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Optimizing Concrete Pavement Type Selection Based on Aggregate Availability

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16. Abstract
Design concept and structural responses of jointed plain concrete pavement (CPCD) and continuously reinforced concrete pavement (CRCP) are quite different. In CPCD, concrete volume changes are allowed to a full extent, and accommodations are made to ensure good load transfer at discontinuities, i.e., transverse contraction joints. On the other hand, concrete volume changes are restrained to a significant degree in CRCP by longitudinal reinforcement and base friction. Because of this vastly different behavior between the two pavement types, concrete with a high CoTE is not an ideal material for CRCP. In other words, the performance of CRCP with a high CoTE concrete will be compromised, with resulting spalling. Concrete with a high CoTE should be used for CPCD if at all possible. This study investigated the correlation between spalling and concrete CoTE. CRCP sections with severe spalling were identified. Sections with no spalling and delamination distresses were also identified. A minimum of two cores were taken from those sections, and CoTE and modulus of elasticity were evaluated. There was an excellent correlation. Concrete material properties of selected coarse aggregates were extensively evaluated in the laboratory. Also, an in-depth analysis was made of the life-cycle cost of the pavement with coarse aggregates from different sources. Concrete with a CoTE larger than 5.5 microstrain per °F is quite prone to severe spalling. The findings from this study indicate that, if concrete with a CoTE greater than 5.5 microstrain/°F is used in CRCP, the risk of severe spalling increases substantially. Accordingly, if the only coarse aggregate type available locally produces concrete with a CoTE greater than 5.5 microstrain/°F, it is strongly recommended that this aggregate not be used in CRCP. Instead, the use of CPCD should be considered. Whether low CoTE aggregates need to be brought in for use in CRCP, or a locally available high CoTE aggregates is utilized in CPCD, should be based on the local experience with CPCD performance.

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Chapter 1 Introduction

Currently, two types of rigid pavement are used in TxDOT. One is jointed plain concrete pavement (CPCD) and the other is continuously reinforced concrete pavement (CRCP). Even though both pavement types use the same materials on the surface layer and have similar pavement structures, the behavior and structural responses of the two pavement types are vastly different. In short, concrete volume changes in CPCD due to temperature and moisture variations are allowed and provisions made to ensure good load transfers at discontinuities (transverse contraction joints). On the other hand, volume changes in CRCP are severely restrained by longitudinal reinforcement and base friction. Because of this difference in pavement behavior, concrete with high volume change potential, i.e., concrete with a high coefficient of thermal expansion (CoTE) is not suitable for CRCP. In other words, there is a compatibility issue between rigid pavement type and Portland cement concrete (PCC) material properties. Ignoring this compatibility issue results in less than optimum rigid pavement type.

Coarse aggregate occupies about 40 percent of PCC volume, and thus has substantial effects on PCC properties, such as CoTE, modulus of elasticity, and drying shrinkage. On the other hand, its effects on strength are not as significant. According to the FY 2010 TxDOT PMIS, TxDOT has 12,345 lane miles of CRCP and 1,399 punchouts. All the distresses recorded as punchouts in the Amarillo, Childress, Dallas, Fort Worth, Houston, Lubbock, and Wichita Falls districts were visually investigated under the TxDOT rigid pavement database project (0-6274). The findings indicated that about half of the distresses recorded as punchouts in PMIS were actually large spalling and distresses caused by delaminations. They were not due to the structural deficiency of the pavement system. In Texas, spalling and delaminations normally develop in CRCP when certain coarse aggregate types are used. TxDOT recognized this, and has sponsored several research studies since the mid-1980s to address this issue, with no solutions obtained. On the other hand, these coarse aggregate types have been used in CPCD with almost no spalling issues. As stated earlier, the fundamental design concepts and structural behaviors of CRCP and CPCD are vastly different, and these coarse aggregate types are not compatible with CRCP behavior, resulting in spalling and delaminations, although they are quite compatible with CPCD behavior, and the performance of CPCD with those aggregates has been satisfactory.

Proper selection of PCC pavement type based on coarse aggregate type will enhance overall PCC pavement performance, thus minimizing maintenance and repair costs. The primary objective of this research was to develop specific requirements for design and construction for CPCD with local aggregates based on identification of coarse aggregate properties associated with spalling/delamination in CRCP and characteristics of local coarse aggregates.

There are four primary technical objectives in this project:

(1) Investigate characteristics of locally available coarse aggregate types, along with the performance of CPCD and CRCP with certain coarse aggregates.
(2) Identify locally available coarse aggregate sources for Atlanta, Houston, Amarillo, Paris, and Wichita Falls Districts. Provide cost analysis for those aggregate sources.

(3) Develop guidelines for the selection of optimum rigid pavement types based on traffic level/functional classification, base supports, and locally available materials.
(4) Provide specific requirements for design and construction when using these local coarse aggregates in CRCP and/or CPCD.

This report addresses the technical objectives in the following chapters:

Chapter 2 describes the research conducted at Texas Tech University to identify concrete properties affected by coarse aggregate properties that are closely related to spalling and delaminations in CRCP. The effort in this phase is based on the characterizations of material properties such as CoTE and modulus of elasticity of concrete from CRCP sections with spalling and delaminations and not based on theoretical analysis.

Chapter 3 documents the studies conducted at the University of Texas at Austin and Texas State University at San Marcos to evaluate the characteristics of concrete with coarse aggregate types specifically requested by TxDOT. Also, the guidelines developed for the optimum pavement type selection are described.

Chapter 4 describes the work conducted on cost analysis of coarse aggregate, along with the descriptions on the life-cycle cost analysis. This work was conducted by the research team at Texas State University at San Marcos.

Chapter 5 provides specific requirements in the form of materials/construction specifications and/or design standards for CPCD with coarse aggregates that are not suitable for CRCP.

Chapter 6 describes the conclusions and recommendations.

Chapter 2 Coarse Aggregate Properties Associated with Spalling and Delaminations

2.1 Literature Review

Spalling in concrete pavements has been studied since the early 1960s. Initial research studies focused on the identification of factors affecting spalling, discussed the possible mechanisms of spalling, and addressed the investigation of crack spalling and joint spalling. Zollinger and Barenberg (1990) defined crack spalling as "the breakdown of the pavement along the cracks leading to the loss of concrete and the disintegration of the load transfer mechanism." Roadway Maintenance Evaluation User's Manual developed for the State of Texas (Epps et al. 1974) defined spalling as the breakdown or disintegration of slab edges at joints or cracks or directly over reinforcing steel, usually resulting in the removal of sound concrete. It also classifies spalling into three groups: 1-15%, 16-50% and over 50%, depending on the number of spalled cracks or joints in the concrete pavement.

Usually spalls are classified based on their depth, length, and width. Spalling of CRCP is recorded in terms of the number of spalled cracks in the Texas Pavement Evaluation System (PES). A crack with a width of 1 in or more and a length of at least 12 in can be considered as spalled. This distress classification considers only transverse cracks. Recording of spalling data both numerically (number of spalled cracks/joints and length of cracks/joints spalled) and qualitatively (low, medium, or high severity) is suggested by the Pavement Distress Identification Manual of the Strategic Highway Research Program (SHRP 1994). Spalling severity is classified as a function of the depth, width and frequency of spalling.

2.1.1 Field Observations

In a study conducted under the National Cooperative Highway Research Program (NCHRP 1979), spalling was categorized mainly into two groups, minor and severe. Field observations made in this study indicated that most spalling was minor and observed along transverse cracks. This was frequently related to the surface widening of cracks as a result of fracturing of the mortar in the concrete pavement surface along the crack face. This type of spalling was believed to consist of flaking of the mortar in the concrete mix on either side of the crack. It was noted that minor spalling remained stable with no signs of a progressive form of deterioration or structural consequences.

Studies by Guiterrez de Velasco and McCullough (1981) revealed that severe spalling is usually produced due to construction operations. Further, they proposed that spalling is influenced by traffic, pavement age, and location along the transverse crack (distance from the pavement edge). The primary causes of spalling proposed by McCullough et al (1979) are:

1. Entrapment of road debris in cracks, which may cause a stress concentration under a buildup of compressive force
2. A combination of shear and tensile stress under wheel load
3. Poor concrete at the surface due to overworking during the finishing process

Another finding is that even though minor spalling is a stable form of distress, in general, spalling increased with traffic and age of concrete. Further, it was noted that any correlation between severe spalling and minor spalling or crack spacing did not exist, although spalling was observed to increase with increase in crack width.

