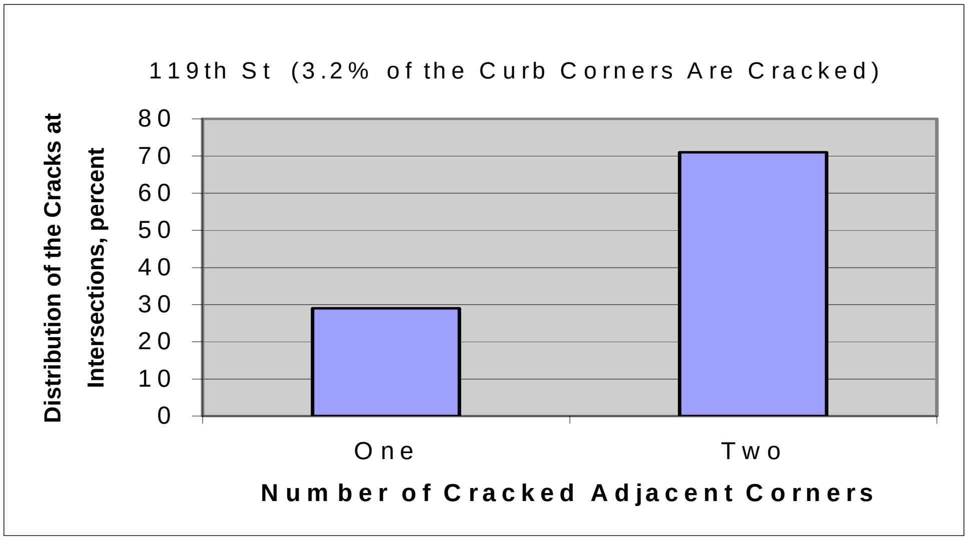
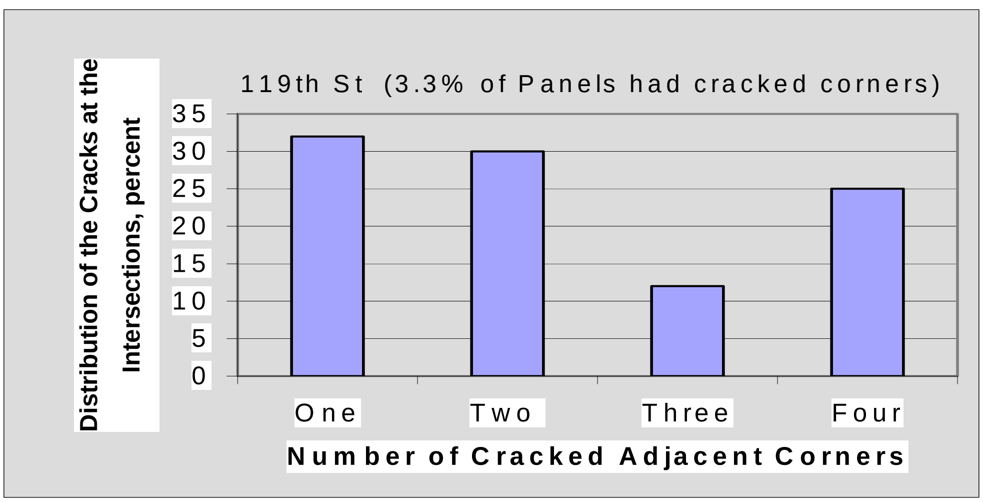


FIGURE 8: Joint Faulting



(a) Corner Cracks at Curb Locations



(b) Corner Cracks at Mid-Slab Locations

FIGURE 9: Occurrence of Corner Cracks

PHOTO 1: Concrete Placement

PHOTO 2: Hand Pour

PHOTO 3: Existing Highway

PHOTO 4: Main Line Paving

PHOTO 5: Surface Finish

PHOTO 6: Turning Lane Paving