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Investigation of chip seal performance under cold climate conditions

Abstract: The chip seal is an economical type of asphalt pavement that was constructed for single- or double-layer aggregate-bitumen. Chip seals are applied to prime-sealed granular pavement surfaces in order to waterproof the surfaces of sub-layers, produce smooth and high-skid-resistance surfaces for vehicles and protect pavements against the detrimental effects of traffic and climate. Unlike bituminous hot mixtures, numerous factors can affect the performance of chip seal. One of the most important factors is climate. The aim of this study is to determine the performance variation of chip-sealed pavements under cold climate conditions. Three different chip-sealed roads in Erzurum, which is in one of the coldest regions of Turkey, were examined with non-destructive tests. Sandpatch, British pendulum, light weight deflectometer and dynamic cone penetrometer tests were performed. Chip-sealed pavements' densities and surface temperature variation and base courses layer's thicknesses were also measured, and deteriorations were observed and scored by researchers. Test results showed that flushing due to the higher temperatures during the short summer season, deficiency of the base course's bearing capacity and moisture in the base course are the most important causes of deterioration in this type of chip seal, and the chip seals in this region must be resealed in <1 year because of the premature deterioration.

Keywords: chip seal; cold climate; environmental condition; *in situ* performance; non-destructive test.

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DOI 10.1515/secm-2014-0041

Received February 13, 2014; accepted January 1, 2015; previously published online May 5, 2015

1 Introduction

Turkey's highway networks consist of chip-sealed road pavements. Chip seal is preferred for economic reasons and ease of application. Chip seals are a type of asphalt pavement that is used in order to have smooth higher friction resistance surfaces. Many factors could affect the performance of these pavements. One of the most important factors is climate. Researchers reported that not only traffic loads affect the pavement's performance, but also climate and some pavement's properties affect pavement performance [1–9]. Tighe [10] stated that both climate and pavement thickness are important to determine a roughness index. Similarly, Haider et al. [11] indicated that climate has an important effect on the formation of reflection cracks and longitudinal cracks on pavements. Service life and performance characteristics of chip seals depend on fine adhesion between binder, aggregates and pavement surface, flexibility characteristics and durability of binder in different climate conditions [5].

The kinds of deterioration of chip seals were allocated into two groups according to the cause of deterioration by Bahia et al. [12]. The first group was related to construction, and deterioration under service should be analyzed in a different way. Flushing, ravelling and fatigue cracks stood out as the most common kind of chip seal deterioration. The second type of deterioration originated from environmental factors. This type of deterioration occurred due to the weakening of the sub-layers, traffic (traffic speed, traffic volume and tire pressure) and climate (temperature, rain and moisture). According to Bahia et al. [12], the binding properties strongly affect the performance of chip seals. Environmental effects, especially climate, are the most important factors when choosing binder type. According to Neaylon et al. [7], the primary factor that causes premature deterioration in chip-sealed pavement is heavy traffic. Heavy traffic is also the main cause of flushing deteriorations, which are common at wheel paths.

In this study, traffic patterns in one of the highest-altitude and coldest regions of Turkey, Erzurum, were monitored for 1

Table 1: Characteristics of test sections.

Properties	Test roads		
	Erzurum-Karagöbek	Tortum-Uzundere	Akşar-Göle
Code of test road	A ₂	B ₂	C ₁
Date of opening to traffic	September 1, 2008	September 1, 2008	July 1, 2009
Length of test section (m)	500	500	800
Traffic direction	Undivided	Undivided	Undivided
Binder type/additive ^a % (A)	160/220 0.1% A	160/220 0.1% A	160/220 0.1% A
Aggregate Tipi	Basalt with andesite	Limestone	Basalt with andesite
Shoulder condition	Yes (insufficient pavement)	Yes (insufficient pavement)	Yes (insufficient pavement)
Layer of prime coat	Yes	Yes	Yes
Aggregate average least dimension (mm)	12.5	9.5	12.5

^aAntistripping agent.

year (C₁) and 2 years (A₂ and B₂). These chip-sealed routes in Erzurum were then assessed based on performance. A and B test roads were resealed before the end of the first year. However, the test results obtained in this region showed that all of the test roads failed earlier than expected. The main causes of this were determined to be high temperatures for a short period of time during summer and using unmodified binder and deficiency of base layers on the roads.