The influence of coarse aggregate type on spalling was noted by Shelby and McCullough in 1960. McCullough et al. in 1979 noted that concrete made with limestone coarse aggregate showed less spalling than concrete with siliceous river gravel coarse aggregate. The lower modulus of elasticity, higher concrete strength, and better bonding characteristics of the limestone concrete were considered to be responsible for lower frequency of spalling. Spalling was noted as discontinuities formed during the process of crack propagation as the forming crack takes the path of least resistance by McCullough et al. (1975). In another study (NCHRP 1979), deeper spalls were related to structural weakness, while shallow and wide spalls were related to weakened horizontal planes in the surface of the concrete. Also, it was suggested that stress concentrations induced by load and deflection of pavement under traffic could lead to the development of spalling. Tayabji and Colley (1986) noted that spalling can result from incompressible material deposited in the cracks under expansive strains during temperature increases, based on finite element analysis. They also indicated that restraint to volume change from temperature variation through the slab depth can cause spalling due to stresses caused by adjacent slabs butting together. The use of joint filler to retard the development of crack spalling by inhibiting edge raveling, admission of incompressible material, and corrosion of reinforcement was suggested by Wright (1981).

Studies conducted in Minnesota (Tracy 1978) on two projects noted that spalls usually occurred in the wheel path region. It was also observed in the same study that most of the spalls extending to the reinforcement occurred due to chloride-induced corrosion of the reinforcement. However, spalling over reinforcement was not a major problem in CRCPs in a study by Zollinger and Barenberg (1990).

Extensive bending stress analysis results were reported by Zollinger and Barenberg (1990). They noted that the loss of bending stiffness at a transverse crack was related to spalling in CRCPs. They indicated that considerable shear and normal stresses are created at the face of the transverse cracks that have potential for spalls to be developed. Also, they noted that the crack width and depth to reinforcement influence the stiffness of the concrete, and these will also influence the stresses in spalls. Additionally, they indicated that pavement support condition contributes to the spall stresses.

In addition to spalling in CRCPs, research studies have focused on spalling in jointed concrete pavements. Spalling data from the Michigan Road Test was analyzed by Smith et al. (1990) and following conclusions were derived:

1. The primary cause of spalling is the incompressible material in the joints, and incompressible material can be prevented from infiltrating the joints for a considerable period of time by preformed sealants.
2. D-cracking has a dramatic effect on joint spalling.

Further observations were made from the results of the analysis of the Long Term Pavement Performance (LTPP) database for spalling in jointed pavements. These results were noted in a report published by Strategic Highway Research Program (SHRP 1994) and consisted of the following observations:

1. Joint seal type has a considerable influence on spalling in some studies, while in others it has been found to be insignificant.
2. Due to a high number of freeze-thaw cycles, spalling may tend to increase in jointed plain concrete pavements.

2.1.2 Modeling of Spalling

Several models were developed to predict spalling in terms of a number of input variables, with age being a primary variable. One of the available models for JCP is the PEARDARP model (Van Wijk 1985, Kopperman 1986) shown in Eq. 1.

$$F_s = 1 - e^{-\propto(J-8)} \tag{1}$$

where, F_s = fraction of joints spalled;

$\alpha = 0.0000162A^{3.0806}$;

J = transverse joint spacing in feet;

and A = age of pavement in years.

However, the PEARDARP model is not able to predict spalling satisfactorily and actually over-predicts transverse joint spalling (Smith et al. 1990). Also, the study by Smith noted that spalling occurred on pavements irrespective of age and joint spacing. They also observed that spalling in pavements was associated with materials or other problems such as aggregates under unfavorable climate condition, corrosion of dowel bars, or locking up of the joints. Consequently, based on the spalling data from the Michigan Road Test, Smith et al proposed Equations 2 and 3 for the spalling in jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP), respectively.

$$\text{JTSPALL} = AGE^{2.178}(0.0221 + 0.5494\,DCRACK - 0.0135\,LIQSEAL - 0.0419\,\text{PREFSEAL} + 0.0000362\,\text{FI}) \tag{2}$$

$$\text{JTSPALL} = AGE^{4.1232}(0.00024 + 2.69E-05\,DCRACK + 3.07E-04\,REACTAGG \tag{3}$$

$-3.3E\text{-}05\ LIQSEAL - 0.0003\ PREFSEAL + 1.4E\text{-}07\ FI)$

where, JTSPALL = number of medium-high severity joint spalls per mile;
AGE = age of pavement since original construction in years

If no D-cracking exists, DCRACK = 0. Otherwise, it is 1. If no liquid sealant exists in the joint, LIQSEAL = 0. Otherwise, it is 1. ; PREFSEAL = 0, if no preformed compression seal exists, and 1, if a preformed compression seal exists; FI = freezing index in degree days below freezing; and REACTAGG = 0, if no reactive aggregate exists, and 1, if reactive aggregate exists. The model for JPCP (Eq. 2) had a coefficient of determination (R^2) of 0.59 and a standard error of estimate (SEE) of nine joints per km. The model for JRCP (Eq. 3) had an R^2 of 0.47 and an SEE of eight joints per km.

Two more spalling predictions models for jointed concrete pavements are derived from the sensitivity analysis of selected pavement distresses (SHRP 1994). The models shown in Eqs. 4 and 5 were developed for JPCP and JRCP, respectively, by statistical analysis using variables likely to affect spalling.

$$\text{SPALLJP} = 9.79 + 10.09\,[-1.227 + 0.0022\,(0.9853\,AGE + 0.1709\,FT)^2] \quad (4)$$
$$\text{SPALLJR} = -79.0 + 0.604\,(AGE)^{1.5} + 0.129\,(TRANGE)^{1.5} \quad (5)$$

where SPALLJP = predicted mean percentage of transverse joint spalling (all severities) as a percentage of total joints for JPCP; SPALLJR = predicted mean percentage of transverse joint spalling (all severities) as a percentage of total joints for JRCP; TRANGE = mean monthly temperature range (mean maximum daily temperature minus mean minimum daily temperature for each month over a year); FT = number of mean annual air freeze-thaw cycles; and AGE = age since construction (in years).

The analysis for JPCP used 56 survey sections, with a model R^2 of 0.335 and a root mean square error (RMSE) of 11.05% joints. The JRCP analysis used 25 survey sections with a model R^2 of 0.644 and an RMSE of 16.6% joints. Considerable differences were noticed between actual and predicted spalling values for both of the models.

2.1.3 Additional Spalling Data Analysis Results

Spalling data in CRCP from the Texas concrete pavement database was analyzed by Dossey and Hudson (1993). Both minor and severe spalling for each pavement section were included in this database. In this database, minor spalling was defined as edge cracking where the loss of material has formed a spall of one half inch wide or less (Dossey and Wiessmann 1989). The rest of the spalling was categorized as severe spalling. Survey section information such as pavement thickness, coarse aggregate type, subbase treatment, type of subgrade soil, yearly temperature range, average annual rainfall, and the estimated average daily traffic and its projected growth rate were included in the database. The analysis of the spalling data led to the following observations (Dossey and Hudson 1993):

1. Out of the total survey sections, 85 percent were more than five years old at the time of the first survey.
2. No spalling was observed in 72 percent of the surveyed sections.
3. Analysis of variance (ANOVA) was conducted with only two-way interactions considered between each factor as well as age of the pavement. This analysis revealed that the following factors and interactions were the best predictors of spalling in descending order of significance:
    a. interaction between age and coarse aggregate type
    b. interaction between age and rainfall
    c. age itself
    d. interaction between age and subbase type/treatment

2.2 CRCP Spalling and Delaminations in Texas

2.2.1 Spalling Data Analysis and Selection of Field Sections

Extensive field evaluations of the performance of CRCP in Texas clearly indicate that the majority of the distresses is related to the issues of construction/materials quality and coarse aggregate type used, not necessarily related to the deficiencies in the structural capacity of CRCP. Two significant distress types related to coarse aggregate type are spalling and delaminations. When siliceous river gravel (SRG) was used in CRCP as coarse aggregate in concrete, the frequency of spalling increased substantially, especially in the Houston area. TxDOT recognized the significance of this issue for its operations in the area of ride quality and financial impact more than 30 years ago. To develop solutions to address this issue, TxDOT sponsored several research studies starting in the middle of the 1980s, including projects 0-422, 0-1244, 0-1700, 0-4826, 0-5549 and 0-5832. Even though extensive research efforts were made to find solutions to these significant problems and some valuable technical information was obtained, no good solutions were discovered that could have allowed the use of SRG in CRCP. The primary reason was the complicated nature of the problems and the number of factors involved. In the current study, little effort was made to identify measures to prevent spalling and delaminations when SRG is used in CRCP, since sufficient effort was already made in the last 25 years, and it is unlikely that additional efforts in this study would be successful. Rather, the effort focused on identifying material characteristics that lead to spalling and delaminations in CRCP. The study approach was (1) to identify CRCP sections with severe spalling and delamination problems, and those with no problems, (2) to take concrete cores from those sections, (3) to conduct materials testing on those cores, and (4) to analyze the information.

Table 2.1 shows the quantity of spalling (SPL), punchout (PCH), asphalt concrete patch (ACP), and Portland cement concrete patch (PCP) in CRCP based on distress survey results of the 2010 pavement management information system (PMIS). The order of Table 2.1 is arranged by spalling quantity above 10 spallings per 0.5 lane mile. Table 2.1 shows that there is no correlation between spalling and punchout. However, there is some correlation between spalling quantity and PCP quantity. It can be hypothesized that most of PCPs were done to repair spalling.

