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15. Supplementary Notes Project performed in cooperation with the Texas Department of Transportation and the Federal Highway Administration.  In an effort to identify the benefits of using optimized aggregate gradation (OAG) in paving concrete, three test sections were constructed; each in Wichita Falls (US287 northbound), Fort Worth (SH114B northbound), and Dallas (SH121 westbound) Districts. Both normal aggregate gradation (NAG) and OAG sections were placed side by side in Wichita Falls project. Only OAG sections were placed in Fort Worth and Dallas projects. Fresh and hardened concrete properties of NAG and OAG concretes in Wichita Falls project and OAG concrete in Fort Worth and Dallas projects were evaluated. For the evaluation of pavement performance as affected by the use of NAG and OAG concretes, efforts were made to measure early-age CRCP performance indicators such as crack spacing and crack widths. Since the same cement contents and water-cement ratios were used in NAG and OAG concretes, there was little difference in strength-related hardened concrete properties. On the other hand, differences were observed in in-situ COTE and drying shrinkage between NAG and OAG concretes, with lower values for OAG concrete. Even with lower in-situ COTE and drying shrinkage, more cracks formed in OAG concrete than in NAG concrete section, which signifies the effects of other factors such as setting temperatures on cracking behavior of CRCP. It is expected that better long-term performance will be achieved with OAG concrete due to the lower values of COTE and drying shrinkage.					
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## **Pilot Implementation of Optimized Aggregate Gradation for Concrete Paving**

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Moon Won

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# **Chapter 1. Introduction**

## **1.1 Background**

It is reported that crack width is one of the primary controlling factors of CRCP performance and thus, keeping cracks tight is essential to the optimal performance of CRCP (Ref MEPDG). Theoretical models such as MEPDG indicate a good correlation between crack widths and long-term CRCP performance. Crack width is primarily determined by the degree of drying shrinkage, coefficient of thermal expansion (COTE) of concrete, zero stress temperature (setting temperature), and temperature variations. If concrete experiences less drying shrinkage and thermal movement, crack width will be kept tighter, resulting in better performance.

From a concrete materials standpoint, one way to achieve less drying shrinkage, thermal movement, and heat of hydration is to reduce the volume of cement paste in concrete. One of the most practical ways of achieving this is to use optimized aggregate gradation in concrete mix.

To take advantage of the benefits of optimized aggregate gradation in CRCP, TxDOT constructed CRCP sections in three projects with optimized aggregate gradation. In one project, test sections were placed where both optimized and normal aggregate gradations were used for a side-by-side comparison. Efforts were made, both in the laboratory and in the field, to identify the benefits of using optimized aggregate in CRCP.

## **1.2 Objectives**

The objective of this study is to evaluate the concrete material properties and pavement behavior from actual construction projects using optimum aggregate gradation. In order to achieve this objective, the following tasks were conducted: (1) evaluation of fresh concrete, (2) evaluation of hardened concrete, and (3) evaluation of field pavement behavior.

## **1.3 Scope**

The scope of this report is limited to the evaluation of concrete properties and early-age pavement performance indicators. Concrete material properties and pavement performance indicators evaluated in this study include: (1) slump, air content, bleeding, and setting time of fresh concrete; (2) compressive and flexural strength, modulus of elasticity, in-situ coefficient of thermal expansion (COTE), and in-situ shrinkage of hardened concrete; and (3) temperature and relative humidity and transverse crack spacing.



## Chapter 2. Test Sections

For this study, test sections were placed in three construction projects. They are located in Wichita Falls, Fort Worth, and Dallas Districts. In this chapter, the details of each section are described, including the location, pavement structure, concrete mix proportions, and construction date. The information on pavement structure and concrete mix proportions for each test section is presented in Table 2.1.

### 2.1 Wichita Falls Test Section

This test section is located on US 287 northbound, about 13 miles northeast from Wichita Falls. The concrete at this test section was placed over a period of 6 days in 2005. The normal aggregate gradation (NAG) section was placed for 3 days: August 25, 29, and 30. Optimized aggregate gradation (OAG) section was placed on August 31, and September 1 and 6, 2005. The total lengths of the NAG and OAG sections were 4056 ft and 4057 ft, respectively. The slab thickness of this test section is 13 in. Figures 2.1 and 2.2 illustrate the location and construction layout of this test section, respectively.

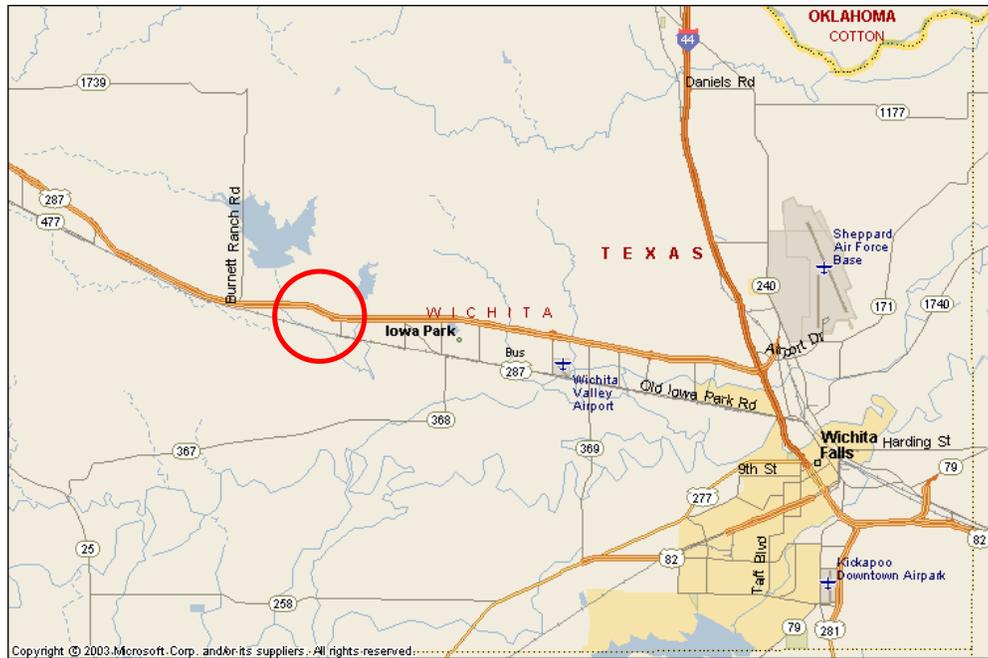


Figure 2.1: Location of Wichita Falls Test Section, US 287

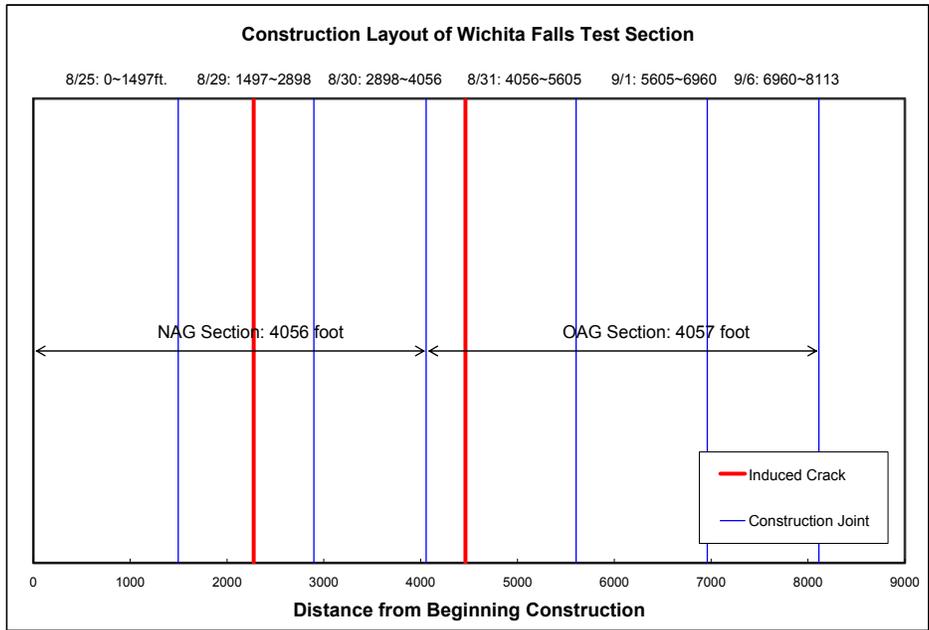


Figure 2.2: Construction Layout of Wichita Falls Test Section, US 287

## 2.2 Fort Worth Test Section

This test section is located on SH 114B (Texan Trail) northbound (Figure 2.3). The concrete was placed on September 19, 2006. Unfortunately, a control section with NAG was not constructed and direct comparisons of concrete properties and pavement behavior containing NAG and OAG were not made.

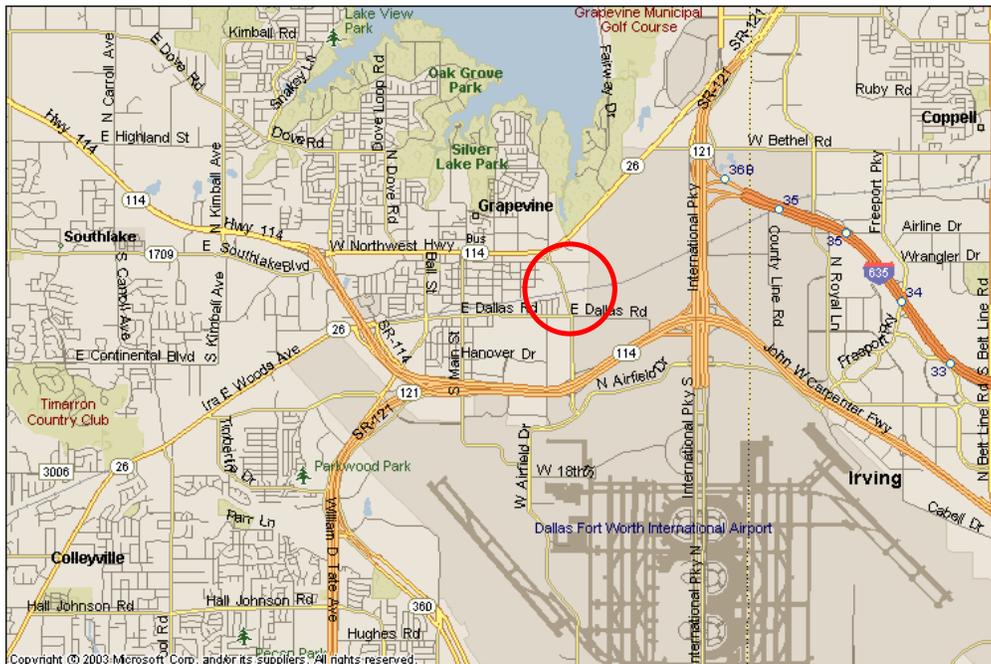


Figure 2.3: Location of Fort Worth Test Section, SH 114B

### 2.3 Dallas Test Section

This test section is located on SH 121 eastbound between Preston Road and Ohio Dr (Figure 2.4). It was constructed on November 16, 2006. The unique feature of this project is that the intermediate size aggregate used in this test section was lightweight aggregate (LWA). As in the Fort Worth Section, a control section with NAG was not constructed and direct comparisons of concrete properties and pavement behavior containing NAG and OAG were not made.

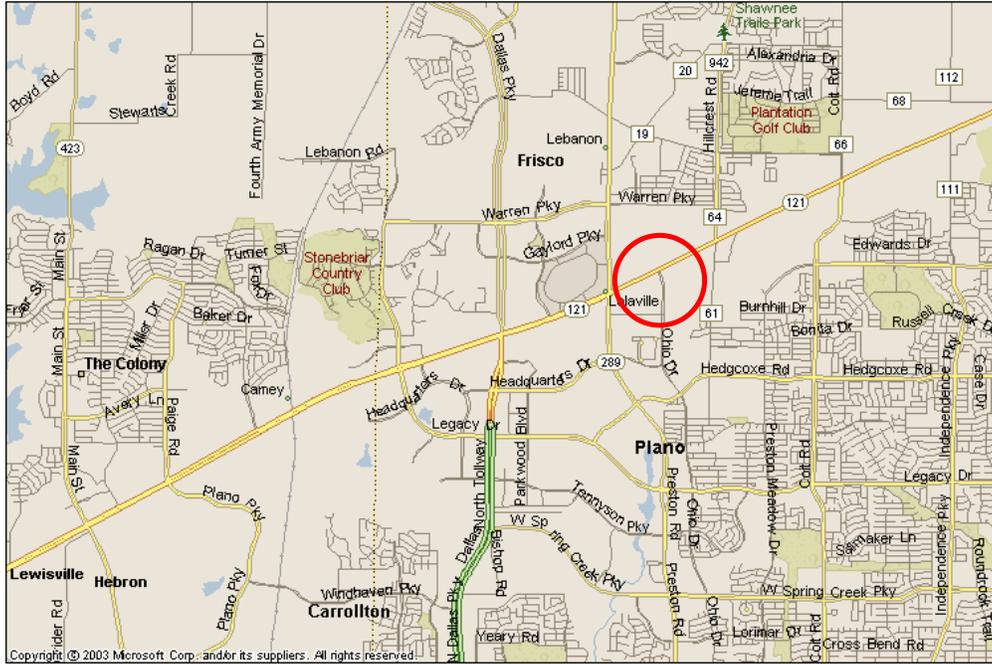


Figure 2.4: Location of Dallas Test Section, SH 121

**Table 2.1: Information for Each Test Section**

	Wichita Falls		Fort Worth	Dallas
Aggregate Gradation	NAG	OAG	OAG	OAG
Highway Location	US 287	US 287	SH 114B	SH 121
Pavement Thickness (inch)	13	13	8	13
Construction Date (MM/DD/YY)	08/25,29, 30/05	08/31/ 09/01, 06/05	09/19/06	11/16/06
Mix Design				
Cement Factor (sk./cy)	5.75	5.75	4.75	5.50
Max. Aggregate Size (inch)	1.75	1.75	1.5	1.5
Water (lbs/cy)	223	223	202	241
Cement (lbs/cy)	324	324	335	413
Coarse Aggregate 1(lbs/cy)	1895	1606	1517	1790
Coarse Aggregate 2 (lbs/cy)	-	450	525	300
Fine Aggregate (lbs/cy)	1248	1096	1252	857
Fly Ash (lbs/cy)	172	172	112	91
Air Entraining (oz/100lbs c.w.t)	7.6	7.6	4	2.5
Reducer Retarding (oz/100lbs c.w.t)	13.6	13.6	16	15

## Chapter 3. Materials and Early-Age Pavement Behavior Evaluation

This chapter presents the results of the evaluations of fresh and hardened concrete and early-age pavement performance indicators as affected by coarse aggregate volumes. As discussed in the previous chapter, test sections in Wichita Falls District had sections with both NAG and OAG. In the other test sections, only OAG was utilized and direct comparisons between OAG and NAG were not made. Even in the sections in Wichita Falls District, the difference between OAG and NAG concretes in terms of the volume of coarse aggregate was quite small. The resulting difference in concrete material properties and early-age pavement performance indicators between them was not large.