2 Materials and methods

2.1 General characteristics of the test roads

In this study, non-destructive tests were performed at regular intervals on the roads in Erzurum Region of Turkey. Erzurum-Karagöbek test road (A₂) is located on the Erzurum-Tortum State Highway and 500 m in length. The test road is undivided and consists of completely filled cross-sectional profile. Also, the road is double-chip-sealed pavement, and it has base layer of 13.6 cm thickness. Tortum-Uzundere (B₂) test road is located on Tortum-Artvin State Highway and 500 m in length. The test road is also undivided and double-chip-sealed pavement, and the base layer is 12.4 cm. Akşar-Göle test road (C₁) is located on Akşar-Ardahan State Highway and 800 m in length. The test road are undivided and it has a cutting cross-sectional profile also double chip-sealed pavement and base layer with 16 cm thickness. Test section properties and traffic surveying results are given in Tables 1 and 2, respectively.

2.2 Material

A series of laboratory tests were performed in order to determine the aggregate and bitumen's engineering

properties. Accordingly, the aggregate test results and binder characteristics used in test roads are shown in Tables 3 and 4, respectively.

2.3 Method

The study consists of assessments made as a result of *in situ* non-destructive tests and observations. The non-destructive tests were performed on the right lanes, with five different test zones and three different test points for each zone. These points are right wheel path (1), interval of the wheel path (2) and left wheel path (3) (Figure 1). The non-destructive tests included sand patch [19], British pendulum [20], light weight deflectometer (LWD) [21], and dynamic cone penetration (DCP) [22] tests as well as electromagnetic density measurements [23], temperature measurements with thermal camera and base layer thickness measurements with ground-penetrating radar (GPR) [24]. Additionally, each of the test roads were assessed and scored by specialists according to the seven common deterioration types (potholes, ravelling, loss of texture, flushing, polishing, rutting and settlement), and general deterioration indexes were created in relation to the test roads.

Table 2: Traffic surveying results during test years.

AADT	Car	Light truck	Bus	Truck	Semi-trailer truck
Erzurum-Karagöbek (A ₂)					
2532	1654	272	32	526	48
Tortum-Uzundere (B ₂)					
1309	823	158	7	298	23
Akşar-Göle (C ₁)					
2440	1881	197	51	278	33

AADT, annual average daily traffic.

Table 3: Aggregate characteristics used in test roads.

Aggregate tests	Aggregate samples			Limit	Specification
	A ₂	B ₂	C ₁		
Los Angeles abrasion (%)	24.2	20.6	14.8	≤ 35	[13]
Loss of aggregate impact (%)	16.7	9.9	11.4	≤ 12	–
Na ₂ SO ₄ soundness (%)	3.87	2.1	5.3	≤ 12	[14]
Flakiness index (%)	28.7	27.8	21.2	≤ 35	[15]
Bulk specific gravity (g/cm ³)	2.290	2.653	2.426	–	[16]
Water absorption (%)	3.3	1.3	1.9	–	[16]
Nicholson stripping (%)	83.7	33.1	65.0	≤ 50	[17]
Vialit adhesion test (number of dropped aggregate particles)	5	10	7	≤ 10	[18]
pH	6.3	8.3	7.1	–	–

Table 4: Binder properties used in test roads.

Properties	Values	Standard	Limit of TCK specification
Refinery	Batman	–	–
Type of bitumen	160/220	–	–
With/without additive	% 0.1	–	–
Penetration	192	ASTM D5	160/220
Softening point	41.6	ASTM D36	39–47
Loss of heating	2.8	–	Max. 0.8
Penetration after RTFOT	57.6	ASTM	–
Permanent penetration	29.7	D 2872	Min. 43.0
Softening points after RTFOT	57.2		Min. 37.0
Rising of softening point	15.6		Max. 11.0

RTFOT, rolling thin film oven test.

2.3.1 Sand patch test

The pavement surface selected for test must be dry and clean. Before the experiment is performed, the ripped particles on the surface should be swept with a stiff bristle brush. In this experiment, the known volume of sand (25 ml) is poured on the test surface and spread into a circular patch with the surface depressions filled to the level of the peaks. Spreading process is performed with a hard rubber disc 25 mm thick and 60–75 mm diameter. Macro

texture of the pavement selected for test is determined as the volume of sand divided by the area of the patch. The volume of the sand can be calculated using Equation (1), and texture depth can be calculated using Equation (2) [19].

$$V = \frac{\pi d^2 h}{4} \quad (1)$$

$$T_d = \frac{40v}{\pi d^2} \quad (2)$$

where T_d = the texture depth in mm, V = the exact volume of the sand in ml, and d = the average radius of the sand patch in mm.