Sections with severe spalling were selected based on the 2010 PMIS, where a minimum of two concrete cores were taken for materials property evaluations. Table 2.2 shows the information of the sections selected for this study.

Table 2.1 CRCP Spalling Information Based on 2010 PMIS

[SPL ; Spalling, PCH ; Punchout, ACP ; Asphalt Concrete Patch, PCP ; Portland Cement Concrete Patch]

PMIS YEAR	DISTRICT	Highway	Reference Maker		SPL QTY	PCH QTY	ACP QTY	PCC QTY
2010	Beaumont	SL0573 R	426	+ 0.5	65	0	0	13
2010	Houston	IH0045 R	21	+ 0.5	58	0	0	0
2010	Dallas	US0067 R	408	+ 0.5	57	0	0	0
2010	Dallas	IH0020 R	468	+ 0.0	54	0	0	4
2010	Houston	IH0045 L	13	+ 0.5	53	0	0	0
2010	Yoakum	SH0071 R	660	+ 0.0	51	0	0	1
2010	Yoakum	SH0071 R	662	+ 1.0	46	0	0	2
2010	Houston	SL0008 R	696	+ 0.5	44	0	0	2
2010	Houston	SH0288 L	516	+ 1.0	40	0	0	0
2010	Dallas	SL0012 R	596	+ 0.0	40	0	0	0
2010	Houston	IH0045 L	13	+ 0.0	37	0	0	13
2010	Dallas	IH0035EL	419	+ 0.0	34	0	0	1
2010	Beaumont	FM0366 R	0	+ 0.0	34	0	0	4
2010	Houston	IH0045 L	17	+ 0.0	33	0	0	0
2010	Houston	SL0008 L	690	+ 0.0	33	1	0	0
2010	Houston	SH0288 R	508	+ 1.5	32	0	0	0
2010	Houston	SL0008 L	688	+ 0.0	31	0	0	0
2010	Houston	SL0008 R	696	+ 1.0	31	1	0	1
2010	Yoakum	SH0071 R	660	+ 0.5	31	0	0	0
2010	Houston	SH0288 L	476	+ 0.5	30	0	0	0
2010	Houston	IH0045 R	22	+ 0.5	29	3	0	2
2010	Beaumont	FM0366 L	0	+ 0.0	29	0	0	5
2010	Houston	IH0045 L	20	+ 0.0	28	0	0	11
2010	Houston	US0090 R	860	+ 0.7	28	0	0	0
2010	Yoakum	SH0071 R	660	+ 1.2	28	0	0	0

(continued)

PMIS YEAR	DISTRICT	Highway	Reference Maker		SPL QTY	PCH QTY	ACP QTY	PCC QTY
2010	Yoakum	SH0071 R	0634A	+ 1.9	27	0	0	0
2010	Dallas	SL0012 R	630	+ 0.0	27	1	0	10
2010	Dallas	US0067 L	408	+ 0.5	27	0	0	0
2010	Dallas	IH0035EL	420	+ 0.0	26	0	0	7
2010	Dallas	SL0012 R	594	+ 1.5	25	1	0	0
2010	Houston	IH0045 R	17	+ 0.0	24	0	0	2
2010	Houston	SH0288 L	508	+ 0.5	24	0	0	0
2010	Houston	US0090 R	854	+ 1.0	24	0	0	0
2010	Yoakum	SH0071 R	662	+ 0.3	24	0	0	1
2010	Yoakum	SH0071 R	648	+ 1.0	24	1	0	0
2010	Atlanta	SL0151 L	742	+ 0.0	24	0	0	0
2010	Houston	IH0045 L	14	+ 0.0	23	1	0	37
2010	Houston	IH0045 L	17	+ 0.5	23	1	0	0
2010	Houston	SH0288 L	518	+ 1.5	23	0	0	2
2010	Houston	SH0288 L	508	+ 1.5	23	0	0	0
2010	Atlanta	SL0151 L	742	+ 1.0	23	0	0	0
2010	Houston	US0090 R	850	+ 1.6	22	0	0	0
2010	Houston	US0090 R	852	+ 0.5	22	0	0	0
2010	Houston	SL0008 L	696	+ 0.5	21	0	0	0
2010	Houston	SL0008 L	696	+ 0.0	21	0	0	2
2010	Houston	SL0008 L	690	+ 0.5	21	2	0	0
2010	Yoakum	SH0071 R	646	+ 1.5	21	0	0	0
2010	Houston	SH0099 R	690	+ 1.0	20	0	0	0
2010	Houston	SH0225 R	686	+ 0.0	20	4	0	0
2010	Houston	SL0151 R	742	+ 1.5	20	0	0	0
2010	Houston	IH0045 L	19	+ 0.5	19	0	2	6
2010	Houston	SH0288 R	530	+ 0.5	19	0	0	1

(continued)

PMIS YEAR	DISTRICT	Highway	Reference Maker		SPL QTY	PCH QTY	ACP QTY	PCC QTY
2010	Houston	SL0008 R	690	+ 0.5	19	0	0	0
2010	Yoakum	SH0071 R	648	+ 0.0	19	1	0	0
2010	Dallas	US0067 L	408	+ 1.0	19	0	0	0
2010	Dallas	IH0035ER	420	+ 0.0	19	0	0	3
2010	Dallas	IH0035ER	423	+ 0.0	19	2	0	4
2010	Houston	IH0045 R	15	+ 0.5	18	0	0	0
2010	Houston	SH0288 L	516	+ 1.5	18	0	0	0
2010	Houston	SH0288 R	476	+ 0.0	18	0	0	0
2010	Houston	US0090 L	852	+ 0.5	18	0	0	0
2010	Houston	US0090 L	850	+ 0.0	18	1	0	0
2010	Atlanta	SL0151 L	742	+ 0.5	18	0	0	0
2010	Fort Worth	SH0360 R	264	+ 0.5	17	2	0	0
2010	Houston	SH0146 R	482	+ 0.9	17	0	0	0
2010	Yoakum	SH0071 L	638	+ 1.5	17	0	0	0
2010	Dallas	IH0020 R	456	+ 0.1	17	0	0	18
2010	Dallas	SL0012 L	630	+ 1.5	17	0	1	0
2010	Houston	SL0008 L	690	+ 1.0	16	0	0	0
2010	Houston	IH0045 R	39	+ 0.5	16	0	0	0
2010	Yoakum	SH0071 R	638	+ 1.5	16	0	0	0
2010	Yoakum	SH0071 L	636	+ 1.4	16	0	0	0
2010	Dallas	SH0114 R	620	+ 1.1	16	0	0	0
2010	Dallas	IH0020 R	468	+ 0.5	16	0	0	6
2010	Dallas	SL0012 R	628	+ 1.0	16	1	3	8
2010	Dallas	SL0012 L	630	+ 1.0	16	0	0	0
2010	Dallas	IH0035EL	418	+ 0.5	16	0	0	0
2010	Dallas	IH0035ER	419	+ 0.5	16	0	0	3
2010	Houston	US0290 L	732	+ 0.5	15	0	0	3

(continued)

PMIS YEAR	DISTRICT	Highway	Reference Maker		SPL QTY	PCH QTY	ACP QTY	PCC QTY
2010	Houston	IH0045 R	21	+ 0.0	15	17	0	1
2010	Houston	IH0045 L	16	+ 0.5	15	0	0	0
2010	Houston	SH0288 L	500	+ 1.5	15	2	0	6
2010	Houston	SH0288 R	510	+ 1.4	15	0	0	1
2010	Houston	SH0288 L	532	+ 0.0	15	0	0	11
2010	Houston	SH0099 R	688	+ 0.5	15	0	0	0
2010	Houston	FM1093 L	668	+ 1.5	15	0	0	4
2010	Houston	FM1093 L	668	+ 1.0	15	0	0	0
2010	Houston	US0090 L	856	+ 0.0	15	0	0	0
2010	Houston	SH0225 R	696	+ 1.5	15	0	0	0
2010	Yoakum	SH0071 L	638	+ 0.5	15	0	0	0
2010	Dallas	SL0012 R	630	+ 1.0	15	3	0	0
2010	Houston	SL0151 R	742	+ 1.0	15	0	0	0
2010	Atlanta	US0079 L	308	+ 3.3	15	0	0	0
2010	Fort Worth	IH0020 L	424	+ 0.0	14	0	0	0
2010	Houston	SH0036 R	690	+ 0.3	14	0	0	0
2010	Houston	SH0288 L	510	+ 0.5	14	0	0	0
2010	Houston	SH0288 R	508	+ 1.0	14	0	0	0
2010	Houston	FM1876 R	480	+ 1.0	14	3	0	0
2010	Houston	US0290 R	724	+ 1.5	14	0	0	0
2010	Houston	US0290 R	732	+ 1.0	14	6	0	7
2010	Houston	SL0008 L	686	+ 1.7	14	0	0	0
2010	Houston	IH0045 L	49	+ 0.0	14	1	0	0
2010	Houston	FM1093 L	670	+ 0.5	14	2	0	1
2010	Dallas	US0067 R	408	+ 1.0	14	0	0	0
2010	Dallas	US0067 L	408	+ 0.0	14	0	0	0
2010	Houston	FM1093 R	666	+ 0.3	13	0	0	0