### 3.1 Evaluation of Fresh Concrete

Fresh concrete properties were evaluated for slump and slump loss, bleeding, and setting time.

#### 3.1.1 Slump

The slump test of fresh concrete was conducted in accordance with Tex-415-A.

#### Wichita Falls Test Section

It has been reported that the use of OAG could improve the workability of the concrete. The workability of the concrete was evaluated by slump testing. Slump was measured five times in this project: three times on the NAG section and two times on the OAG section. Figure 3.1 shows the mix proportions of NAG and OAG concretes. The contractor decided to use the same cement factor and water-cement ratio for both mixes. As a result, the same amount of cement paste was used for both mixes and little difference (7.7 %) in coarse aggregate volumes exists between the two mixes. This little difference in mix designs between NAG and OAG concretes, along with the limitations in the accuracies of the testing procedures for the properties evaluated in this research, made it difficult to accurately identify the benefits of the use of NAG in PCC pavement. Figure 3.2 shows the slump test results with fresh concrete and ambient air temperatures. The slump values of OAG concrete are actually lower than those of NAG concrete, even though ambient and fresh concrete temperatures were lower for OAG concrete. Since other factors also have effects on slump, the test results do not provide accurate assessment of the effects of aggregate gradation on the workability of the concrete.

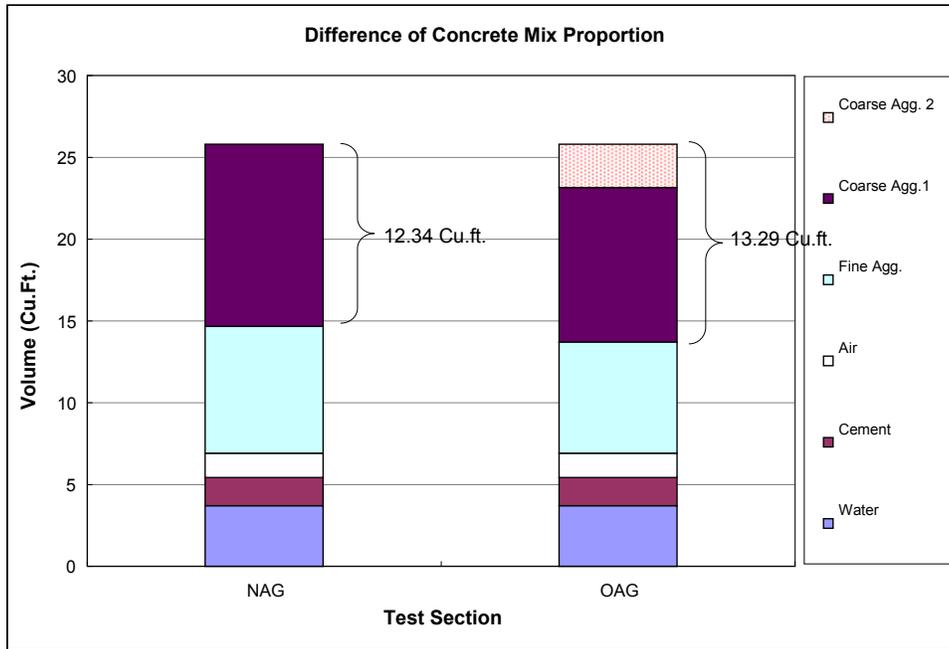


Figure 3.1: Mix Proportions for NAG and OAG in Wichita Falls Test Section, US 287

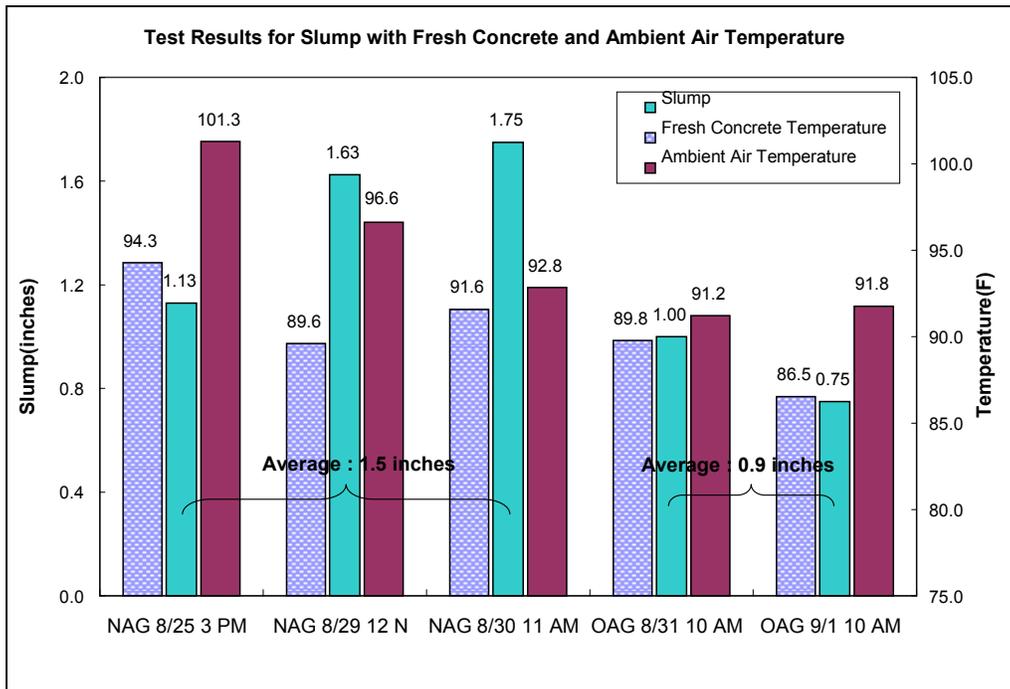


Figure 3.2: Slump Test Results with Fresh Concrete and Ambient Air Temperature for Wichita Falls Test Section, US 287

**Fort Worth Test Section**

A total of four slump tests were conducted in this test section; two tests by a CTR researcher (the first one in the morning and the second one in the afternoon) and the other two by TxDOT personnel. Figure 3.3 shows the test results. This illustrates the variability in slump testing results even for an identical concrete mix.

**Dallas Test Section**

As described earlier, this concrete has lightweight aggregate as intermediate size aggregates. The slump of the concrete was 1.25 in.

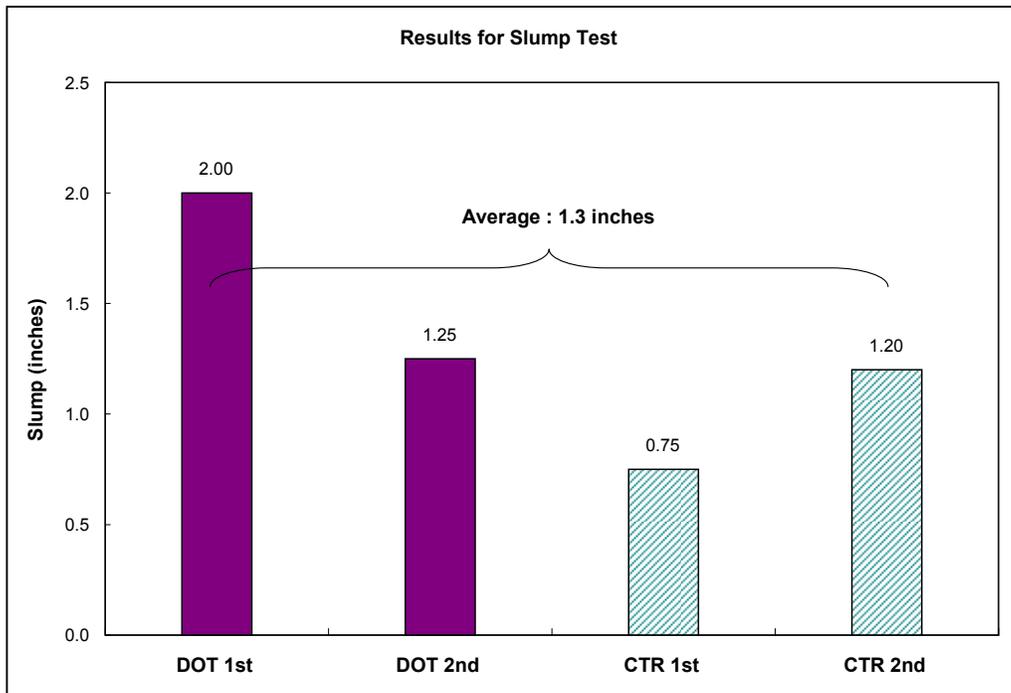


Figure 3.3: Slump Test Results for Fort Worth Test Section, SH 114B

In addition to the slump at a specific time, the changes of slumps over time were also evaluated. The test was performed only on the Wichita Falls test section. It was performed every 15 minutes until the slump value equals to zero. The objective of this testing was to determine whether aggregate gradation will have effects on the changes in slump over time.

Figure 3.4 shows the test results. In almost all the cases, the slump became almost zero after 30 minutes except for the OAG section tested on August 31. As noted in Figure 3.2, due to the variability in slump testing results, it's not feasible to draw firm conclusions from the data shown in Figure 3.4 regarding the aggregate gradation effects on the changes in slump over time.

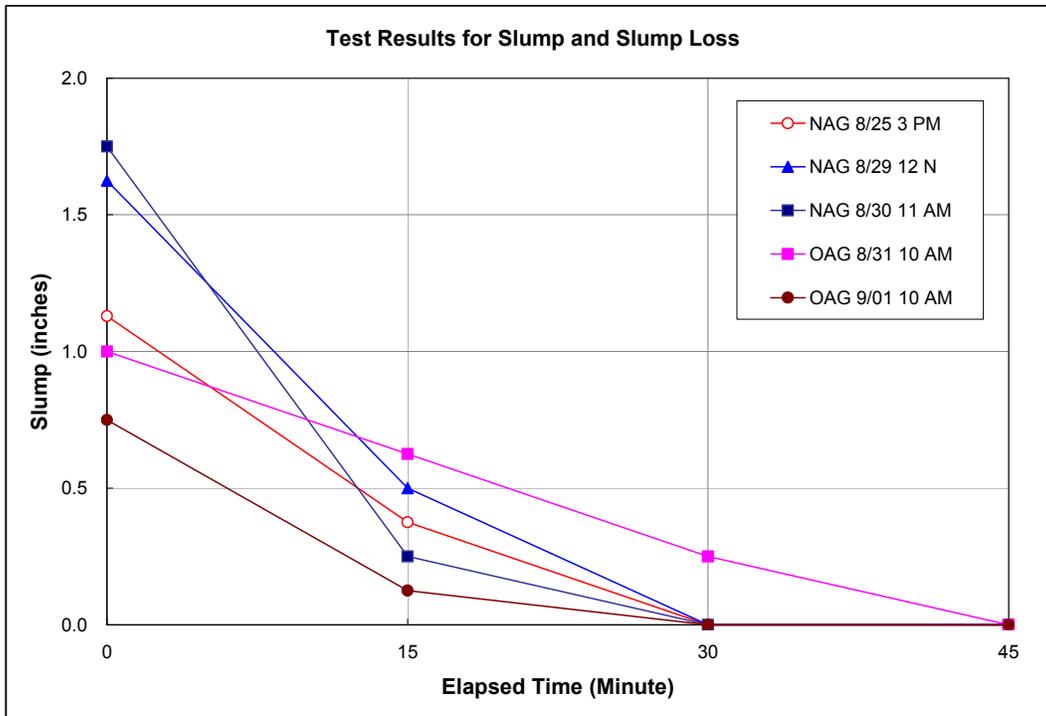


Figure 3.4: Slump Loss Test Results for Wichita Falls Test Section, US 287

### 3.1.2 Bleeding of Concrete

Even though bleeding does not have effects on PCC pavement performance as long as it is not excessive, aggregate gradation might have effects on bleeding of the concrete. The bleeding test of concrete was evaluated in accordance with ASTM C 232.

#### Wichita Falls Test Section

The amount of bleeding water was so small that an injector with a small needle was needed to collect the water as shown in Figure 3.5. Figure 3.6 shows the test results for bleeding of concrete and from a practical standpoint, there was no difference between NAG and OAG concretes.



Figure 3.5: Bleeding Test of Concrete

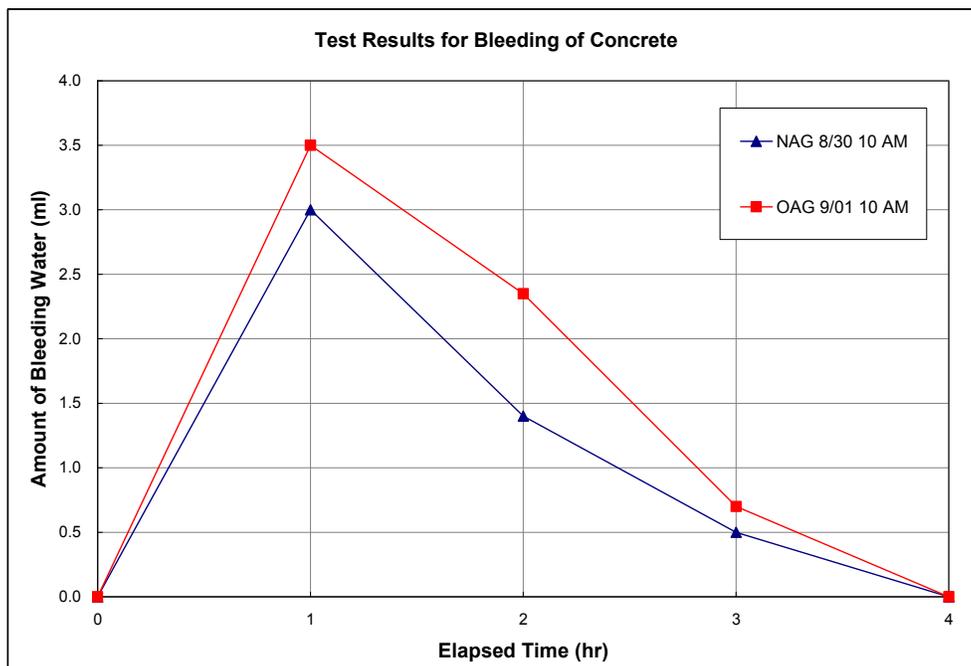


Figure 3.6: Bleeding Test Results for Wichita Falls Test Section, US 287

**Fort Worth Test Section**

The bleeding test for this section was conducted in the morning (10:20 a.m.) and in the afternoon (5:07 p.m.). Figure 3.7 shows the bleeding test results, which shows a slightly larger amount of bleeding in the afternoon than in the morning, even though the difference is quite small. Also, larger bleeding water was observed than that in the Wichita Falls sections.