2.3.2 British pendulum test

British pendulum test (BPT) consists of a movable pendulum and a scale. The tester should be placed in the direction of the traffic flow. The tester is leveled, and the contact path length of the BPT is adjusted according to ASTM D 6951-03, 2006. To start the test, the pendulum arm must be secured in the horizontal position and the pointer rotated anti-clockwise to its stop position parallel with the pendulum arm. Then, the pavement surface and slider are made

Figure 1: Schematic plan of nondestructive test points.

wet with water; the release button is pushed, and the pendulum arm is allowed to swing freely through its arc. Finally, the reading indicated by the pointer is recorded.

2.3.3 Light weight deflectometer

LWD is a lightweight portable tool that is generally performed on low-volume roads and used to determine the mechanical properties of the pavement's layers especially constructed with unbound materials. LWD is a non-destructive test method that can be utilized for quality control and quality assurance processes. In this study, Dynatest 3031 model is used as portable LWD. The device consists of a circular loading plate, freely dropping mass and a transducer which is located center of the loading plate and measures maximum deflections. During the experiment, 20 kg mass is dropped onto a plate with 300 mm diameter. The mass dropped onto the plate generates 15–20 ms loading pulse. The devices that perform in this manner have a loading interval which changes between 1 and 15 kN. Only central deflections are used in this study. Generally, maximum deformation occurs just under the load [9]. The Boussinesq equation is used for backcalculation of the stiffness modulus from applied load and maximum deflections measured on the surface. In the backcalculation process, Poisson's ratio was assumed to be 0.35, and data for thickness of the layers on the route was obtained from GPR. LWD layer modulus can be determined using Equation (3).

$$E_{\text{FWD}} = \left(\frac{K \cdot (1 - V^2) P \cdot r}{\delta_c} \right). \quad (3)$$

where T_d = the texture depth in mm, V = the exact volume of the sand in ml and d = the average radius of the sand patch in mm.

2.3.4 Dynamic cone penetration test

In DCP test a standard penetration force is generated by an 8 kg mass drop from a certain height. DCP is utilized to measure the penetration ratio (PR) of the granular pavement layers. The average penetration of compacted granular layers such as base, sub-base and subgrade corresponding to one drop is called penetration index or penetration ratio. PR is usually used to estimate the strength parameters of *in situ* materials such as California bearing ratio (CBR) and structural number (SN). Pavement structural number can be calculated using Equation (4).

$$SN = \sum_{i=2}^n A_i \times T_i \quad (4)$$

where SN = structural number, A_i = structural layer coefficient for the i th layer from top and T_i = thickness (in inches) of the same layer.

Equation (5) which is developed by Transportation Road Research Laboratory (TRRL) can be used in order to estimate the CBR values from experimental data [9]. In this study Equation (5) is used for calculation of CBR values.

$$\log_{10}(\text{CBR}) = 2.48 - 1057 \log_{10}(\text{penetration rate}) \quad (5)$$

2.3.5 Electromagnetic density measurement

The average of the dielectric constant of pavement layer is a function of the dielectric constant of each component that forms the volume of the materials. Hot mix asphalt consists of air, aggregate and asphalt binder components. The dielectric constant of air is almost 1. But the dielectric constants of asphalt and aggregate are between 5 and 6. The logic of the operation of the device is as follows:

Once the pavement compresses, the air content in the materials reduce; therefore, the dielectric constant of the material increases. Electromagnetic density and moisture tester measures the material's density due to the logic of the operation of the device. The change in density of the seal coat due to traffic is determined by Troxler brand electromagnetic density tester [9, 23].

2.3.6 Temperature measurement

Thermal camera is a device that measures the temperature of the surface with infrared rays. Particularly, seasonal changes in temperature cause a large change of bitumen's rheological properties which is a viscoelastic material. Therefore, the changes that occurred affect the performance of the pavement and can cause a variety of deterioration. Within the scope of this study, the temperature of the surface selected for investigation was determined by using of FLIR brand thermal camera [9].

2.3.7 Based layer thickness measurements with ground-penetrating radar

GPR is one of the most widely used techniques for determination of the thicknesses of the pavement's layer. In this technique, short electromagnetic wave signals are sent into pavements via an antenna from the measuring

[illegible]

where $\sum TP$ =total number of test points, TS =number of test sections, TP =number of test points, DS =number of one test point deterioration score, NDT =number of deterioration types, NP =number of “poor condition” points, NM =number of “medium condition” points and $ScDI$ =score of deterioration index.