(continued)

PMIS YEAR	DISTRICT	Highway	Reference Maker		SPL QTY	PCH QTY	ACP QTY	PCC QTY
2010	Houston	IH0045 L	48	+ 0.5	13	3	0	0
2010	Dallas	SL0012 L	630	+ 0.5	13	0	0	1
2010	Dallas	IH0035ER	419	+ 0.0	13	0	0	0
2010	Fort Worth	SS0303 L	568	+ 0.5	12	0	0	2
2010	Waco	IH0035 R	336	+ 0.3	12	0	0	0
2010	Houston	US0290 L	736	+ 0.0	12	6	0	0
2010	Houston	SH0036 L	690	+ 0.3	12	0	0	0
2010	Houston	SH0288 L	500	+ 1.1	12	0	0	11
2010	Houston	IH0010 R	769	+ 0.5	12	0	0	14
2010	Houston	SL0008 R	690	+ 1.0	12	0	0	0
2010	Houston	US0090 R	850	+ 0.8	12	0	0	0
2010	Houston	US0059 R	520	+ 1.0	12	0	0	4
2010	Houston	SH0146 R	482	+ 1.4	12	0	0	0
2010	Dallas	IH0635 L	15	+ 0.7	12	0	0	3
2010	Dallas	SL0012 R	628	+ 1.5	12	2	0	1
2010	Houston	US0290 L	724	+ 1.0	11	0	0	3
2010	Houston	SH0288 L	516	+ 0.5	11	0	0	0
2010	Houston	SH0288 L	502	+ 1.0	11	0	0	6
2010	Houston	SH0288 R	518	+ 1.0	11	0	0	2
2010	Houston	SH0288 R	520	+ 0.0	11	0	0	1
2010	Houston	SH0006 R	686	+ 0.0	11	0	0	1
2010	Houston	SL0008 R	0	+ 0.0	11	1	0	0
2010	Houston	US0090 R	860	+ 0.2	11	1	0	0
2010	Houston	US0090 R	862	+ 1.0	11	0	0	26
2010	Houston	US0090 R	854	+ 1.5	11	0	0	0
2010	Houston	IH0610 R	15	+ 0.0	11	0	0	50
2010	Houston	US0059 R	526	+ 0.0	11	0	0	0

(continued)

PMIS YEAR	DISTRICT	Highway	Reference Maker		SPL QTY	PCH QTY	ACP QTY	PCC QTY
2010	Houston	US0059 L	520	+ 0.5	11	0	0	0
2010	Houston	SH0225 R	696	+ 1.0	11	0	0	0
2010	Yoakum	SH0071 R	660	+ 0.8	11	0	0	2
2010	Yoakum	SH0071 R	662	+ 1.5	11	0	0	0
2010	Dallas	US0067 R	408	+ 1.5	11	0	0	0
2010	Fort Worth	IH0820 R	0	+ 0.5	10	0	0	0
2010	Fort Worth	SH0360 R	272	+ 1.0	10	0	0	0
2010	Houston	US0290 R	0702A	+ 1.5	10	0	0	0
2010	Houston	US0290 L	726	+ 0.5	10	0	0	0
2010	Houston	IH0045 L	11	+ 0.5	10	0	0	12
2010	Houston	IH0045 R	14	+ 0.5	10	0	0	3
2010	Houston	IH0045 R	17	+ 0.5	10	0	0	0
2010	Houston	SH0288 L	508	+ 1.0	10	0	0	0
2010	Houston	SH0288 R	518	+ 1.5	10	0	0	1
2010	Houston	SH0288 L	472	+ 0.0	10	0	0	13
2010	Houston	SH0099 L	700	+ 0.5	10	1	0	0
2010	Houston	IH0010 L	769	+ 0.4	10	3	0	10
2010	Houston	SL0008 L	690	+ 1.5	10	1	0	0
2010	Houston	SL0008 R	690	+ 0.0	10	0	0	0
2010	Houston	FM1093 L	668	+ 0.5	10	0	0	0
2010	Houston	FM1093 R	668	+ 0.0	10	1	0	0
2010	Houston	US0090 L	848	+ 0.6	10	0	0	0
2010	Houston	SH0225 R	694	+ 0.5	10	0	0	0
2010	Houston	SH0146 R	484	+ 0.5	10	2	0	0
2010	Dallas	SL0012 R	630	+ 0.5	10	0	0	1
2010	Dallas	SL0012 R	630	+ 1.5	10	0	0	2
2010	Dallas	IH0035EL	423	+ 0.0	10	0	0	5
2010	Houston	SL0151 R	742	+ 0.0	10	0	0	0
2010	Houston	SL0151 R	742	+ 0.5	10	0	0	0
2010	Atlanta	US0079 R	308	+ 3.3	10	0	0	0

Table 2.2 CRCP Sections for Concrete Core Sampling

District	Highway	Visual Survey Result in Field
HOUSTON	US 90	Spalling
ATLANTA	US 59	Spalling and Delamination
HOUSTON	FM 523 #2	Spalling
HOUSTON	SH 99	Spalling
HOUSTON	FM 523 #1	Spalling
HOUSTON	FM 1301	Spalling
ATLANTA	US 79BR	Spalling
ATLANTA	SL 151	Spalling and Delamination
BEAUMONT	FM 366	Spalling
HOUSTON	US 59	Spalling
BEAUMONT	SL 573	Spalling
HOUSTON	BW 8	Spalling
HOUSTON	US 290	Spalling
PARIS	IH 30	Delamination
HOUSTON	SH 6	Spalling
YOAKUM	SH 71	Spalling
AMARILLO	IH 40	Delamination
DALLAS	SH 121	No distress
DALLAS	IH 45	Delamination
FORT WORTH	IH 20	Delamination
PARIS	US 75	Delamination
LAREDO	IH 35	Delamination
DALLAS	IH 35	No distress
DALLAS	SH 161	No distress

2.2.2 Field Survey and Concrete Core Sampling

Figures 2.1 and 2.2 show the pavement condition of SH 99 and US 59 in Fort Bend County, Houston District. Spalling repairs were observed at almost every crack on SH 99. Figure 2.2 shows concrete patches.

Figure 2.1 Houston, SH 99

Figure 2.2 Houston, US 59

Figure 2.3 shows the pavement condition of FM 523 in Brazoria County, Houston District. It is interesting to observe a large number of spalling repairs done on the outside shoulder. Table 2.3 shows the distress and cumulative traffic information on FM 523 from the 2010 PMIS. Cumulative traffic is at a low level; however, a large number of patches were observed in October, 2011, more than the numbers recorded in the 2010 PMIS. It appears that most of the patches were made after the 2010 PMIS survey. To address deteriorated ride quality due to severe spalling and repairs, the Houston District plans to place hot mix asphalt overlay on this section.

Figure 2.3 Houston, FM 523

Table 2.3 Traffic History and Distresses on FM 523 in Houston District

Highway	Reference Marker		ESALs from 2001 to 2010 [million]							Number of Distresses (2010 PMIS)			
	Beg.	End.	2001	2002	2003	…	2009	2010	∑ ESALs	SPL	PCH	ACP	PCP
FM 523 L	516+0.0	516+0.4	0.10	0.08	0.08	…	0.08	0.09	1.3	1	1	0	9
FM 523 R	516+0.0	516+0.4	0.10	0.08	0.08	…	0.08	0.09	1.3	1	1	0	3

Figure 2.4 shows the condition of US 90 in Harris County, Houston District. A large number of concrete patches are observed. The primary distress on this highway was delamination of concrete at about 2 in depth, and resulting severe spalling at transverse cracks.

Figure 2.5 shows numerous patches to repair severe spalling on US 290 in Harris County, Houston District. The ride quality of this roadway has deteriorated substantially due to severe spalling and short longevity of repair materials.

Figures 2.6 and 2.7 show the pavement condition of SL 573 in Liberty County and FM 366 in Jefferson County in the Beaumont District, respectively. Even though spalling was observed in both highways, its condition was rather mild compared with those observed in the Houston District or Atlanta District.