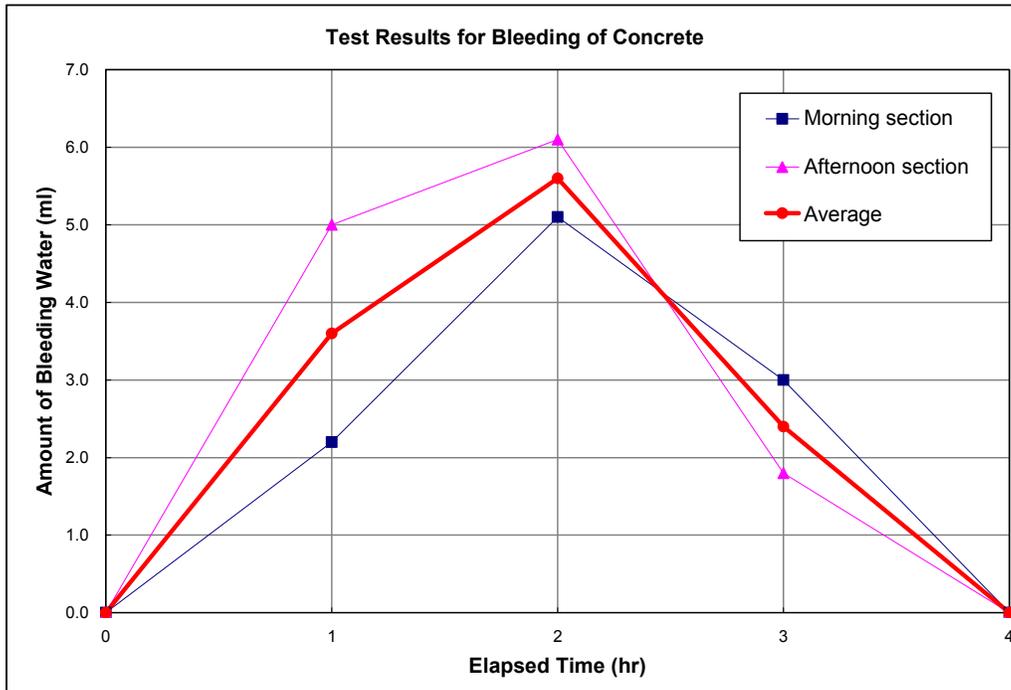


Figure 3.7: Bleeding Test Results for Fort Worth Test Section, SH 114B

**Dallas Test Section**

The bleeding test was conducted two times in the morning (8:50 a.m.). The amount of bleeding was the smallest among all sections as shown in Figure 3.8. It is not known whether the use of lightweight aggregate as intermediate aggregate is responsible for this little bleeding.

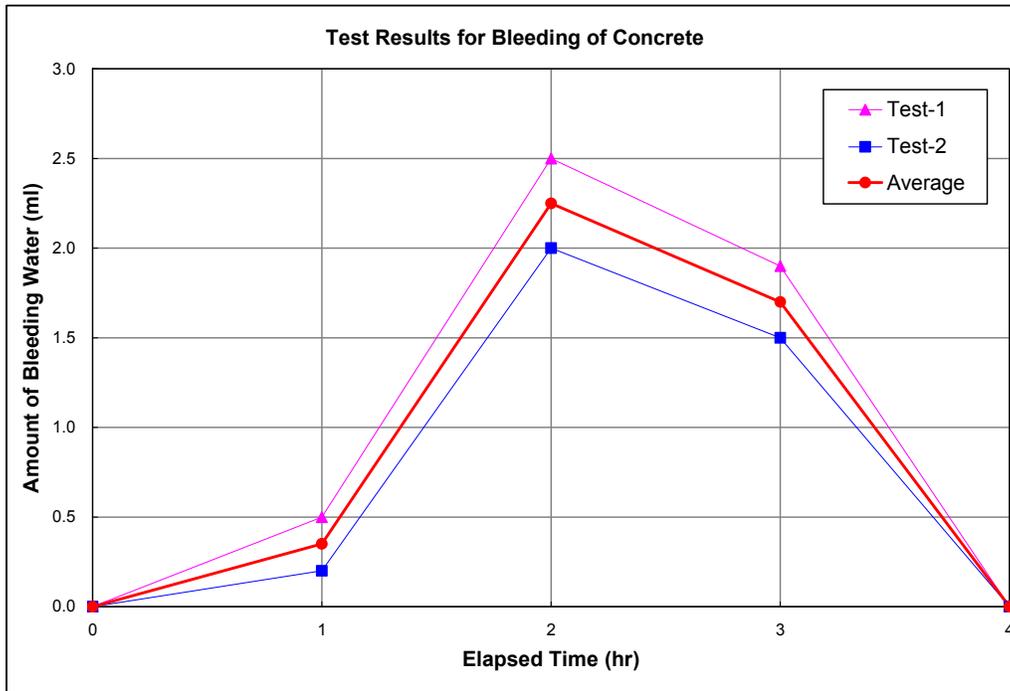


Figure 3.8: Bleeding Test Results for Dallas Test Section, SH 121

### 3.1.3 Setting Time of Concrete

The setting time test of concrete was conducted in accordance with ASTM C 403.

#### Wichita Falls Test Section

Figure 3.9 shows testing for the evaluation of setting time of concrete. Figure 3.10 shows the testing results for OAG and NAG sections. The value of R square from regression analysis is shown to be 0.99 for both test sections. Initial and final setting times of this test section were 2 hours and 36 minutes and 3 hours and 59 minutes for the OAG section and 2 hours and 38 minutes and 4 hours and 9 minutes for the NAG section. The difference in setting times between the two test sections is relatively small. As described earlier, there is little difference in the concrete mix designs for NAG and OAG, which could explain rather similar values of setting times between the two mixes.



Figure 3.9: Testing for Setting Time

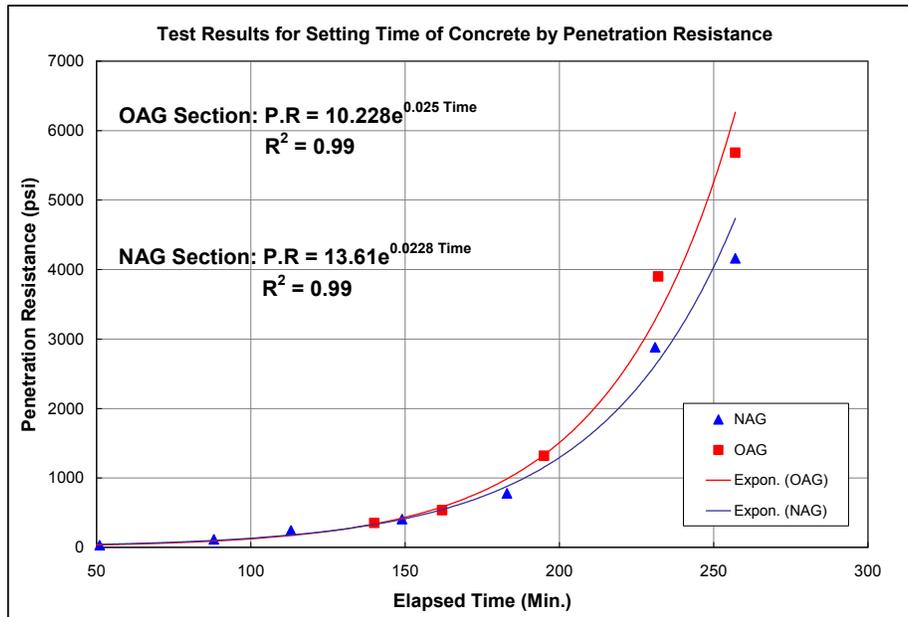


Figure 3.10: Test Results for Setting Times of Concrete for Wichita Falls Test Section, US 287

**Fort Worth Test Section**

Figure 3.11 shows the test results for the setting time of concrete evaluated in the morning and afternoon at the Fort Worth test section. The value of R-square from the regression analysis is shown to be 0.99 and 0.98 for both periods. Initial and final setting

times of this test section were 3 hours and 36 minutes and 4 hours and 34 minutes for the morning section and 3 hours and 43 minutes and 5 hours and 2 minutes for the afternoon section. It is noted longer setting times for concrete placed in the afternoon compared with the concrete placed in the morning. The concrete temperatures were not measured during the setting testing. However, research studies showed that the concrete placed in the morning reached higher setting temperature than the concrete placed in the afternoon. The difference in setting temperatures between morning and afternoon placed concretes could explain faster setting time for morning placed concrete.

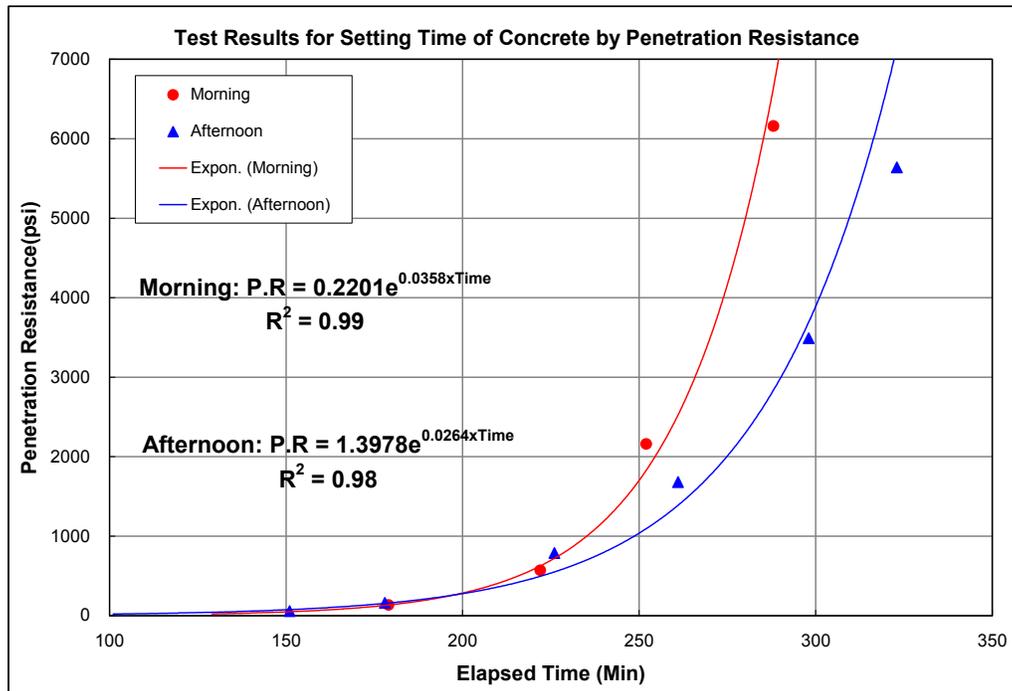


Figure 3.11: Test Results for Setting Time of Concrete for Fort Worth Test Section, SH 114B

**Dallas Test Section**

This test was conducted at 9:25 a.m. on November 16, 2006. Figure 3.12 shows the regression analysis and test results for the setting time of concrete. Initial and final setting times of this test section were 5 hours and 28 minutes and 8 hours and 29 minutes, respectively. The results show longer setting times than in the other test sections. It is noted that the temperature condition during concrete placement in this section was the lowest among all the sections.

Table 3.1 summarizes initial and final setting times from test results for each test section.

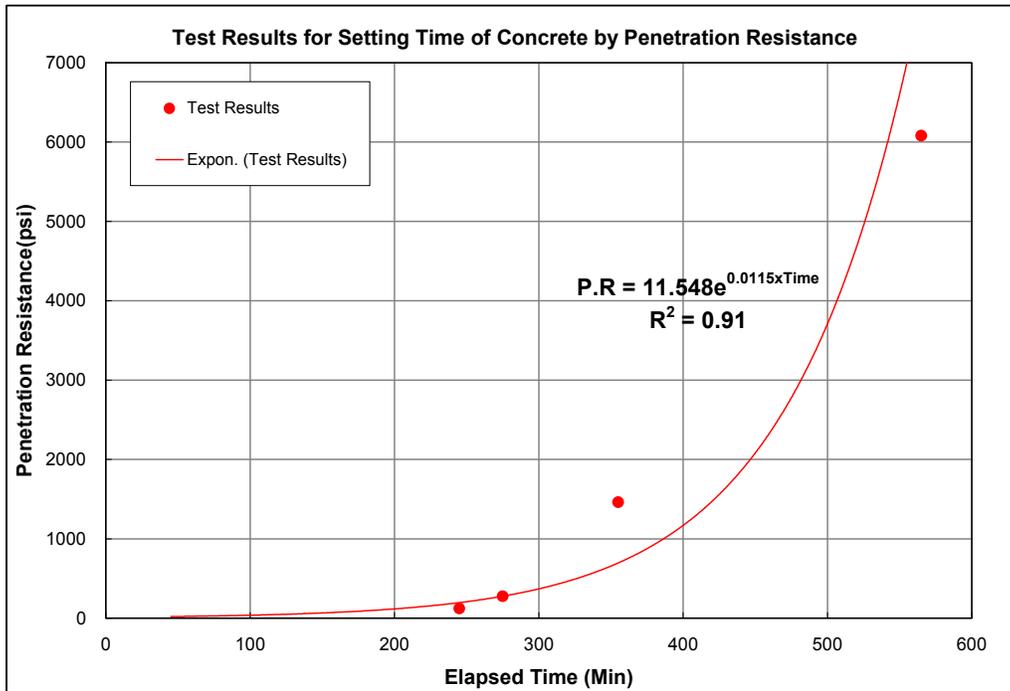


Figure 3.12: Test Results for Setting Time of Concrete for Dallas Test Section, SH 121

Table 3.1: Initial and Final Setting Time of Each Test Section

Test Section	Unit	Wichita Falls US-287		Fort Worth SH-114B OAG	Dallas SH-121 OAG
		NAG	OAG		
Initial Setting Time	Hrs : Min	2 : 38	2 : 36	3 : 40	5 : 28
Final Setting Time		4 : 09	3 : 59	4 : 48	8 : 29

### 3.2 Evaluation of Hardened Concrete

Hardened concrete properties evaluated include compressive strength, flexural strength, modulus of elasticity, coefficient of thermal expansion (COTE), and in-situ drying shrinkage.

#### 3.2.1 Compressive Strength

The compressive strength of concrete was evaluated in accordance with Tex-418-A.

##### Wichita Falls Test Section

Tests were conducted at 1, 2, 7, 13, and 27 days after the concrete placement for the OAG section and at 2, 4, 8, 15, and 29 days for the NAG section. As shown in Figure 3.13,

there is little difference in compressive strength between NAG and OAG concretes, as expected since the same cement factor and water-cement ratios were used for both mixes. Overall, the strength values are quite high, reaching 28-day required strength of 4,000 psi at about one week.