Accordingly, if the deterioration index score of test road is between 100 and 60 or between 60 and 50 or between 50 and 0, road condition has been assessed as good, middle and poor, respectively. Variation of deterioration indexes of test roads is shown in Figure 2.

As a result, the final condition of the A_2 was found to be better than B_2 and C_1 . Besides, all of the test roads’ performances significantly decreased at the end of the test period.

3.2 Non-destructive test results

Non-destructive tests were performed on three different points for each zone, for a total of five different sections on three different test roads. The test results

Figure 2: General deterioration index of test roads.

showed that deterioration at points 1 and 3 which were most affected by vehicle loads was greater than at point 2. Secondary tests and measurements were carried out after resealing. This is the main cause of an increase in macro texture and decrease in density demonstrated in the second tests.

When A_2 , B_2 and C_1 were compared to each other, C_1 had the worst condition with respect to deteriorations at the right wheel paths. B_2 and A_2 followed, respectively. The lowest elasticity modulus values were obtained from the C_1 test road. B_2 and A_2 followed, respectively. According to the LWD test results, the lowest elasticity modulus values were obtained from the C_1 test road. However, the deformation values are high (see Table 6).

Indeed, CBR values were obtained from DCP test as $A_2 < C_1 < B_2$. However, CBR values of A_2 and C_1 are so close to each other. Moreover, Alderson [25] reported that a relationship exists between the hardness of the surface of the base layer and the occurrence of flushing. Consequently, flushing is the most common deterioration type found in the C_1 test road (see Figure 3).

The non-destructive test results at point 2 (interval of the wheel path) are better than at points 1 and 3 (see Tables 6–8). However, the embedding of aggregate and flushing are much higher at these points. Although all of the test roads have low total traffic volume, the deterioration rate is not slow. The final test results showed that the final macro texture depth value of the B_2 test road’s point 2 is under 1 mm. According to the TNZ P17 New Zealand performance based specification [26], chip seal should be resealed once the macro texture depth value decreases to under 0.9 mm. Similarly, the values of polishing at point 2 are better than at points 1 and 3; however, the values are close to the appropriate limit of 50. High density values at these points show that flushing started.

Table 6: Nondestructive test results of right wheel path’s (1) points.

Test/number of periodic test	Test roads							
	Erzurum-Karagöbek (A_2)			Tortum-Uzundere (B_2)			Akşar-Göle (C_1)	
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd
Date of test	July 11, 2009	June 12, 2010	August 4, 2010	July 11, 2009	June 12, 2010	August 4, 2010	June 13, 2010	August 5, 2010
Macro texture depth (mm)	2.30	2.66	2.06	1.74	1.92	0.94	2.00	1.54
Number of polishing	35.2	54.0	49.4	25.1	55.8	49.6	46.8	39.9
Density of chip seals (t/m^3)	2.436	1.939	1.977	2.246	2.018	2.171	1.978	2.058
Deformation ^a (μm)	–	259.2	294.7	–	235.0	245.3	314.0	–
Elasticity modulus of base (MPa)	–	169	150	–	156	197	142	–
Average surface temperature ($^{\circ}C$)	43.0	52.0	59.6	30.8	34.2	42.4	50.6	58.8
Thickness of base layer (cm)	13.6			12.4			16.0	

^aThis term was obtained from LWD test.

Figure 3: Flushing deterioration at C_1 test road.

Table 7: Nondestructive test results of interval of the wheel path's (2) points.

Test/number of periodic test	Test roads							
	Erzurum-Karagobek (A_2)			Tortum-Uzundere (B_2)			Aksar-Gole (C_1)	
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd
Date of test	July 11, 2009	June 12, 2010	August 4, 2010	July 11, 2009	June 12, 2010	August 04, 2010	June 13, 2010	August 05, 2010
Macro texture depth (mm)	2.82	2.97	2.37	1.76	1.93	0.98	2.38	1.70
Number of polishing	32.3	63.0	52.2	11.2	53.0	45.0	50.6	46.0
Density of chip seals (t/m^3)	2.271	1.992	2.014	2.105	2.031	2.173	1.976	2.067
Deformation (μm)	–	264.1	274.1	–	215.2	239.6	341.6	–
Elasticity modulus of base (MPa)	–	170	161	–	181	198	–	138

Table 8: Nondestructive test results of left wheel path's (3) points.