Figure 2.4 Houston, US 90

Figure 2.5 Houston, US 290

Figure 2.6 Beaumont, SL 573

Figure 2.7 Beaumont, FM 366

Figure 2.8 Atlanta, US 59

Figure 2.8 shows a number of surface patches on US 59 in Cass County, Atlanta District. The picture on the right indicates shallow delaminations, with calcium hydroxide leaching out. Figure 2.9 shows the spalling and faulting on SL 151 in Bowie County, Atlanta District. This section was built in 2004 and is relatively new. Figure 2.9-(b) shows a faulting at a transverse crack. This type of faulting is unexpected in CRCP. However, this type of faulting was observed during the repair of distress on IH 45 in the Dallas District. The higher elevation at one side of a transverse crack is due to horizontal cracking at the depth of longitudinal steel at that side. Subsequent warping and curling resulted in the "lift" of the concrete surface at the transverse crack. With continued truck traffic applications, a new transverse crack will develop at the end of the delaminated area. Figure 2.9-(c) shows transverse cracks with narrow spacing. However, close examinations of the cracks reveal that there are actually two different types of transverse cracks – wide (2^{nd} and 4^{th} cracks from left) and tight cracks.

(a) Typical spalling on SL 151

(b) CRCP faulting

(c) Narrow cracks

(d) Horizontal crack

Figure 2.9 Atlanta, SL 151

The two wide cracks are normal transverse cracks observed in CRCP, developed by stresses due to temperature and moisture variations. On the other hand, the other two tight cracks occurred after delaminations took place. A core was taken in this area and delamination was observed at the depth of longitudinal steel as shown in Figure 2.9-(d).

Figure 2.10 shows the spalling and concrete cores on US 79 B road in the Atlanta District. Figure 2.11 shows the core sample taken from SH 6 in Harris County in the Houston District. In this section, only one core was obtained. This section of SH 6 between US 290 and IH 10 was built in 1989. The coarse aggregate used was SRG. Severe spalling occurred within a few years and the ride condition deteriorated over the years. In the middle of the 2000s, the Houston District placed hot mix asphalt overlay to address ride quality issues.

Figure 2.10 Atlanta, US 79 BR

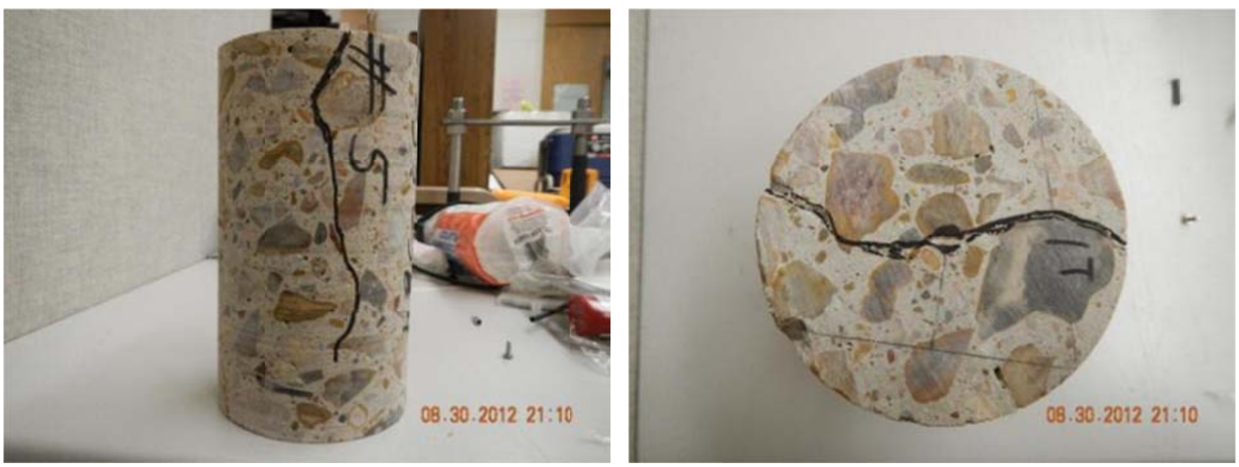


Figure 2.11 Houston, SH 6

Figure 2.12 shows the CRCP condition on IH 30 in Hopkins County in the Paris District. This pavement was constructed by the tube feeding method with two-lift construction. As shown in Figure 2.12 (b), transverse steel is above longitudinal steel and the longitudinal steel spacing is not uniform. Severe delaminations at the depth of longitudinal steel are observed. There are numerous concrete patches in this highway. The cause of the distresses was horizontal cracking at the depth of longitudinal steel. The exact cause of horizontal cracking is not known; however, it could be related to the two-lift paving method.

(a) Pavement Condition **(b) Transverse Steel above Longitudinal Steel**

(c) Delamination **(d) Delamination and distress**

Figure 2.12 Paris, IH 30

Figure 2.13 shows the condition of SH 71 in Fayette County in the Yoakum District. This section was built in the late 80s and early 90s, and this is the project where the first ride specifications were applied. The spalling observed in this section was mild. Figure 2.14 shows the repair of delamination on IH 45 in Navarro County in the Dallas District.

Figure 2.13 Yoakum, SH 71

Figure 2.14 Dallas, IH 45

Figure 2.15 shows the severe spalling due to delamination on IH 20 in Parker County in the Fort Worth District. The steel was placed by the tube feeding method, and the two-lift method was used for the concrete placement. It is not known whether two-lift concrete placement method caused delaminations at the interface between two lifts.

Figure 2.16 shows the condition of IH 35 in La Salle County in the Laredo District. This section of 9-in CRCP was built in the early 2000s as a test section to investigate the viability of concrete overlay on existing asphalt pavement. Three transverse cracks are close to each other, and concrete was fractured into smaller pieces. Traffic data from a WIM (weight-in-motion) station installed in this section revealed the applications of heavy trucks, whose weights were far greater than legal limits. Based on the analysis conducted in TxDOT research study 0-5832, it is concluded that over-weight trucks caused these delaminations and distresses.

Figure 2.15 Fort Worth, IH 20

Figure 2.16 Laredo, IH 35

Figure 2.17 shows full-depth CRCP repairs on IH 40 in Gray County in the Amarillo District. Delaminations at the depth of longitudinal steel and resulting distresses are observed. Asphalt base deteriorated apparently due to moisture infiltration.

(a) Loose Base Condition (b) Delamination

(c) Partial Depth Distress (d) Concrete Cores

Figure 2.17 Amarillo, IH 40

2.2.3 Concrete CoTE and Modulus of Elasticity Tests

In general, it is quite rare to observe spalling distress in CRCP with low CoTE and modulus of concrete, whereas severe spalling is often observed in CRCP with high CoTE and modulus of concrete. A 15-in CRCP section on IH 45 in Montgomery County in the Houston District, built in 1990, is a good example. This section, the inside two northbound lanes just north of Spring Creek, was built as a test section to investigate the effect of coarse aggregate and longitudinal steel amount. The total length of the section was 2,070 ft (920 ft for each aggregate type plus 230 ft transition). Two coarse aggregate types were used – siliceous river gravel (SRG) and crushed limestone (LS). After more than 20 years of service under heavy traffic, there was no single structural distress or punchout. However, there was a huge difference in the spalling performance. There was no single spalling in LS section, while there were numerous repairs done for spalling in the SRG section. Figure 2.18 shows the surface condition in both sections. The pavement in this highway north of Spring Creek was built with LS coarse aggregate, except for a 920 ft test section where SRG was used. It is interesting to note that the surface condition of the outside two lanes where LS was used is quite good, with no spalling even though truck traffic was higher. Detailed evaluations were made in concrete properties with various coarse aggregates in Texas by Green et al in 1987. There was no practical difference in the properties of concrete made with SRG and LS, except for CoTE and modulus of elasticity. There were other studies that found the same information. There might be other concrete properties responsible for the vast difference in spalling performance of CRCP sections built with SRG and LS coarse aggregate. However, extensive research effort since the 1980s has not identified those except for CoTE and modulus of elasticity. In this research study, CoTE and modulus of elasticity were evaluated on the cores obtained from the field.

(a) SRG section **(b) LS section**

Figure 2.18 IH 45 test section in Houston

2.2.3.1 Coefficient of Thermal Expansion (CoTE) Test

Volume changes induced during CoTE testing were quite small, about 3 mils for full temperature changes between 10 °C and 50 °C . It takes about 2.5 hours for temperature variation from 10 °C to 50 °C. Accordingly, the rate of concrete length change of a 7 in specimen is about 0.02 mils per minute, or 0.00002 in per minute. This is an extremely small change rate, and to obtain accurate CoTE values, it is important that a displacement gage has a high accuracy as well as a good stability. Extensive evaluations at TxDOT CSTMP showed that LVDT (linear variable differential transformer) does not provide accurate results. CSTMP staff identified DVRT (differential variable reluctance transducer) as a more accurate and stable displacement gage. A CoTE testing setup was established at Texas Tech University in consultation with TxDOT CSTMP staff, which is shown in 2.19.