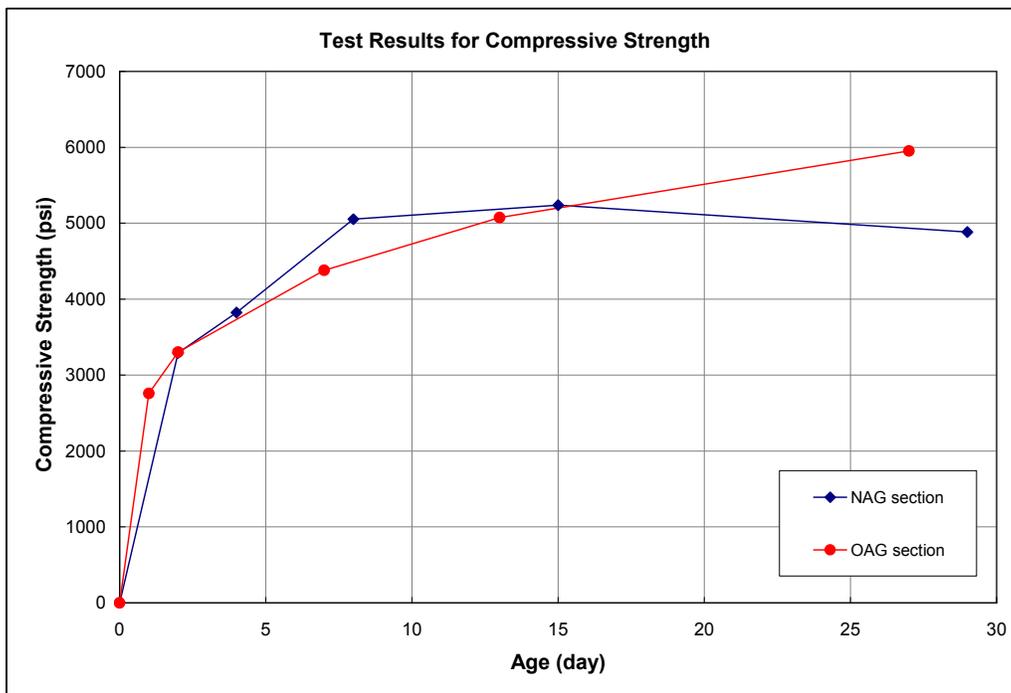
**Fort Worth Test Section**

Figure 3.14 shows the test results for compressive strength for the Fort Worth test section. Compressive strength values were obtained at 7, 14, 21, and 56 days after concrete placement. Testing results for non-OAG concrete, which was placed several months prior to the placement of OAG concrete, were obtained and provided in the Figure 3.14. It is also shown that, for OAG concrete, the 28-day required strength of 4,000 psi was achieved in 5 days.

**Dallas Test Section**

Figure 3.15 shows the test results for compressive strength of the Dallas test section. This test section used lightweight aggregate as intermediate aggregate. Overall compressive strength of this test section is higher than the strength values for the other test sections.

Table 3.2 presents the test results for compressive strength of all the test sections.



*Figure 3.13: Test Results for Compressive Strength of Wichita Falls Test Section, US 287*

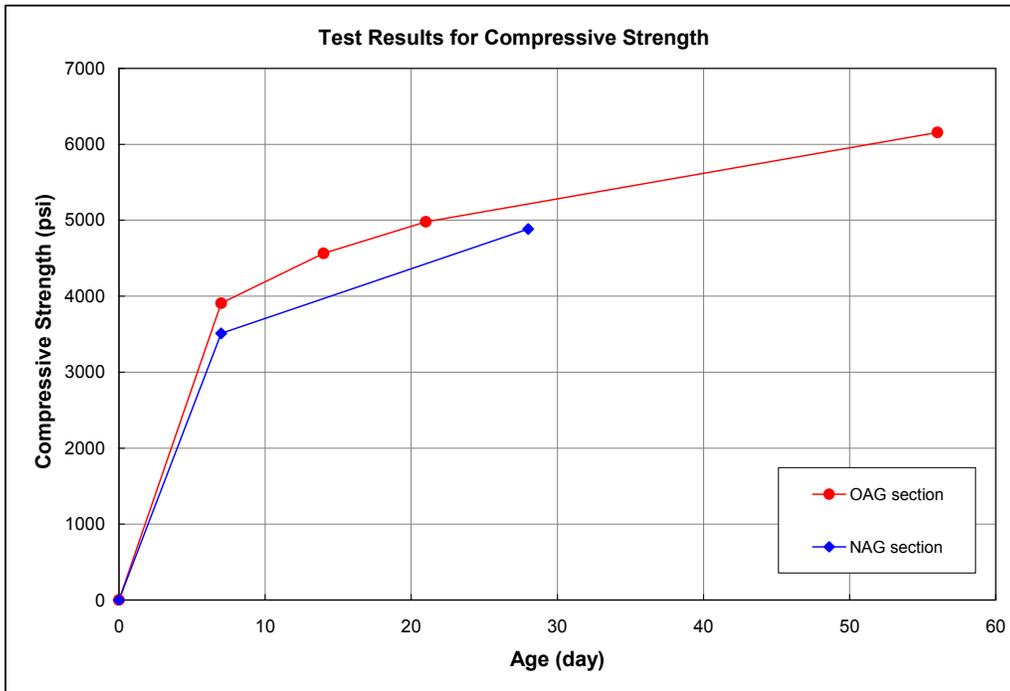


Figure 3.14: Test Results for Compressive Strength for Fort Worth Test Section, SH 114B

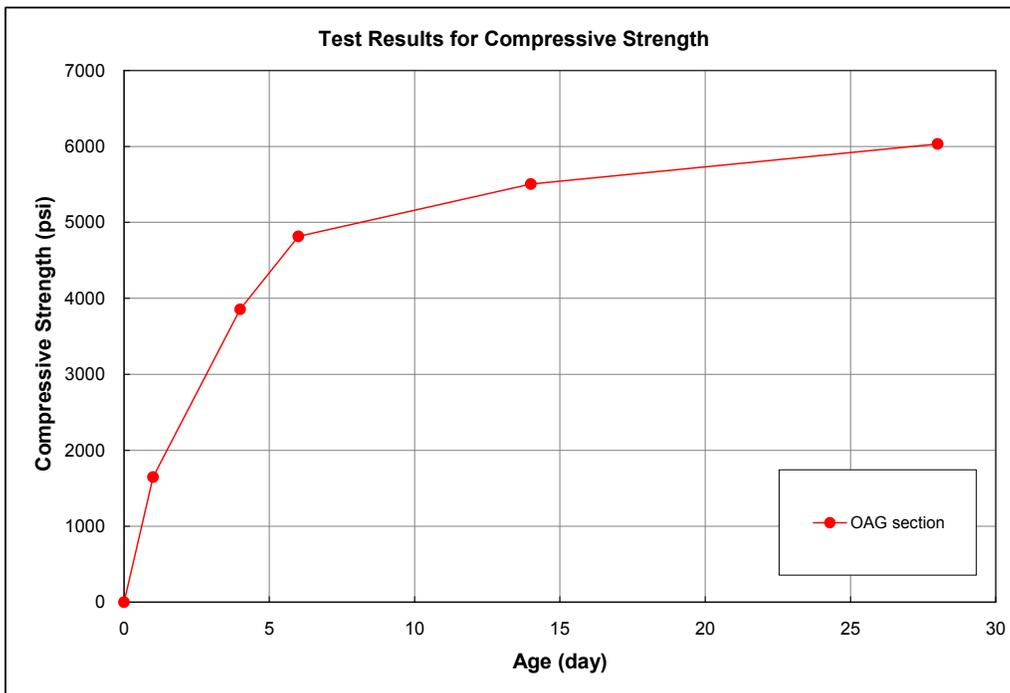


Figure 3.15: Test Results of Compressive Strength for Dallas Test Section, SH 121

**Table 3.2: Test Results for Compressive Strength for Each Test Section**

[Unit: psi]

Ages	Wichita Falls		Fort Worth		Dallas
	NAG	OAG	NAG	OAG	
1	-	2760	-	-	1650
2	3290	3300	-	-	-
4	3820	-	-	-	3860
6	-	-	-	-	4810
7	-	4380	3510	3910	-
8	5050	-	-	-	-
13	-	5080	-	-	-
14	-	-	-	4570	5510
15	5240	-	-	-	-
21	-	-	-	4980	-
27	-	5950	-	-	-
28	-	-	4890	-	6030
29	4880	-	-	-	-
56	-	-	-	6160	-

### 3.2.2 Flexural Strength

The flexural strength of the concrete was evaluated in accordance with Tex-448-A.

#### Wichita Falls Test Section

The testing for flexural strength was conducted at the same ages as the compressive strength test. Figure 3.16 shows the test results. It shows flexural strength of OAG section exhibited approximately 9 percent higher strength than the NAG section. Recall that the cement content and water cement ratio were identical for the two mix designs. Also, note that the two mixes showed quite similar compressive strength (Figure 3.13). It appears that 7.7 % more coarse aggregate volume in OAG concrete than in NAG concrete could have contributed to the 9 % higher flexural strength. Higher flexural strength concrete is expected to result in fewer transverse cracks, with all the other conditions being equal.

#### Fort Worth Test Section

Figure 3.17 shows test results for the flexural strength test for the Fort Worth test section. The flexural strength test was conducted on the same days as the compressive strength test. TxDOT conducted the test at 7 days for the non-OAG section, which was placed several months before this section was placed. Both results are shown in Figure 3.17, which shows flexural strength of the OAG section at 7 days was about 10 percent higher than that of the non-OAG section.

Flexural strength for Dallas test section was not conducted due to time constraints during the field testing.

Table 3.3 shows the test results for flexural strength of the Wichita Falls and Fort Worth test sections.

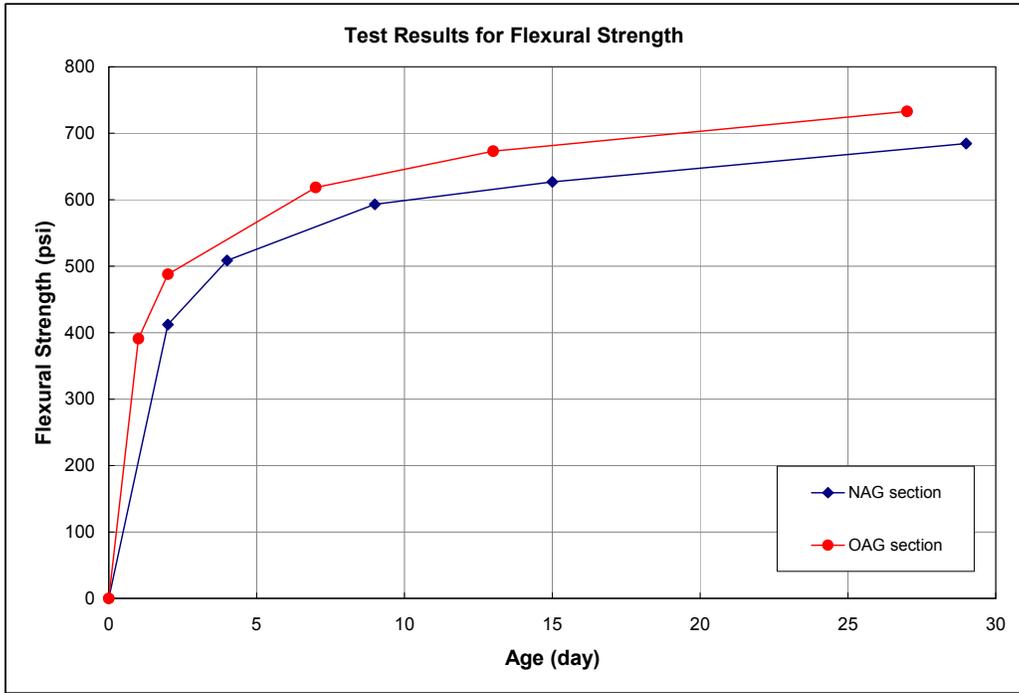


Figure 3.16: Test Results of Flexural Strength for Wichita Falls Test Section, US 287

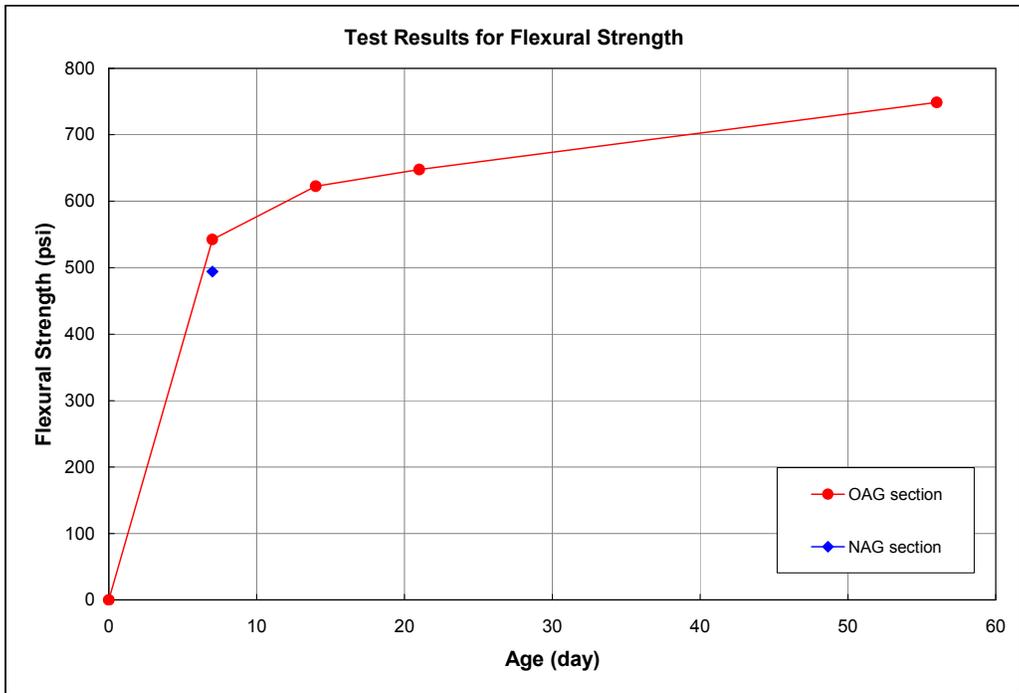


Figure 3.17: Test Results of Flexural Strength for Fort Worth Test Section, SH 114B

**Table 3.3: Test Results of Flexural Strength for Each Test Section**

[Unit: psi]

Ages	Wichita Falls		Fort Worth	
	NAG	OAG	Non-OAG	OAG
1	-	391	-	-
2	412	488	-	-
4	509	-	-	-
7	-	619	494	543
9	593	-	-	-
13	-	673	-	-
14	-	-	-	623
15	627	-	-	-
21	-	-	-	648
27	-	733	-	-
29	685	-	-	-
56	-	-	-	749

### 3.2.3 Modulus of Elasticity

Modulus of elasticity of concrete was evaluated by free-free resonance column methods; this test was performed for the concrete from Wichita Falls and Fort Worth test sections only. However, the results of the Fort Worth test section were not reliable due to an equipment problem and the results are not presented. The same specimens tested for the compressive strength testing were used for modulus of elasticity prior to the compressive strength testing. Figure 3.18 shows the test results for the dynamic modulus of the Wichita Falls test section. It shows the modulus values are in the expected range; however, valid conclusions could not be made for the effects of coarse aggregate volume on the modulus values.

Figures 3.19 and 3.20 show the relationship between the dynamic modulus and compressive strength, and the dynamic modulus and flexural strength, respectively, obtained for the concrete from Wichita Falls test sections. Due to the scatter of the data, conclusive statements cannot be made of the effects of coarse aggregate volume on the relationship between dynamic modulus and compressive or flexural strength.