Test/number of periodic test	Test roads							
	Erzurum-Karagobek (A_2)			Tortum-Uzundere (B_2)			Aksar-Gole (C_1)	
	1st	2nd	3rd	1st	2nd	3rd	1st	2nd
Date of test	July 11, 2009	June 12, 2010	August 4, 2010	July 11, 2009	June 12, 2010	August 04, 2010	July 11, 2009	June 12, 2010
Macro texture depth (mm)	2.47	2.39	1.68	2.04	1.50	0.94	1.87	1.35
Number of polishing	29.2	59.0	55.4	18.5	55.4	49.1	41.4	36.3
Density of chip seals (t/m^3)	2.289	1.949	2.019	2.141	2.012	2.191	1.978	2.058
Deformation (μm)	–	296.0	263.0	–	197.2	226.8	311.6	–
Elasticity modulus of base (MPa)	–	157	165	–	196	189	–	144

Deterioration at point 3 of the A_2 test road, which has height of embankment of more than 2 m, was less than that of other test roads at point 3.

Low CBR values obtained from the DCP test performed at three different zones for each of the test roads showed that the bearing capacity of test roads was inadequate (see Figure 4). Therefore, the lack of bearing capacity of the base layer in each of the three test roads was one of the most important causes of flushing. However, higher temperature values of road surfaces during summer and the use of unmodified bitumen in chip seal construction also caused higher rates of flushing on the roads (see Figure 5).

First and second DCP tests were performed in the sixth and eighth months, respectively. The sixth month was thawing and rainy season for Erzurum region. Therefore, the first CBR values of A_2 and B_2 test roads obtained lower compression than the second DCP test results. The bearing capacity of C_1 test road was decreased, and this also led to decrease in CBR value (see Figure 4).

Water and moisture from melting snow caused the weakening of sublayers on the shoulder of chip-sealed surfaces. This resulted in low values of CBR and elasticity modulus, which indicate flushing. Although the A_2 test road's CBR and elasticity modulus values were low, its

Figure 4: Variation of base layer's CBR according to the DCP test results.

deterioration rate was lower than those of the other test roads. The reason might be the embankment height of the A₂ road which was higher than 2 m (see Figure 6).

Indeed, Towler and Ball [27] reported that chip seals applied with a binder of at least 1.5 l/m² rate were believed to be completely impermeable in the past, but studies indicated that this was not true. Rain and snow water on chip-sealed road pavements penetrate the chip-sealed pavement by way of tire pressure due to high traffic. After evaporating, this water leads to bitumen rising to the road surface by passing through small holes as bitumen bubbles and consequently causes flushing (see Figure 7).

4 Conclusions

The following conclusions were reached as a result of field observations and non-destructive tests 2 years after A₂ and B₂ test roads were opened to traffic and 1 year after C₁ was opened to traffic.

Each of the test roads deterioration index was under the critical limit of 50. Clear deterioration types at the test roads include aggregate embedment and flushing. These deteriorations are quite high. Because all of the test roads' CBR and elasticity modulus were low, flushing further increased. Moisture that leaked into the granular base under the chip-sealed pavement caused low CBR and elasticity modulus values as reported by Güngör and Sağlık [28], and evaporation within the granular base layer affected the increase of flushing. Low elasticity modulus values obtained from test points 2 and 3 in test roads' zones confirmed this.

Although each of the test roads had low traffic volume, the main cause of aggregate embedment and flushing were the use of non-modified bitumen construction on these test roads. Although the average lowest temperature in the region was -2.8°C and the average highest temperature was 34.2°C, the average surface temperatures reached 59°C, which led to flushing because bitumen reached its softening point. Indeed, Güngör and Sağlık [28] also reported that bitumen that came from Turkey was refined, and the chip seal applications used at the highest

Figure 5: Flushing deteriorations at A₂ and B₂ test roads.

Figure 6: Snow shoulder surface at A₂ and C₁ test roads (1, 2, 3); high slope of embankment and permeable zones of shoulder's surface at A₂ test road (2).

Figure 7: Binder bubbles due to the tendency of vaporization of moisture at base layer during summer season.

locations of East Anatolian Region and lowest locations of Mediterranean and Southeast Anatolian Regions should be modified.

Based on these findings, using modified bitumen in the construction of chip seals in cold climate regions and providing shoulder areas with suitable chip seal applications lead to a decrease in flushing of chip seals.

Acknowledgments: The authors would like to thank the Turkish Republic General Directorate of Highways (TCK), the Turkish Scientific and Technological Research Foundation (TUBİTAK) (Project no.: 107G081) and the Scientific Research Projects Coordination Department of Süleyman Demirel University (SDÜ BAP) (Project Number: 1589-D07) for their financial support.

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