Figure 2.19 Schematic for CoTE Test

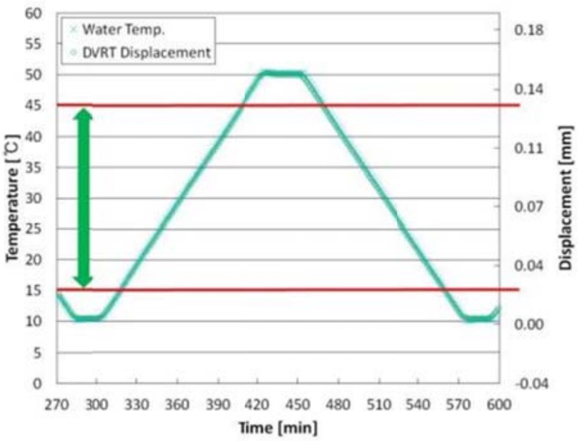
Cores obtained from the CRCP sections with severe spalling distress were cut to a standard length of 7 ± 0.1 in according to the TxDOT Test Procedure for CoTE, Tex-428-A. Before testing, all the specimens were submerged in saturated limewater at 73 ± 4 °F for more than 72 hours. Before placing a specimen in a CoTE frame, specimen length was measured and recorded to the nearest 0.004 in within five minutes after removal from the saturation tank.

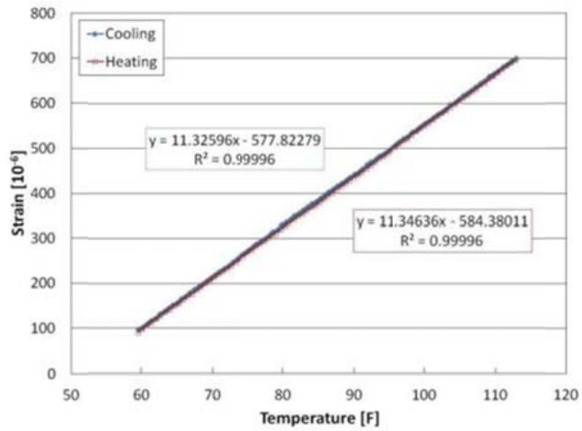
The procedures that followed for CoTE evaluations were in accordance with Tex-428-A, and the steps followed were:

1. Leave frame and water specimen in water bath at room temperature until the water and concrete temperatures become the same at room temperature.
2. Adjust DVRT to a null position.
3. Set the temperature to 10 °C (50 °F).
4. Leave at 10 °C (50 °F) for 30 minutes.
5. Set the temperature at 50 °C (72 °F), with the time from 10 °C (50 °F) to 50 °C (72 °F) taking 2.5 hours.
6. Keep at 50 °C (72 °F) for 30 minutes.
7. Set the temperature at 10 °C (50 °F), with the time from 50 °C (72 °F) to 10 °C (50 °F) taking 2.5 hours.
8. Keep at 10 °C (50 °F) for 30 minutes.
9. Repeat Steps 5 thru 8 two more times, to total three cycles.
10. Compute CoTE based on the last cycle.

Temperature and displacement were measured and recorded every one minute for the entire three cycles. Only increasing or decreasing temperature points between 15 °C to 45 °C (59 °F to 113 °F) were used for regression analysis.

Correction factor was determined in accordance with Tex-428-A with the reference cylinder provided by TxDOT CSTMP, whose CoTE value was known to be 8.48 με/°F. The correction factor was estimated at -2.86 με per °F. Figure 2.20 shows the temperature range for CoTE calculation and CoTE value for the reference cylinder.





(a) Temperature Range for CoTE Calculation (b) CoTE value of Calibration Cylinder
Figure 2.20 CoTE Test for Reference Cylinder

2.2.3.2 Modulus of Elasticity Test

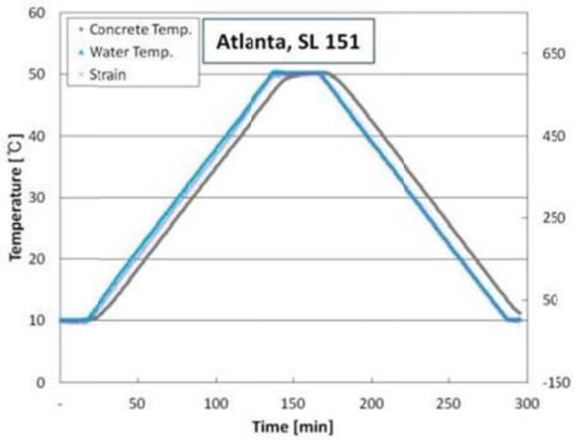
Modulus of elasticity of concrete cores was estimated by the free-free resonance method in accordance with ASTM C 215-08, "Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens." The concrete specimen was placed on the rubber pad so that it could vibrate freely in the longitudinal mode. An accelerometer was firmly attached at the center of one end surface of the concrete specimen. The accelerometer was attached to the main body acquisition and display unit. The other end of the concrete specimen was hit by the ball-peen hammer, while making sure to provide a perpendicular impact on the surface of the concrete specimen as shown in Figure 2.21. This process was repeated until getting a stable resonant frequency value. With the measured resonant frequency, and mass and dimensions of the concrete specimen, modulus of elasticity was estimated.

Figure 2.21 Resonant Frequency Test

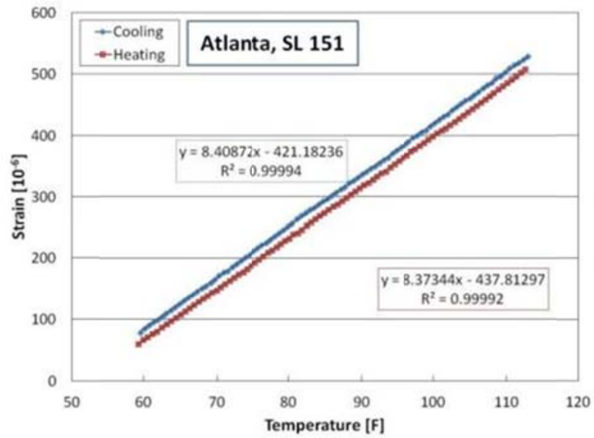
2.2.4 CoTE and Dynamic Young's Modulus of Elasticity Test Results

Figure 2.22 shows the CoTE test data of concrete cores from SL 151 in the Atlanta District and US 90 in the Houston District.

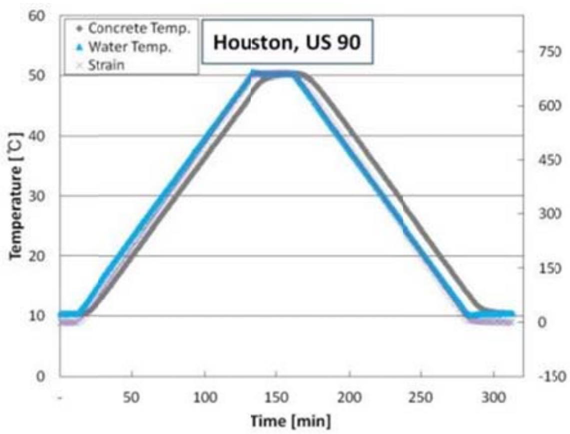
Table 2.4 summarizes the test results for CoTE and modulus of elasticity from all the cores obtained in this project. The columns were arranged in terms of CoTE values in descending order. It is noted that there is an excellent match between CoTE and spalling performance. The only exception is the CoTE value at SH 6 in the Houston District. It is recalled that there was a minor crack in the concrete core, and it appears that the crack reduced the CoTE value. Communication of a coarse aggregate producer who supplied the coarse aggregate to the US 290 project stated that they also provided coarse aggregate to the SH 6 project. Accordingly, the CoTE values for both US 290 and SH 6 projects should be quite close. Table 2.4 also shows a decent correlation between CoTE and modulus values. According to Table 2.4, concrete in the sections with spalling distress had CoTE values larger than 5.5 per °F. On the other hand, concrete in the sections with delaminations only had CoTE values lower than 5.5 per °F. This implies that delaminations observed in this investigation were due to something other than high CoTE values. It is hypothesized that two-lift concrete paving, the application of over-weight vehicles, or deteriorated base was responsible for the delaminations.