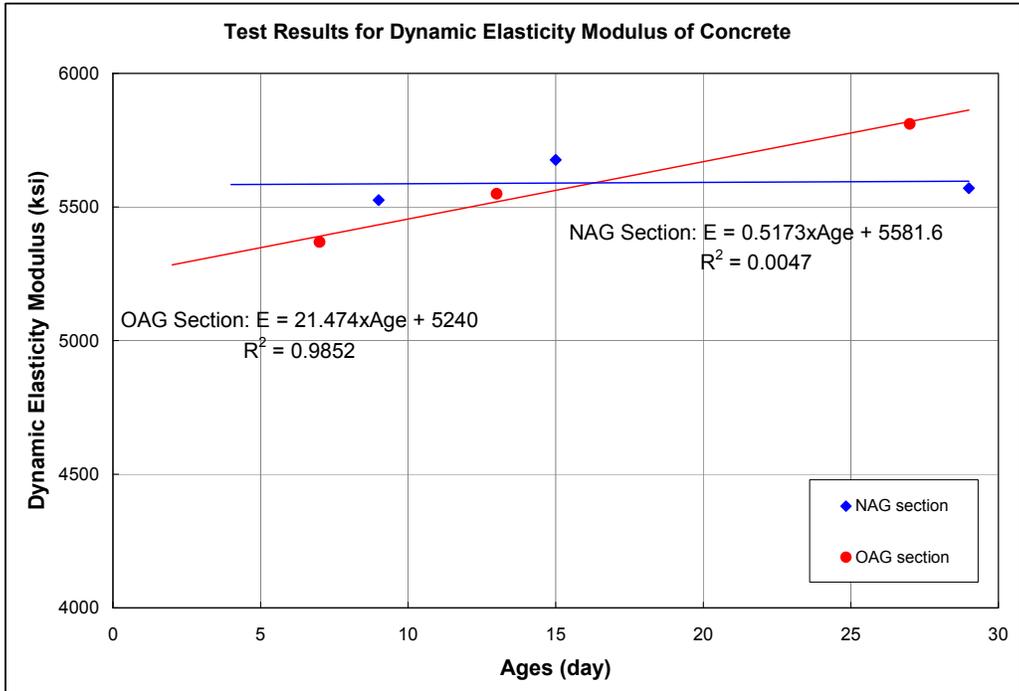


Figure 3.18: Test Results of Dynamic Elasticity Modulus for Wichita Falls Test Sections

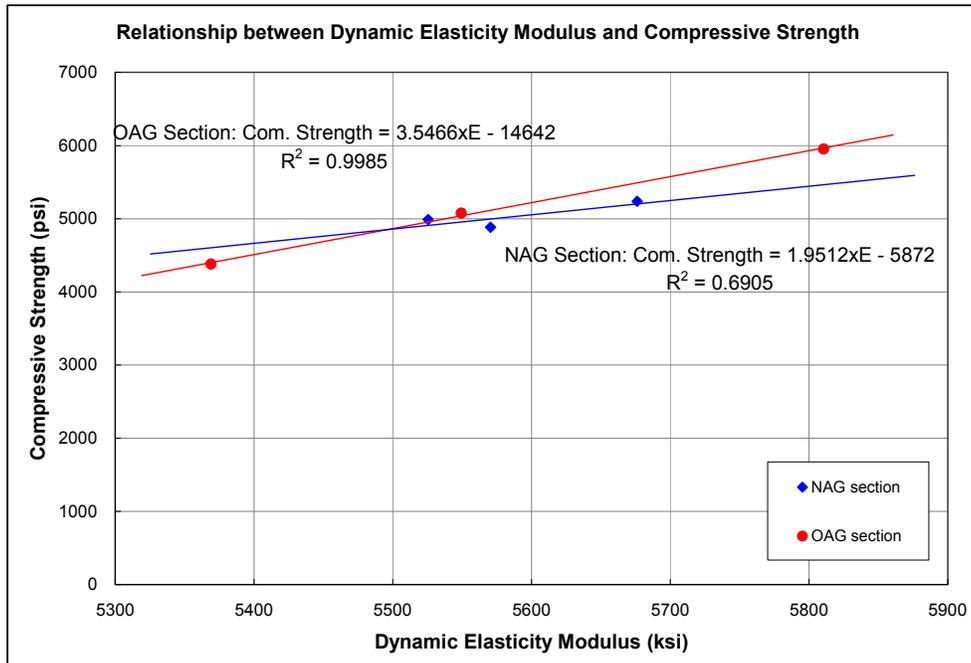


Figure 3.19: Relationship between Dynamic Elasticity Modulus and Compressive Strength

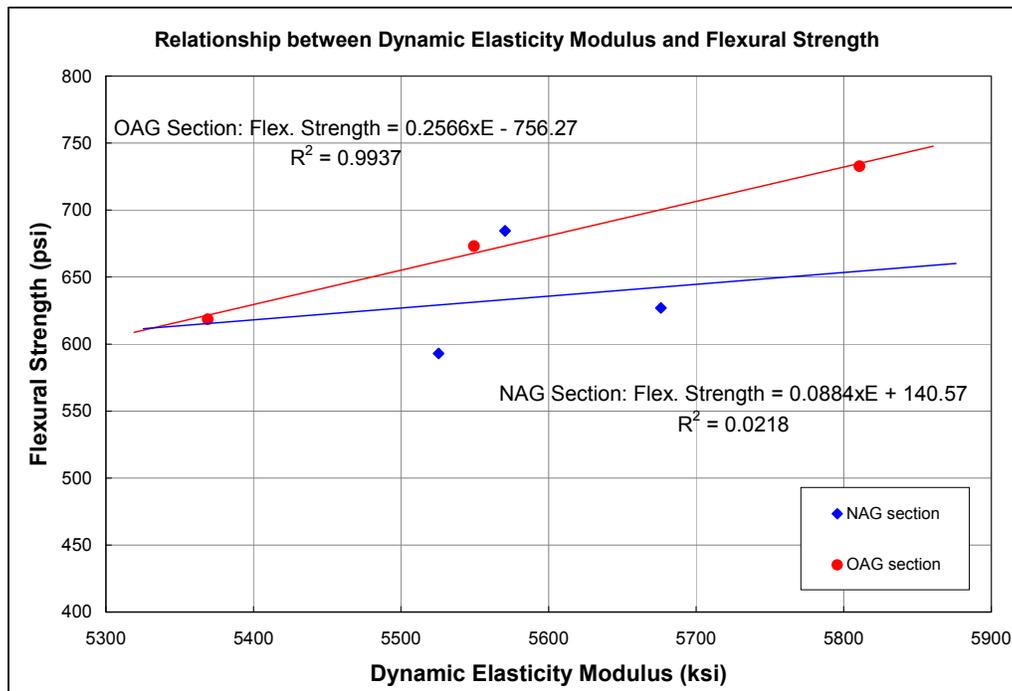


Figure 3.20: Relationship between Dynamic Elasticity Modulus and Flexural Strength

### 3.2.4 In-Situ COTE

The COTE of concrete depends on the coarse aggregate type as well as the volume of the aggregate used in the concrete. In CRCP, volume changes of concrete due to shrinkage and temperature variations result in transverse cracks and their characteristics have an effect on performance.

In this project, the COTE of concrete was evaluated in the field and laboratory. For in-situ COTE evaluation, a method developed during TxDOT research project 0-1700 was used. In this method, concrete prisms are made with vibrating wire gages embedded in the middle of the specimens, which are cured and placed at the job site. The data logger records the temperature and volume changes resulting from the temperature and moisture variations. The improved method for in-situ COTE measurement would be to use a non-stress cylinder. This was achieved by placing a non-stress cylinder with an embedded gauge in actual pavement. This study used two types of cylinders. One is a cylinder that blocks any moisture movement between the inside and outside of the cylinder. The other is a cylinder with perforations that allow the movement of moistures between the inside and outside of the cylinder. The strain obtained from the former is solely due to temperature variations, while that obtained from the latter is due to both temperature and moisture variations. In-situ drying shrinkage of the concrete would be the difference between them.

For the evaluation of COTE in the laboratory, TxDOT improved the current AASHTO TP-60 and came up with a repeatable and accurate test method. For the laboratory evaluation, specimens, after being made and cured in accordance with Tex-447-A, were tested. The laboratory COTE evaluations were conducted in accordance with Tex-428-A by Construction Division, Materials and Pavements Section (CSTMP) staff.

Figure 3.21 shows the in-situ COTE test results of NAG and OAG concretes in the Wichita Falls test section. The COTE values of the OAG and NAG concretes were 4.9 and 5.7 microstrain/°F, respectively, which amounts to 14 % difference  $((5.7-4.9)/5.7*100)$ . It

appears that the larger coarse aggregate volume in OAG concrete than in NAG concrete, even though it was only 7.7 % more in OAG concrete, contributed to this difference in in-situ COTE. Note that coarse aggregate has lower COTE than cement paste, and the more coarse aggregate volume in the concrete, the lower the COTE.

Figures 3.22 and 3.23 show the test results for the laboratory testing of OAG concrete conducted by TxDOT. This test was carried out two times. From this test, the average COTE of OAG concrete obtained was 4.21microstrain/°F, which is 0.7 micro-strain/°F small than the in-situ COTE. The reason for this difference is not known. The difference in relative humidity during testing might explain part of the difference. In Tex-428-A, the specimens are fully saturated during the testing, while the in-situ concrete was not quite possibly fully saturated. Concrete at less than fully saturation has higher COTE than fully saturated concrete does.

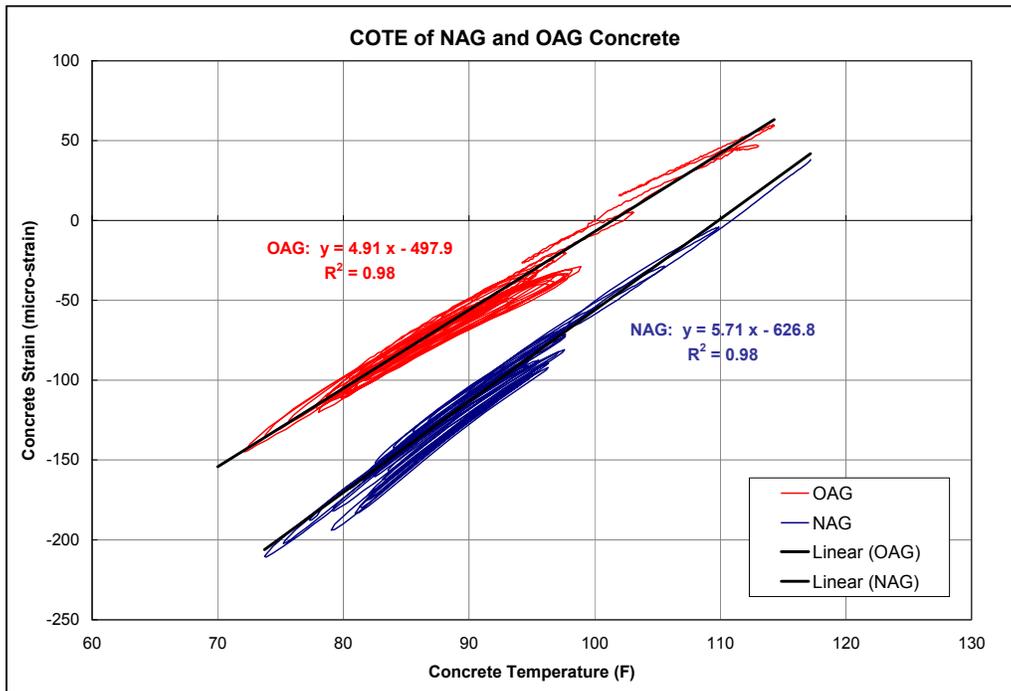


Figure 3.21: COTE of NAG and OAG concrete in Wichita Falls Test Section, US 287

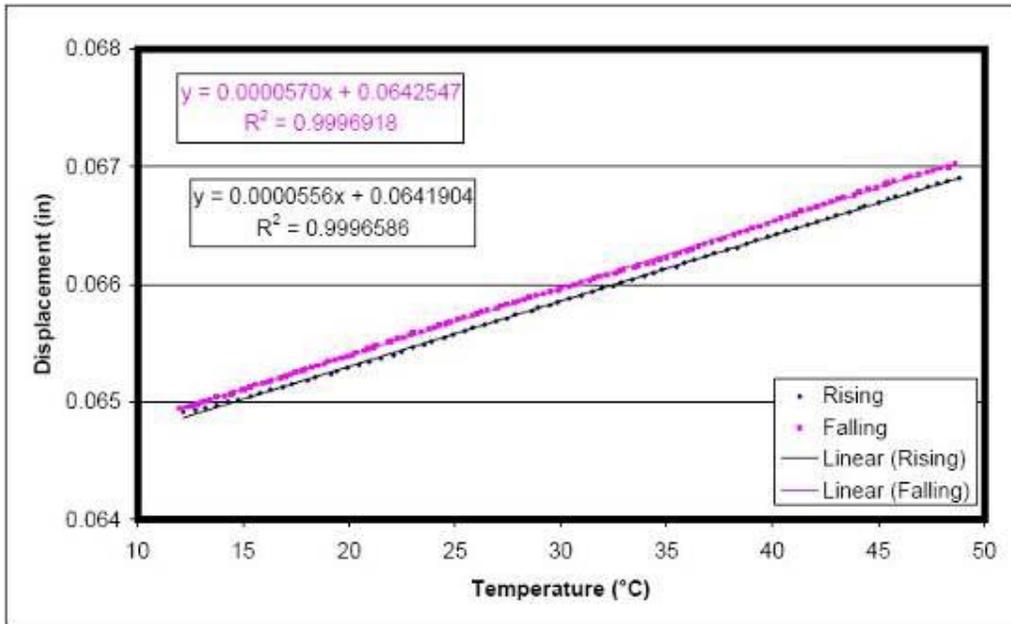


Figure 3.22: Laboratory COTE1 of OAG concrete conducted by CSTMP staff

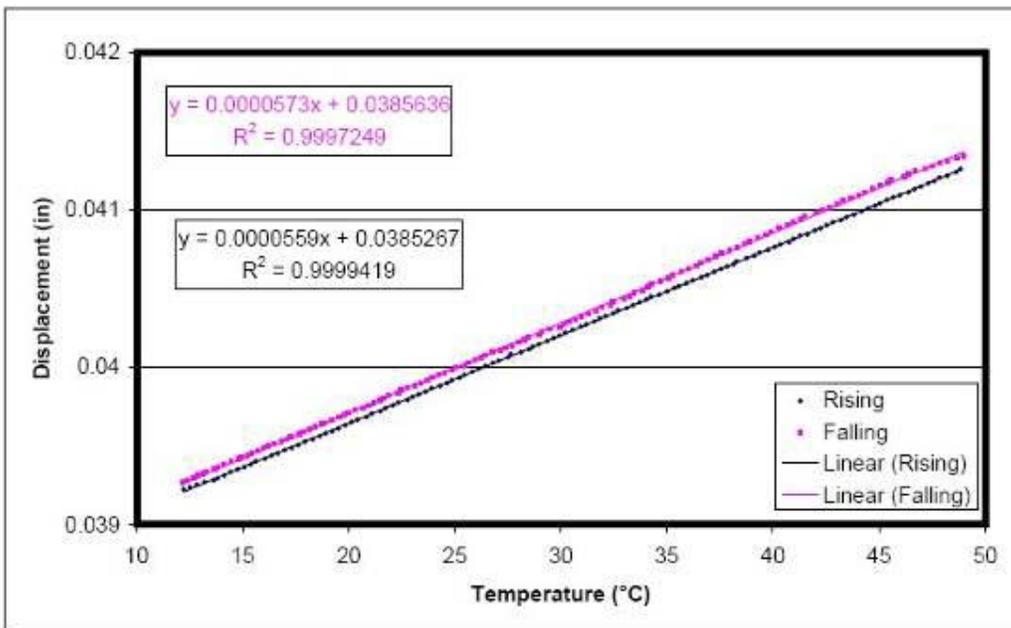


Figure 3.23: Laboratory COTE2 of OAG concrete conducted by CSTMP staff

### 3.2.5 In-Situ Drying Shrinkage

Cement paste shrinks or swells due to the loss or gain of moisture in the cement paste. Therefore, drying shrinkage of concrete is sensitive to the volume of cement paste in concrete.

For in-situ drying shrinkage evaluations, test specimens used for in-situ COTE evaluations were utilized. The in-situ COTE testing procedures described previously provided in-situ drying shrinkage in Wichita Falls test sections.

### Wichita Falls Test Section

Figure 3.24 shows the test results for drying shrinkage of NAG and OAG test sections obtained from non-stress cylinders for 29 days. Note that the non-stress cylinder placement times were different. Figure 3.24 shows more in-situ drying shrinkage at NAG section than at OAG section, which is somewhat expected. Recall that OAG concrete has more coarse aggregate volume and less cement paste than NAG concrete, even though the difference is not large.

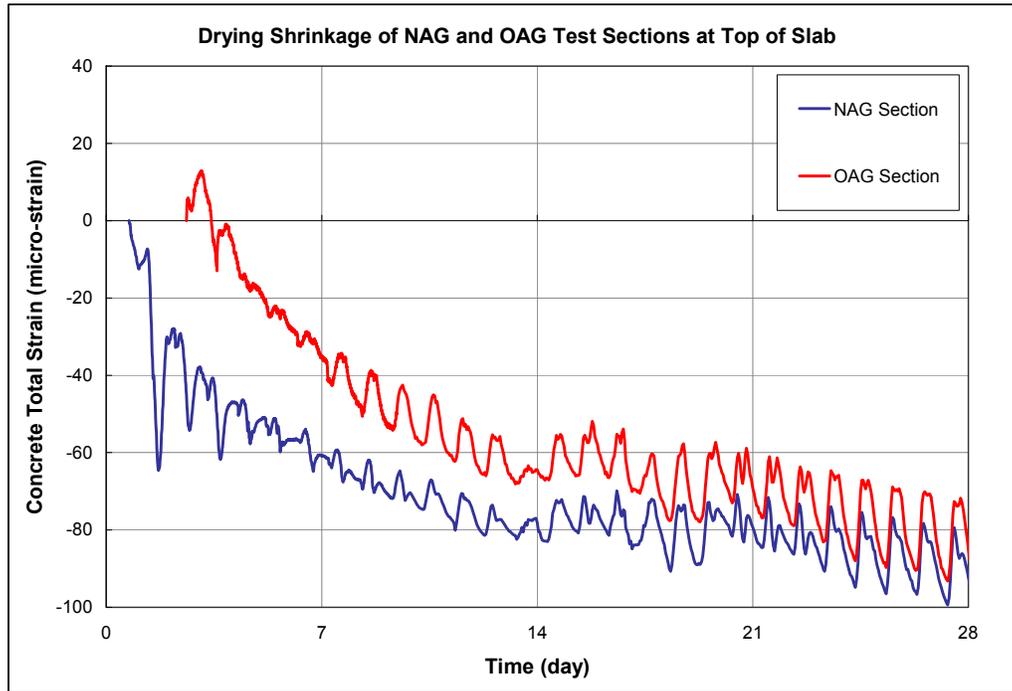


Figure 3.24: Drying Shrinkage of NAG and OAG Concretes in Wichita Falls Test Section, US 287

### Dallas Test Section

As discussed earlier, lightweight aggregate was used as intermediate aggregate for the Dallas test section, which was constructed on November 16, 2006. In order to measure the shrinkage of concrete, two small beams were made with vibrating wire strain gauges installed in the center of the beams. One beam was made with concrete as received from the test section. The other beam was made with concrete after lightweight aggregates were removed from the concrete. Both beams were stored in the same room and subjected to the identical environmental condition. The testing setup is shown in Figure 3.25.

Figure 3.26 shows the test results for shrinkage. For calculation of shrinkage, COTE of concrete was assumed as 4.0 micro-strain/°F. As shown in Figure 3.26, the shrinkage of concrete containing lightweight aggregate was about 20 percent lower than that of the concrete without. This could be due to the so-called “internal curing,” which means that lightweight aggregates, thanks to their high void contents, absorb water and when the surrounding paste becomes dry, they release moisture to the cement paste, thus helping keep the concrete moist and minimizing drying shrinkage.



Figure 3.25: Drying Shrinkage Test Setup in Laboratory

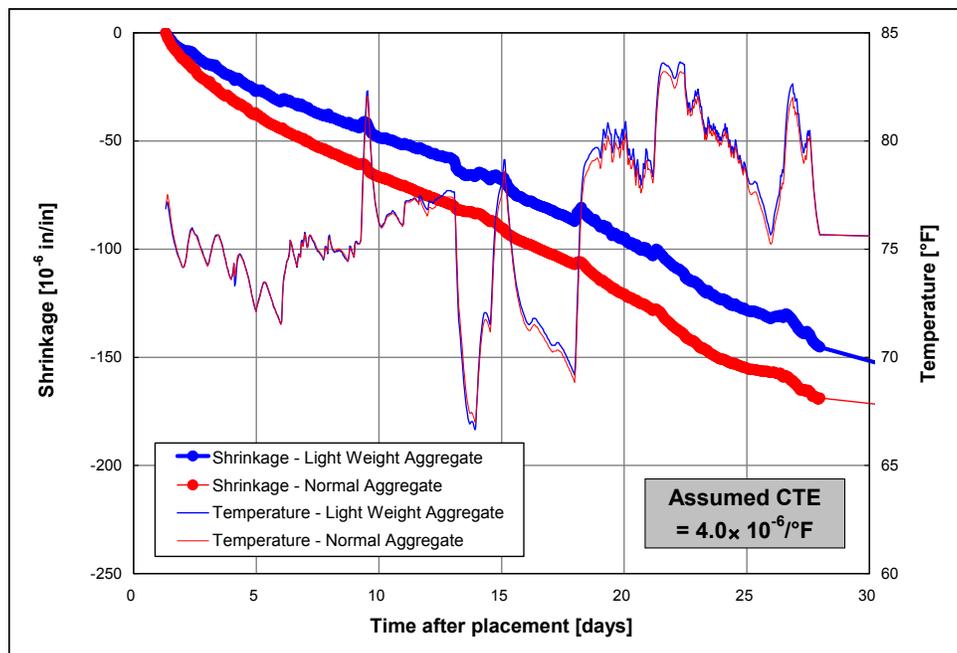


Figure 3.26: Drying Shrinkage of OAG Concrete Using LWA in Dallas Test Section, SH 121

### 3.3 Evaluation of Early-Age Pavement Behavior

#### 3.3.1 Concrete Temperature

Temperature variations ( $\Delta T$ ) after concrete placement, more specifically the difference between zero-stress temperature and concrete temperature at any time, affect the thermal stress of concrete in CRCP. Crack width also depends on the temperature variations. To monitor the concrete temperatures in the Wichita Falls sections, temperature measuring gages, called I-buttons were installed at the top, middle, and bottom of the slab at the locations near the induced cracks in NAG and OAG sections. The locations were 2,350 and 4,465 feet from the beginning of the construction.

Figures 3.27 and 3.28 show the results for concrete and ambient air temperatures measured by I-buttons in the NAG and OAG sections, respectively. The NAG section was placed on August 29 and the OAG section on August 31, which means that the environmental condition during the construction of both test sections is different. It is shown that the maximum concrete temperature in the NAG section exceeded 120 °F, while that in the OAG section was below 120 °F. Since the environmental condition during the concrete placement was different for both sections, a simple comparison cannot be made regarding the effect of coarse aggregate content, more precisely the effect of cement paste, on the concrete temperature.

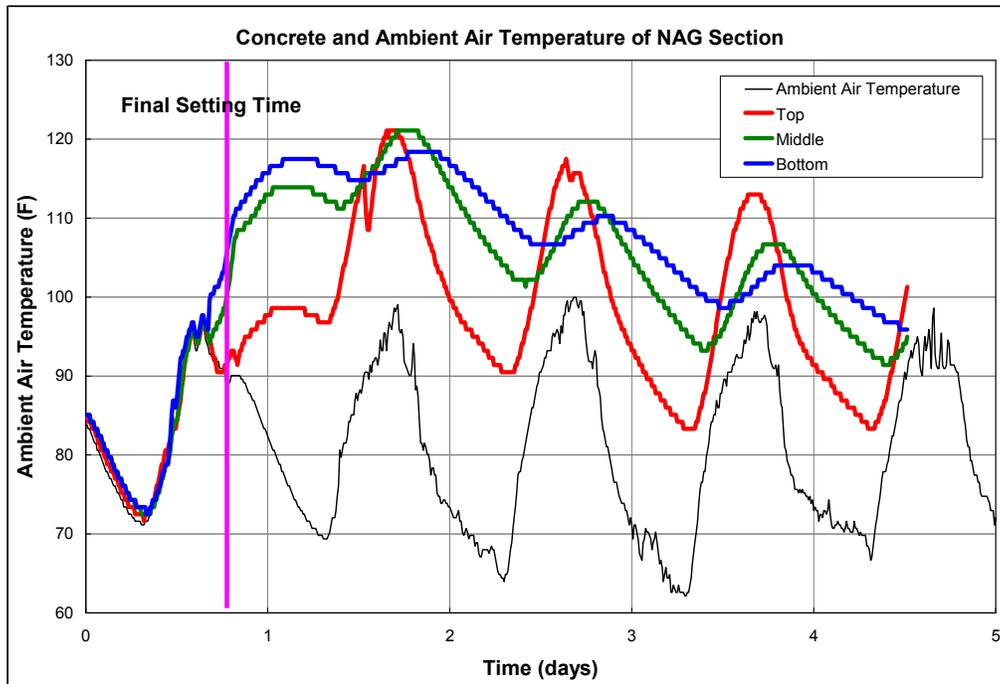


Figure 3.27: Concrete and Ambient Air Temperatures of NAG Section Measured by I-Button

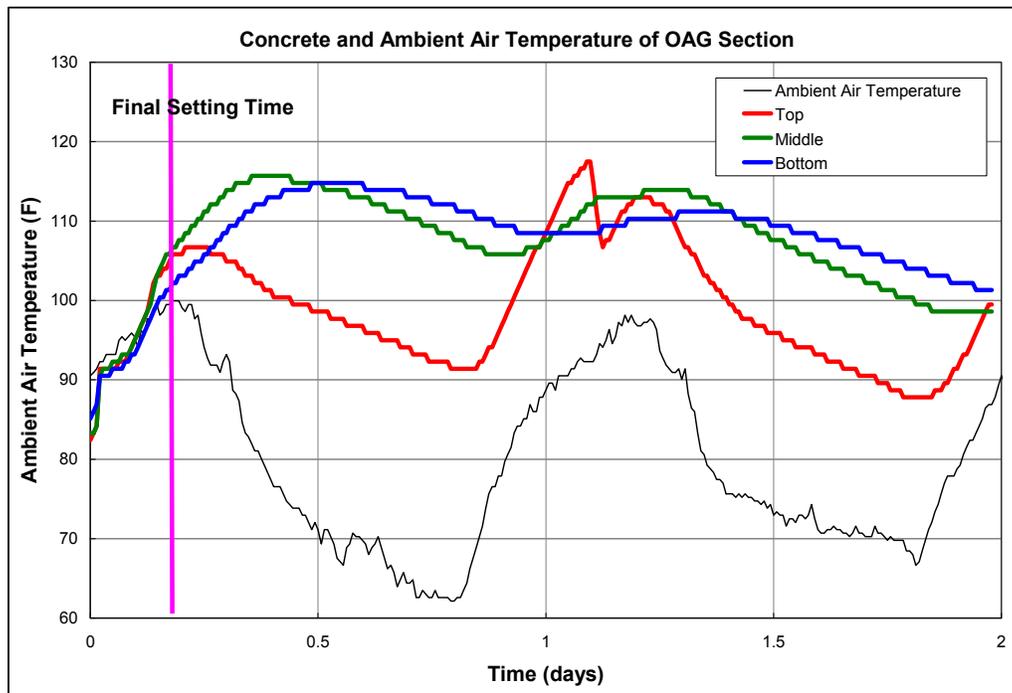


Figure 3.28: Concrete and Ambient Air Temperatures of OAG Section Measured by I-Button

### 3.3.2 Relative Humidity

Variations in relative humidity (RH) in concrete induce the concrete stresses by concrete volume changes such as drying shrinkage and swelling. In this study, RH was measured in the Wichita Falls test sections only. For the RH measurement in NAG section, gages called “hygrobutton” were installed at surface, 0.5-inch, 1-inch from surface, mid-depth, and 1-inch from bottom of concrete slab. In OAG section, they were installed at surface, 0.5-inch, and 1-inch from the surface. Hygrobuttons were also installed in two stress-free cylinders that were used for the measurement of in-situ COTE. The applicability of hygrobuttons to the measurements of RH in concrete has not been positively confirmed. Rather, they were tried in TxDOT Research Project 0-1700 and they are quite convenient for use in the field, since they have memory chips within the gages so that external data logger is not needed.

Figure 3.29 shows the RH values at various depths in the NAG section. It shows the concrete RH at surface, 0.5-inch, and 1-inch depth appears to decrease while at the mid-depth, the variations in RH are minimal. It is also noted that the RH at the bottom of the slab actually increased over 100 % during the measurement period. RH values from the stress-free cylinder that is completely sealed are indicated by “autogenous.” RH values from the stress-free cylinder that allows moisture exchange with surrounding concrete are indicated by “independent.” It is noted that the RH values from both stress-free cylinders increased above 100 % RH over the measurement period. Since the stress-free cylinders were installed at the mid-depth, it was expected that the “independent” RH values would be quite close to the RH values at the mid-depth. Figure 3.29 shows that’s not the case, and the cause for this discrepancy couldn’t be identified. Getting more than 100% RH has been one of the issues with hygrobuttons. The test results of the OAG section, presented in Figure 3.30, show a similar trend. The comparison of RH values in NAG and OAG section at the same depth shows little difference, which could indicate that (1) the RH measurements using hygrobutton

are repeatable, even though the values could go above 100 %, and (2) coarse aggregate contents do not appear to have effects on RH in concrete.