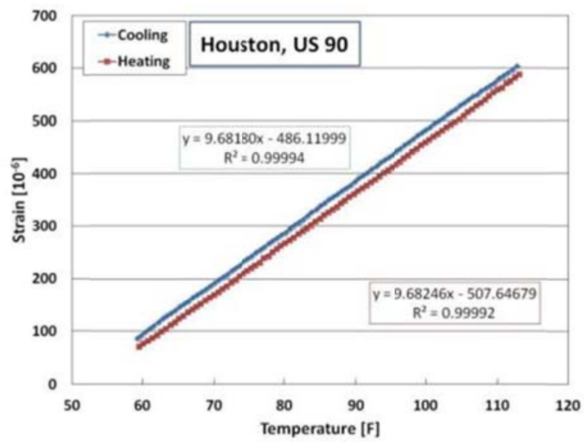
(a) Heating and Cooling (ATL, SL 151)



(b) CoTE value before Correction (ATL, SL 151)



(c) Heating and Cooling (HOU, US 90)



(d) CoTE value before Correction (HOU, US 90)

Figure 2.22 Example CoTE Test Data

Table 2.4 CoTE and Modulus Test Results

District	Highway	Visual Survey Result	CoTE [microstrain/°F]	Modulus of Elasticity [psi]
HOUSTON	US 90	Spalling	6.27	6,989,850
ATLANTA	US 59	Spalling and Delamination	6.26	6,625,290
HOUSTON	FM 523 #2	Spalling	6.02	6,645,870
HOUSTON	SH 99	Spalling	5.91	5,821,200
HOUSTON	FM 523 #1	Spalling	5.87	6,818,333
HOUSTON	FM 1301	Spalling	5.86	6,119,610
ATLANTA	US 79BR	Spalling	5.78	5,621,280
ATLANTA	SL 151	Spalling and Delamination	5.70	6,075,510
BEAUMONT	FM 366	Spalling	5.68	6,346,608
HOUSTON	US 59	Spalling	5.67	6,165,180
BEAUMONT	SL 573	Spalling	5.66	7,234,780
HOUSTON	BW 8	Spalling	5.58	5,656,560
HOUSTON	**US 290**	**Spalling**	**5.57**	**5,749,170**
PARIS	**IH 30**	**Delamination**	**5.38**	**5,070,030**
HOUSTON	*SH 6*	Spalling	*5.31*	*4,924,500*
YOAKUM	SH 71	Minor Spalling	5.21	5,325,810
AMARILLO	IH 40	Delamination	4.83	5,793,270
DALLAS	SH 121	No distress	4.22	6,816,390
DALLAS	IH 45	Delamination	4.13	6,581,190
FORT WORTH	IH 20	Delamination	3.94	5,233,200
PARIS	US 75	Delamination	3.94	5,787,390
LAREDO	IH 35	Delamination	3.83	6,350,400
DALLAS	IH 35	No distress	3.33	4,971,540
DALLAS	SH 161	No distress	2.75	5,303,760

Figure 2.23 shows the map with CRCP projects where cores were taken. The blue dots indicate locations with CoTE values of smaller than 5.4 per °F, while red dots indicate those with CoTE values greater than 5.4 per °F. There is a good agreement between the locations of CRCP sections with spalling problems and the locations of red dots. Based on this excellent correlation, it is considered that coarse aggregates that produce concrete with CoTE greater than 5.4 per °F is not a good material for CRCP; rather, those materials should be used for CPCD.

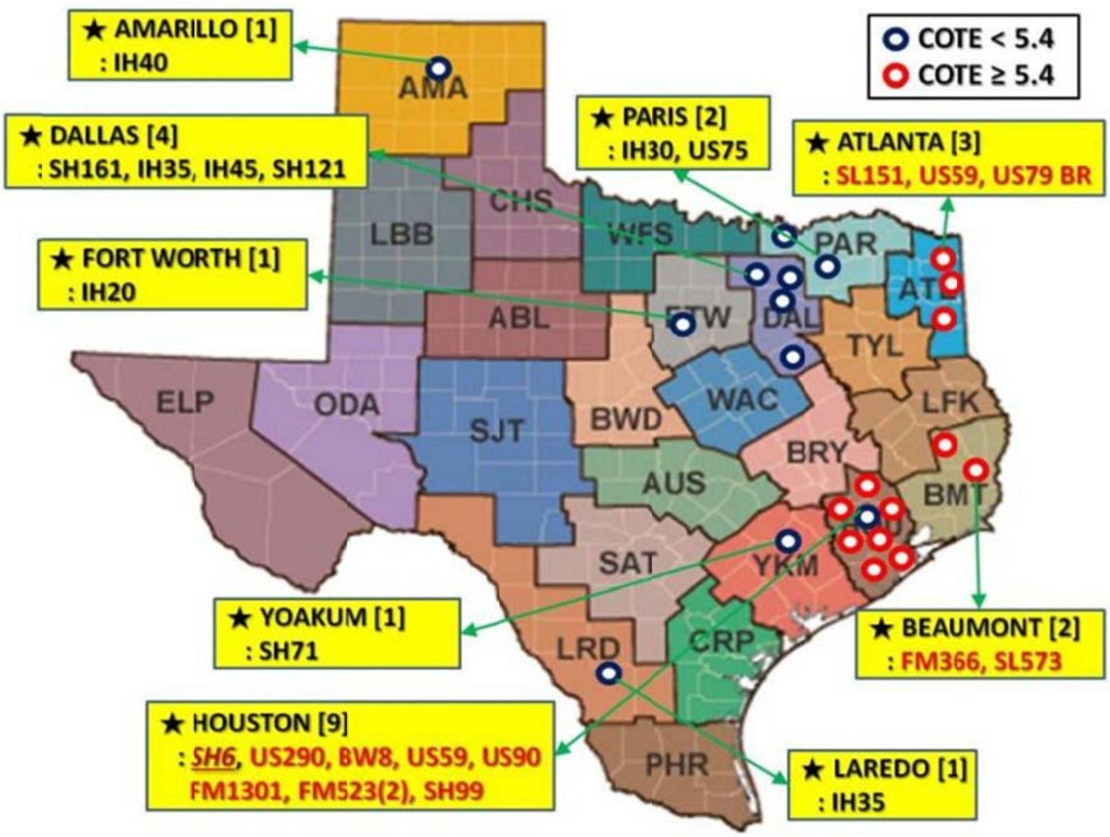


Figure 2.23 Map for Concrete Core Sampling

Chapter 3: Characteristics of Local Coarse Aggregates

3.1 Material Properties of Local Coarse Aggregates

Coarse aggregate occupies typically about 40 to 50% concrete volume. Consequently, physical properties of concrete are highly affected by the properties of constituent coarse aggregates. TxDOT has about 16,400 lane miles of concrete pavement. About 75% of the concrete pavement is CRCP. As discussed in the previous chapter, significant distresses have been observed in CRCP when certain coarse aggregates were used. Current research identifies CoTE incompatibilities between the mortar matrix and the coarse aggregate as one of the prime reasons for distress development (Choi et al. 2011). Some districts, such as Houston, do not have good quality local aggregate sources. Those districts have to haul aggregates over greater distances for pavement construction. This increases the material cost, as well as the overall project cost. Some of these less compatible aggregates, however, have been used in concrete pavement, and concrete pavements with contraction design (CPCD) (generally known as jointed concrete pavement (JCP)) using these aggregates showed much less distress when compared to CRCP using them. So, it would seem economically important to select a concrete pavement type based on the quality of available local coarse aggregates.

Five districts were selected (Atlanta, Houston, Amarillo, Paris, and Wichita Falls), based on the volume of concrete pavement in each district and on the scarcity of good quality local aggregate sources. This report documents critical physical properties of the selected coarse aggregates from each of the five districts, based on the results of project-specified laboratory tests.
Good quality aggregate is being depleted because of the high volume of concrete use. This is forcing the issue of using lower quality aggregates in concrete. In concrete pavement the use of incompatible aggregate usually causes premature deterioration of that pavement. The educated selection of pavement type, based on critical physical properties of aggregate, can reduce aggregate-related distress. So, the objective of the task by UT-Austin was to:

- Identify sources of coarse aggregate in or near the TxDOT districts of Atlanta, Houston, Amarillo, Paris, and Wichita Falls that are incompatible for CRCP but could be good candidates for CPCD.
- Determine critical physical properties of these aggregates and concrete made from them, based on the laboratory testing.

3.2 Aggregate Selection

The District Offices of Atlanta, Houston, Amarillo, Paris, and Wichita Falls were contacted and queried for aggregate selection. Two sources of aggregate for each district were selected. Table 3.1 shows the selected aggregate sources. Texas and Oklahoma have four sources each, and the other two sources are from Arkansas. Four of the coarse aggregates are siliceous gravel, two are a natural blend of siliceous and limestone gravel, and one source each of granite, sand stone, rhyolite, and dolomite. The mineralogies of the aggregates were obtained from the TxDOT Concrete Rated Source Quality Catalog (CRSQC) data sheet. H-OK and I-OK were not on the list and were identified by petrographic analysis performed by TxDOT personnel.

Table 3.1 List of Selected Coarse Aggregate Sources

District	***Producer***	***Material Type****
Houston	A-TX	PCSG
Houston	B-TX	PCSG
Atlanta	C-AR	PCSG
Atlanta	D-AR	PCSG
Amarillo	E-TX	PCSLG
Amarillo	F-TX	PCSLG
Wichita Falls	G-OK	CG
Wichita Falls	H-OK	CR
Paris	I-OK	SS
Paris	J-OK	CD

*PCSG= partly crushed siliceous gravel, PCSLG= partly crushed siliceous and limestone gravel, CG= crushed granite, SS= sand stone, CR= crushed rhyolite, and CD= crushed dolomite

3.3 Aggregate Testing

The following aggregate tests were performed in this task.