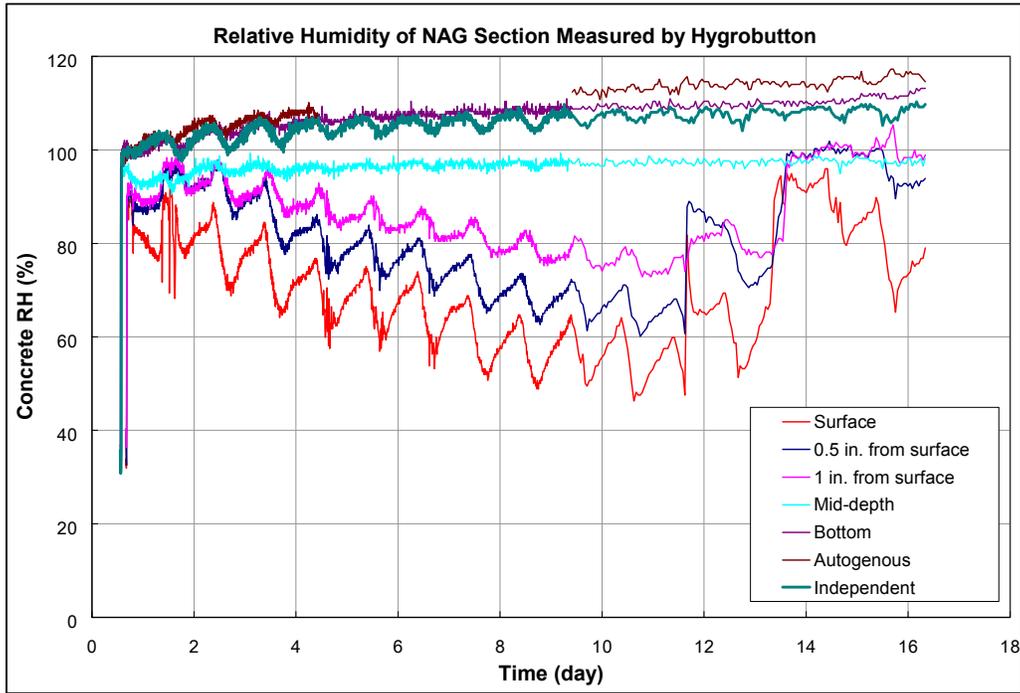


Figure 3.29: Concrete Relative Humidity of NAG Section Measured by Hygrobutton

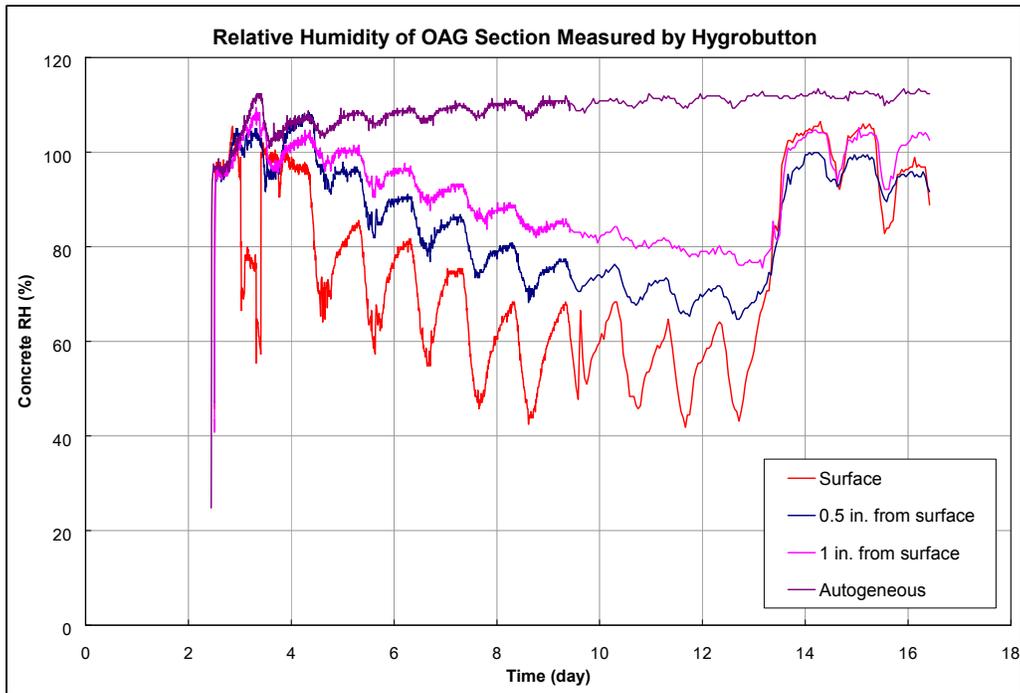


Figure 3.30: Concrete Relative Humidity of OAG Section Measured by Hygrobutton

### 3.3.3 Crack Spacing and Crack Width

Since CRCP usually provide good long-term (30 to over 40 years) performance with structural distresses taking place many years after the construction, it's quite a challenge to evaluate the effect of coarse aggregate volume in concrete on CRCP performance within the timeframe of this research project. It has been reported that transverse crack spacing and crack widths are good performance indicators of CRCP(4). In this study, efforts were made to obtain information on transverse crack spacing and crack widths.

#### Wichita Falls Test Section

Crack spacing of the Wichita Falls test section was measured on September 1, 2, 7, and 14, 2005. An additional survey was made for 200 feet of the induced crack on September 26 and October 12, 2005.

Table 3.4 summarizes the crack survey results. "Distance" denotes the distance from the construction joint and the locations of induced cracks for NAG and OAG sections are 2,275 and 4,462 ft, respectively. It is shown that, even though the length of each distance unit for crack spacing are not uniform, variability exists in crack spacing within NAG and OAG sections, which is expected since transverse cracking in CRCP is somewhat random. The overall average crack spacing for NAG section was 10.53 ft while that of OAG section was 9.20 ft. Transverse crack spacing in NAG section is 1.33-ft larger than that in OAG section, which is somewhat contrary to what's expected. Since OAG concrete had lower COTE and drying shrinkage along with higher flexural strength, it is expected that there will be fewer transverse cracks in OAG section, with larger crack spacing. This signifies the effects of other variables such as setting temperature on transverse crack spacing, and simple comparisons of crack spacing between the two sections to evaluate the effects of coarse aggregate volume on crack spacing may not be valid. It is recalled that the crack spacing values were obtained by combining the crack spacing measured at different times, even though the survey time for crack spacing on 200-ft length in both sections was the only major difference.

Figure 3.31 shows the number of cracks and average crack spacing for 200 feet of the induced crack location. The number of cracks in the OAG and NAG sections was 23 and 24, respectively. The average crack spacing of the OAG and NAG sections was 8.87 ft and 8.17 ft, respectively. Since the COTE of OAG concrete (4.9 microstrain/°F) was lower than that of NAG concrete (5.7 microstrain/°F), it is expected that, with all the other conditions being identical, fewer cracks will form in the OAG section than in the NAG section. On the other hand, other factors such as setting temperatures and drying shrinkage also have substantial effect on cracking. Recall that the same cement content as well the same water-cement ratio were used for NAG and OAG concretes and the difference in coarse aggregate volume between them is quite small (7.7 %). It is not feasible to positively quantify the effects of coarse aggregate volume on transverse crack spacing.

Figures 3.32 and 3.33 illustrate transverse cracks within 200 feet of the induced crack surveyed on October 12, 2005, on the NAG and OAG sections, respectively. The variability in the transverse crack spacing is visually evident.

Efforts were made to measure crack width in both NAG and OAG sections. Vibrating wire strain gages (VWSG) were installed and transverse cracks were induced at the locations of VWSGs. Transverse cracks formed at those locations. However, as can be seen in Figure 3.38, additional transverse crack formed within 1.5-ft of the induced crack in the NAG section. Even though transverse crack spacing does not seem to have direct impact on crack width, two cracks within 1.5-ft could alter the crack widths. Also note that the two transverse cracks at each side of the induced crack in the OAG section are further apart from the

induced crack. It was not feasible to accurately evaluate the effect of coarse aggregate volume in concrete on crack widths. On retrospect, three transverse cracks should have been induced, with VWSGs installed in the middle induced crack.

**Table 3.4: Summary for Crack Survey Results**

<b>Section</b>	<b>Distance (ft)</b>	<b>Number of Cracks</b>	<b>Average Crack Spacing</b>	<b>Construction Date</b>	<b>Last Surveyed Date</b>	<b>Age at Surveyed Date</b>
NAG	0~400	32	11.9	8/25/05	9/14	20
	800~1000	19	10.83	8/25/05	9/14	20
	1800~2100	28	10.75	8/29/05	9/14	16
	2075~2175	7	13.78	8/29/05	9/14	16
	2175~2375	24	8.17	8/29/05	10/12	44
	2375~2475	8	11.6	8/29/05	9/14	16
	2400~2700	22	13.51	8/29/05	9/14	16
	2900~3300	47	8.61	8/30/05	9/14	15
	3300~3700	41	9.99	8/30/05	9/14	15
OAG	4262~4362	12	8.07	8/31/05	9/14	14
	4362~4562	23	8.87	8/31/05	10/12	42
	4562~4662	11	9.29	8/31/05	9/14	14
	5800~6200	41	9.76	9/1/05	9/14	13

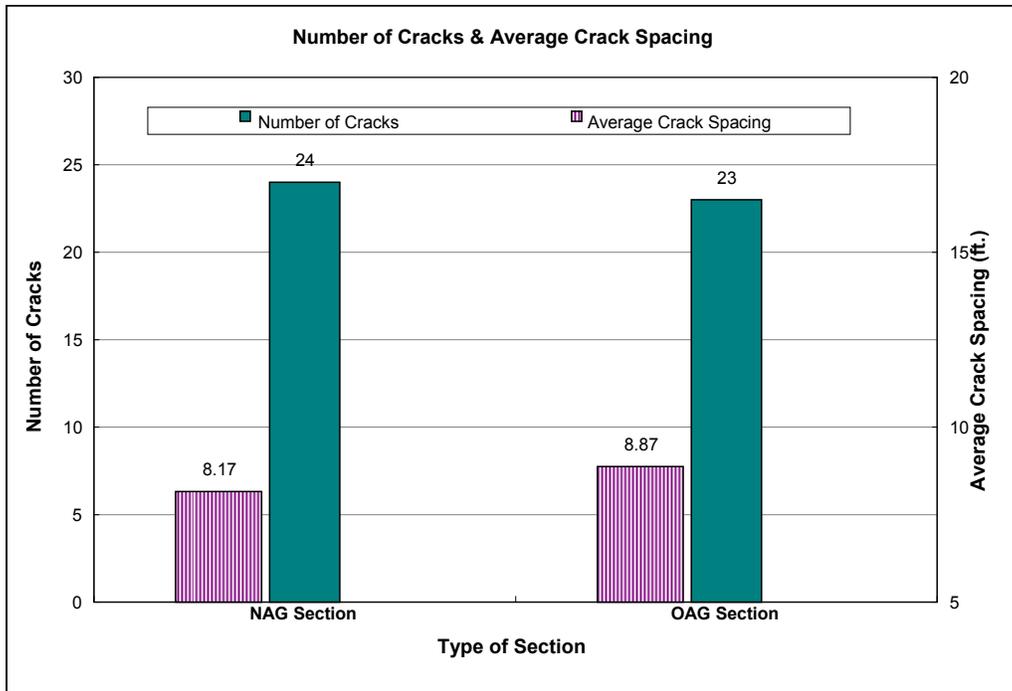


Figure 3.31: Number of Cracks and Average Crack Spacing

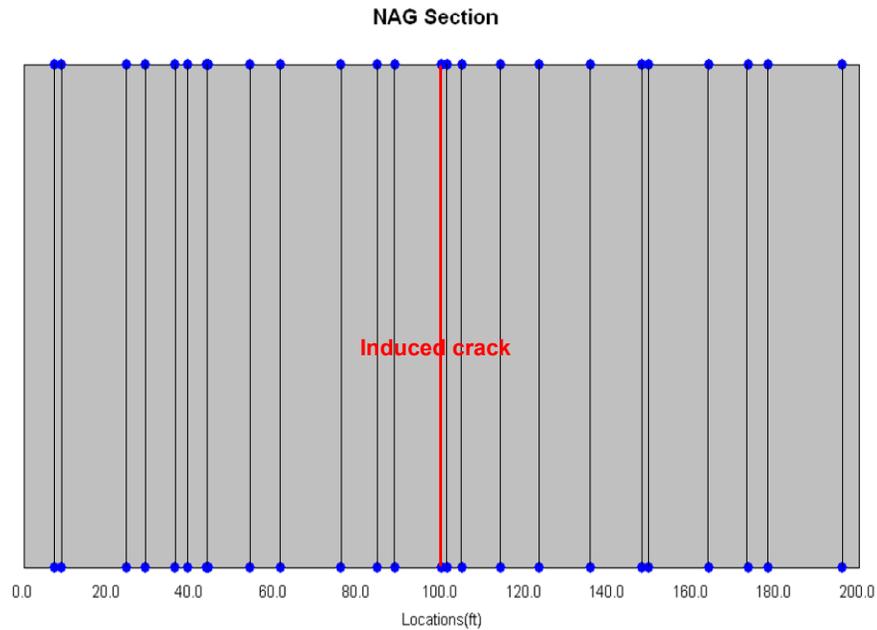


Figure 3.32: Crack Spacing of Wichita Falls NAG Test Section, US 287

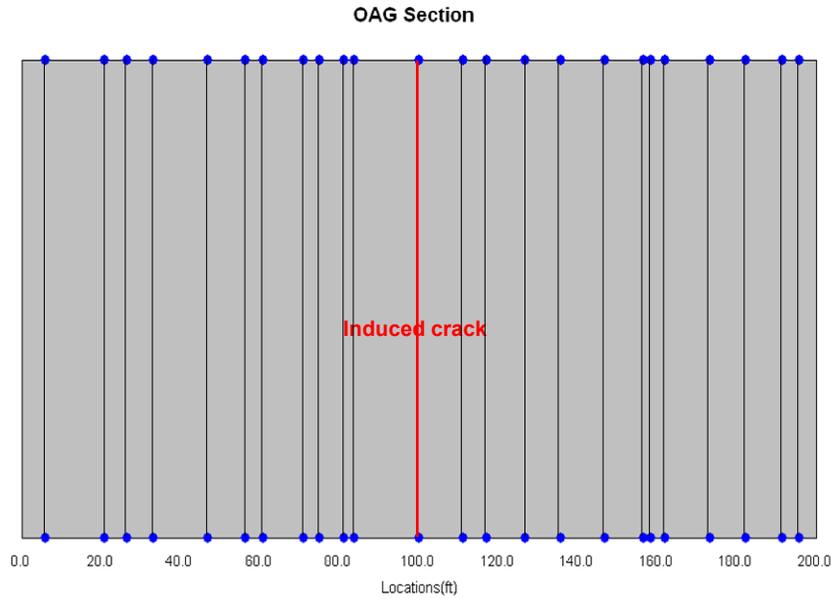


Figure 3.33: Crack Spacing of Wichita Falls OAG Test Section, US 287

**Fort Worth Test Section**

The crack survey of this test section was conducted on November 15, 2006, 57 days after construction. This survey was performed up to 310 ft from the transverse construction joint. The average crack spacing was 11.1. Figure 3.34 shows the transverse crack distribution pictorially. It shows variability in crack spacing. Only two transverse cracks formed within 100-ft from the transverse construction joint, which results in 50-ft of average crack spacing. This is due to the less restraint on concrete volume changes near the transverse construction joint area.

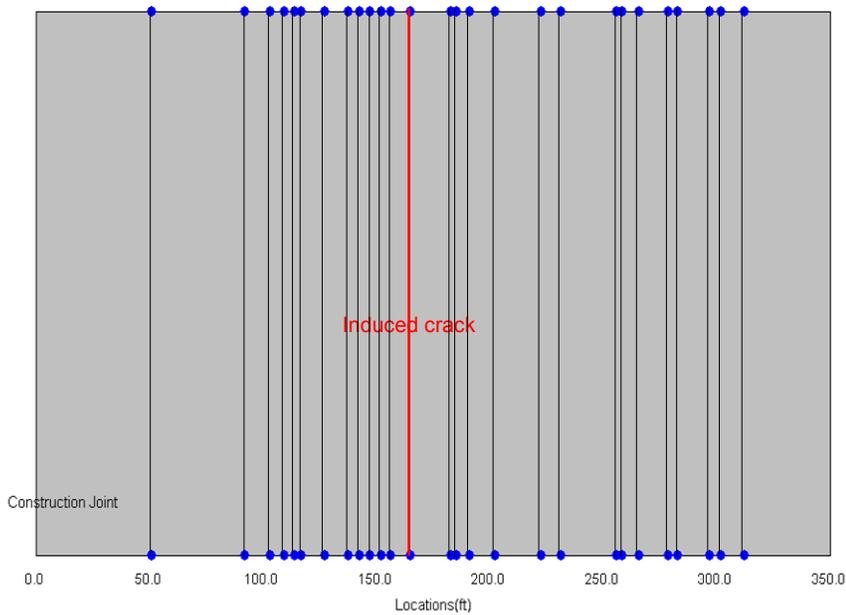
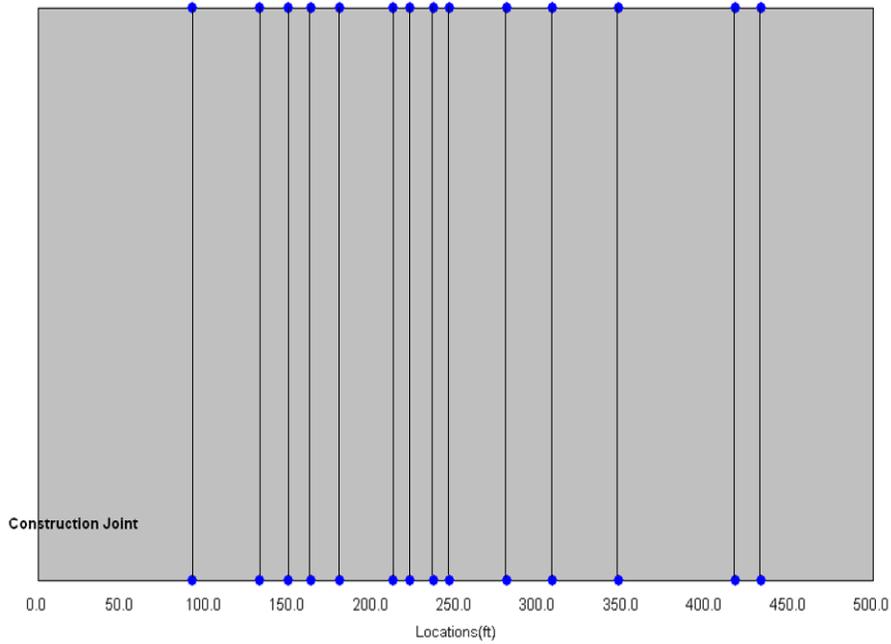


Figure 3.34: Crack Spacing of Fort Worth Test Section, SH 114B

**Dallas Test Section**

The crack survey of this test section was conducted on February 1, 2007, 77 days after the construction. This survey was performed up to 430 ft from the transverse construction joint. The average crack spacing was 31.0, the largest among all test sections, which is considered partly due to the lowest setting temperature of this concrete. Figure 3.35 shows the transverse crack distribution pictorially.



*Figure 3.35: Crack Spacing of Dallas Test Section, SH 121*



## Chapter 4. Conclusions

In an effort to identify the benefits of using optimized aggregate gradation (OAG) in paving concrete, three test sections were constructed; each in Wichita Falls (US287 northbound), Fort Worth (SH114B northbound), and Dallas (SH121 westbound) Districts. Both normal aggregate gradation (NAG) and OAG sections were placed side by side in Wichita Falls project. Only OAG sections were placed in Fort Worth and Dallas projects.

The NAG and OAG concretes used in Wichita Falls project had the same cement contents and water cement ratios. Ideally, the use of OAG could make the use of less cement to achieve the same strength. Since this was the first project utilizing OAG in Texas, the contractor was conservative, which explains identical cement contents and water-cement ratios for both NAG and OAG concretes. The only difference between the two mixes was coarse aggregate volume (0.95 cf per cy of concrete) and resulting fine aggregate volume as well. Since the strength-related concrete properties depend on water-cement ratio, large differences in strength-related concrete properties are not expected between these mixes.

Fresh and hardened concrete properties of NAG and OAG concretes in Wichita Falls project and OAG concrete in Fort Worth and Dallas projects were evaluated. For the evaluation of pavement performance as affected by the use of NAG and OAG concretes, efforts were made to measure early-age CRCP performance indicators such as crack spacing and crack widths. Based on the findings from this study, the following conclusions are made:

1. Evaluations made on the properties of fresh concrete in Wichita Falls project do not show substantial difference between NAG and OAG concretes. There was little difference in workability, bleeding and setting characteristics between OAG and NAG concretes. The reason for the little difference is considered due to the little difference in the two mix designs as explained above. In the Fort Worth and Dallas projects, direct comparisons between NAG and OAG concretes were not feasible due to the absence of NAG concrete sections. However, fresh concrete material properties measured in those two projects are within the normal ranges.

2. In Wichita Falls project, differences in strength-related properties between NAG and OAG concretes, such as compressive and flexural strength and modulus of elasticity, were not large. The exception is flexural strength. OAG concrete provided about 9 % higher flexural strength than NAG concrete, even though the same cement contents and water-cement ratios were used for both mixes.

3. In the Wichita Falls project, OAG concrete produced lower in-situ coefficient of thermal expansion (COTE) than NAG concrete. The in-situ COTE of OAG concrete was 4.9 microstrain/°F and that of NAG concrete was 5.7 microstrain/°F. It appears that the larger coarse aggregate volume in OAG concrete than in NAG concrete, even though it was only 7.7 % more in OAG concrete, contributed to this difference in in-situ COTE.

4. In Wichita Falls project, in-situ drying shrinkage obtained from stress-free cylinders show larger in-situ drying shrinkage at NAG section than at OAG section, which is somewhat expected. It is because OAG concrete has more coarse aggregate volume and less cement paste than NAG concrete, even though the difference is not large, and it is cement paste that shrinks, not coarse aggregates.

In Dallas project, lightweight aggregate was used as intermediate aggregate. The effect of lightweight aggregate on drying shrinkage was evaluated. One concrete specimen was made with concrete as received from the test section. The other specimen was made with concrete after lightweight aggregates were removed from the concrete. The shrinkage of concrete containing lightweight aggregate was about 20 percent lower than that of the

concrete without. This could be due to the so-called “internal curing,” which means that lightweight aggregates, thanks to their high void contents, absorb water and when the surrounding paste becomes dry, they release moisture to the cement paste, thus helping keep the concrete moist and minimizing drying shrinkage.

5. In Wichita Falls project, the overall average crack spacing for NAG section was 10.53 ft while that of OAG section was 9.20 ft. Transverse crack spacing in NAG section is 1.33-ft larger than that in OAG section, which is somewhat contrary to what’s expected. Since OAG concrete had lower COTE and drying shrinkage along with higher flexural strength, it is expected that there will be fewer transverse cracks in OAG section, with larger crack spacing. This signifies the effects of other variables such as setting temperature on transverse crack spacing, and simple comparisons of crack spacing between the two sections to evaluate the effects of coarse aggregate volume on crack spacing may not be valid.

6. Efforts were made to measure crack width in both NAG and OAG sections in Wichita Falls project. Vibrating wire strain gages (VWSG) were installed and transverse cracks were induced at the locations of VWSGs. Transverse cracks formed at those locations. However, additional transverse crack formed within 1.5-ft of the induced crack in the NAG section, while the two transverse cracks at each side of the induced crack in the OAG section were further apart from the induced crack. It was not feasible to accurately evaluate the effect of coarse aggregate volume in concrete on crack widths.

7. Long-term CRCP performance as affected by the use of NAG and OAG concretes was not evaluated in this study. According to MEPDG and CRCP-11, CRCP with larger crack spacing provides better long-term performance. In the Wichita Falls project, transverse crack spacing in NAG section was larger than that of OAG section, and the analysis by MEPDG and CRCP-11 will show better performance of NAG section than OAG section. There are limitations in the ability of the current theoretical models to accurately predict long-term CRCP performance. The long-term performance of the sections in Wichita Falls project will be monitored under TxDOT’s rigid pavement database project.

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## **Appendix A: Recommended Modifications to Special Provision**

Observations of paving operations and research findings from this study did not identify reasons to modify the current Special Provision to Item 421 for the use of optimized aggregate gradation for Portland cement concrete paving.



# Appendix B: Most Current TxDOT Special Provision to Item 421 for the Use of Optimized Aggregate Gradation for Concrete Paving

2004 Specifications

## SPECIAL PROVISION

### 421---XXX

#### Hydraulic Cement Concrete

For this project, Item 421, "Evaluation of Early-Age Pavement Behavior," of the Standard Specifications, is hereby amended with respect to the clauses cited below, and no other clauses or requirements of this Item are waived or changed hereby.

**Article 421.2. Materials, Section E. Aggregate, Section 1. Coarse Aggregate.** The last sentence of the fourth paragraph is voided.

**Article 421.2. Materials, Section E. Aggregate, Section 2. Fine Aggregate.** The fifth paragraph is voided and replaces by the following:

Acid insoluble (%) =  $\{(A1)(P1)+(A2)(P2)\}/100$   
where:

*A1* = acid insoluble (%) of aggregate 1

*A2* = acid insoluble (%) of aggregate 2

*P1* = percent by weight of aggregate 1 of the fine aggregate blend

*P2* = percent by weight of aggregate 2 of the fine aggregate blend

**Article 421.2. Materials, Section E. Aggregate, Section 2. Fine Aggregate.** The eighth paragraph is voided and replaces by the following:

For all classes of concrete, provide fine aggregate with a fineness modulus between 2.30 and 3.10 as determined by Tex-402-A.

**Article 421.2. Materials, E. Aggregate.** is supplemented by the following:

**4. Optimized Aggregates Gradation.** Provide coarse and fine aggregates meeting the requirements in Sections 421.2.E.1 and 421.2.E.2, unless otherwise stated.

Coarse and fine aggregate gradation charts shown in Sections 421.2.E.1 and 421.2.E.2 will not apply for aggregates used for optimized concrete mix designs.

Sample and test coarse and fine aggregates as specified below.

**Contractor Material QC Sampling and Testing Frequencies**

<b>Material</b>	<b>Test Method</b>	<b>Frequency<sup>1</sup></b>
Coarse Aggregate	Sieve Analysis Tex-401-A	Minimum of 2 per day
Fine Aggregate	Sieve Analysis Tex-401-A	Minimum of 2 per day

1. 1 test per day for concrete pours less than 200 cubic yards.

When multiple coarse aggregate grades are blended to produce an optimum gradation, use the following equation to determine the loss by decantation for the combined coarse aggregates.

$$\text{Total Loss by Decantation (\%)} = \{\sum(P_i)(D_i)\}/100$$

Where:

$P_i$  = percent of coarse aggregate<sub>i</sub> of the combined coarse aggregate

$D_i$  = loss by decantation of aggregate<sub>i</sub>

Submit sieve analysis reports showing the cumulative combined percent passing, the cumulative combined percent retained, and the combined percent retained, including all standard sieves starting with the nominal maximum aggregate size to the No. 200 sieve. Use the following charts, developed in accordance with Tex-470-A, “Optimized Aggregate Gradation for Portland Cement Concrete Mix Designs,” to determine the optimum combined aggregate gradation:

- Coarseness Factor Chart,
- Percent Retained Chart, and
- 0.45 Power Chart.

**Article 421.4. Construction, Section A. Classification and Mix Design, Section 6. Mix Design Options.** The second paragraph is voided and replaced by the following:

For concrete classes not identified as structural concrete in Table 5 and designed using less than 520 lb. of cementitious material per cubic yard, use one of the mix design Options 1-8 shown below, except that Class C fly ash may be used instead of Class F fly ash for Options 1, 3, and 4 unless sulfate-resistant concrete is shown on the plans.

**Article 421.4. Construction, Section A. Classification and Mix Design, Section 6. Mix Design Options, Section c. Option 3** is voided and replaced by the following:

**c. Option 3.** Replace 35 to 50% of the cement with a combination of Class F fly ash, GGBFS, UFFA, metakaolin, or silica fume. However, no more than 35% may be fly ash, and no more than 10% may be silica fume.

**Article 421.4. Construction, Section A. Classification and Mix Design** is supplemented by the following:

**7. Concrete Mix Designs with Optimized Aggregate Gradation.** When shown on the plans, design Class P concrete, meeting the strength requirements as specified in Item 360, using optimized aggregate gradations meeting the requirements of Section 421.E.4.

Submit proposed concrete mix design information including:

- material source information,
- material property information,
- combined aggregate sieve analysis and charts according to Tex-470-A, “Optimized Aggregate Gradation for Portland Cement Concrete Mix Designs,”
- concrete mix design proportions, and
- results from trial batches.

During concrete production, monitor the aggregate gradation by plotting the results of each sieve analysis on the coarseness factor chart, percent retained chart, and 0.45 power chart in accordance with Tex-470-A “Optimized Aggregate Gradation for Portland Cement Concrete Mix Designs.” Perform the first sieve analysis before each production day.

Adjust aggregate proportions, if necessary, to keep the coarseness factor and workability factor plotted within the workability box on the coarseness factor chart as described in Tex-470-A, “Optimized Aggregate Gradation for Portland Cement Concrete Mix Designs.”

Adjustments to the aggregate proportions that also require adjustments to water or cementitious material contents will require development of new concrete mix proportions and results from trial batches.