- Los Angeles (L.A.) abrasion
- Sulfate soundness
- Absorption and specific gravity
- Micro-Deval (MD)
- Aggregate Imaging Measurement System (AIMS)
- Unconfined freeze-thaw, and
- Aggregate crushing value (ACV).

L.A. abrasion and sulfate soundness tests were performed to see whether each aggregate meets the Item 421 of the TxDOT Book of Standard Specifications (2004). L.A. abrasion was done according to TxDOT Test Procedure Tex-410-A (1999). Grade B aggregate gradation was used with 11 metal balls. Figure 3.1 shows the L.A. abrasion loss of the aggregates. Item 421 limits the L.A. abrasion loss to a maximum of 40%, and all the aggregate sources satisfy this requirement. F-TX showed the highest loss of 29%, and H-OK showed the lowest loss of 11%.

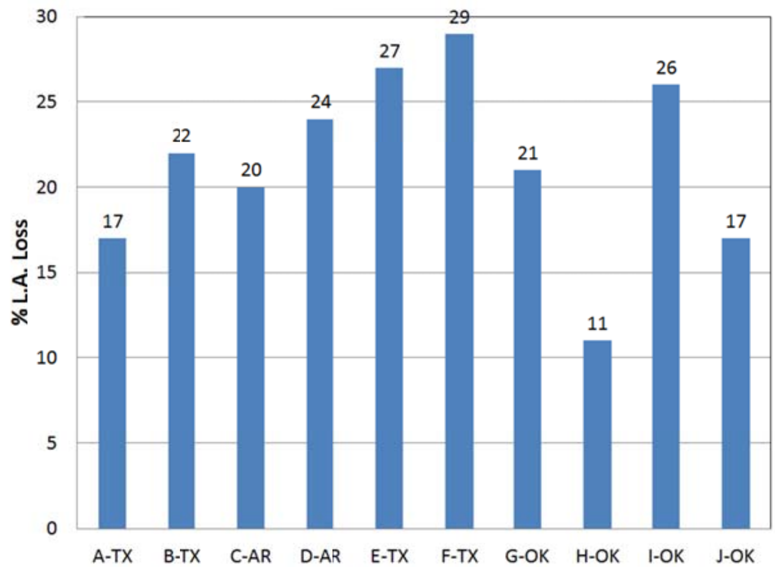


Figure 3.1 L.A. Abrasion Loss for Aggregates

Five-cycle sulfate soundness was performed according to TxDOT Test Method Tex-411-A (2004). Figure 3.2 shows sulfate soundness test loss values for all the aggregates. According to Item 421 the allowable limit for five-cycle sulfate soundness loss is a maximum of 18%. All the sources qualify under this limit. E-TX showed the highest loss of 12%, whereas B-TX, G-OK, H-OK and J-OK showed the lowest loss of 1%.

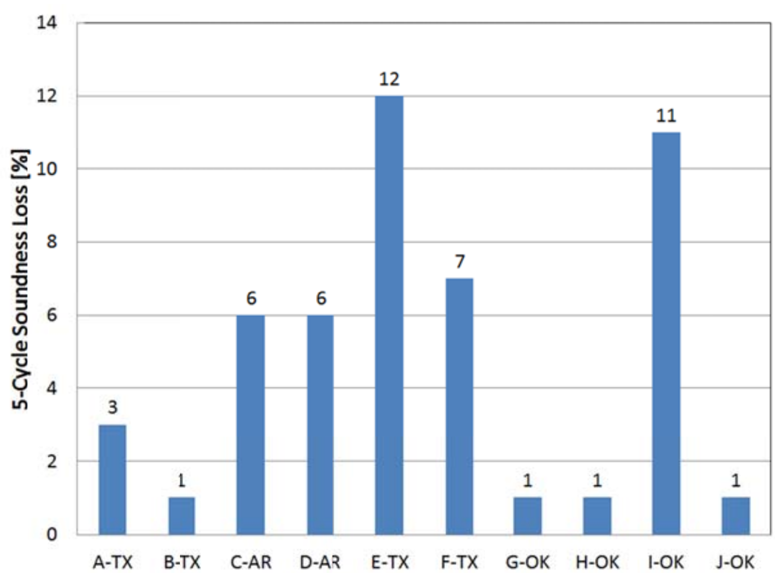


Figure 3.2 Sulfate Soundness of Aggregates

Specific gravity (SG) and absorption of aggregates were determined according to Tex-403-A (1999). Obtained results are shown in Figure 3.3 Specific gravity of the aggregates varied from 2.47 to 2.65. I-OK had the lowest SG and highest absorption, because it is sand stone and has a very porous structure. Despite that, absorption varied from 0.7% to 1.2%.

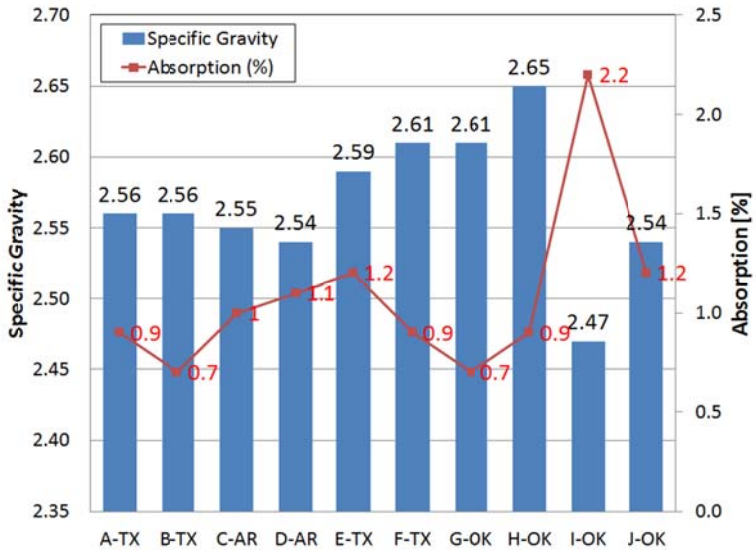


Figure 3.3 Specific Gravity and Absorption of Aggregates

Micro-Deval (MD) was performed according to Tex-461-A (2005). Figure 3.4 shows Micro-Deval losses of selected aggregates. E-TX showed the highest MD loss of 16%, and A-TX showed the lowest loss of 2%. Partly crushed siliceous and limestone gravel showed the maximum loss, probably because of the presence of softer limestone. Sandstone also showed higher loss, but siliceous gravel and igneous rocks showed relatively lower MD loss, due to higher hardness. Research done by the Ministry of Transportation of Ontario (MTO) in Canada established that Micro-Deval is a very good indicator of field performance. MTO adopted an MD loss of up to 13% for concrete pavement. According to Chris Rogers (1998) aggregate can perform well with up to 17% MD loss.

Figure 3.5 shows the relationship between L.A. abrasion and MD loss in these aggregates. A linear relationship is observed with a positive slope, i.e. aggregate with high L.A. loss generally shows high MD loss.

Figure 3.6 shows the relationship between absorption versus L.A. abrasion and MD loss. If the two outliers (as shown in circle) are not considered, a positive correlation is present. Aggregates with high absorption tend to show high L.A. abrasion and MD loss. Because absorption is an indicator of porosity, higher absorption means higher porosity and lower aggregate strength (softer material). Softer materials show higher L.A. and MD losses.

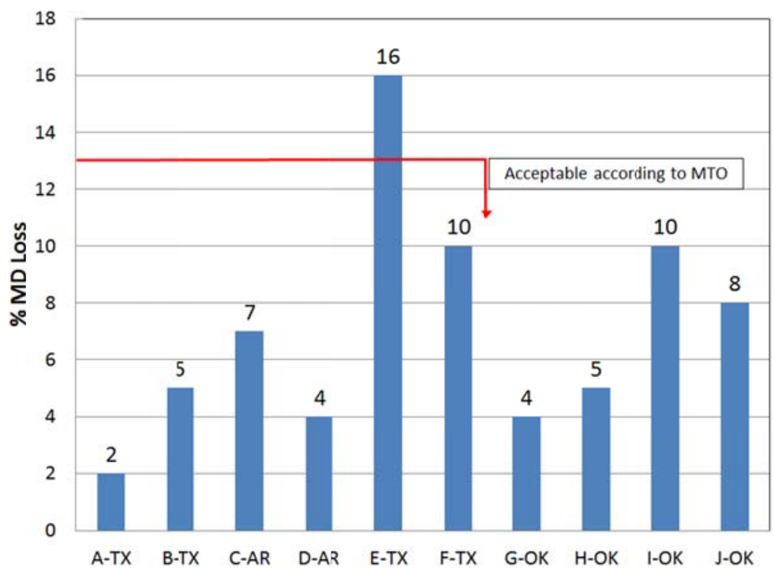


Figure 3.4 Micro-Deval Loss for Aggregates

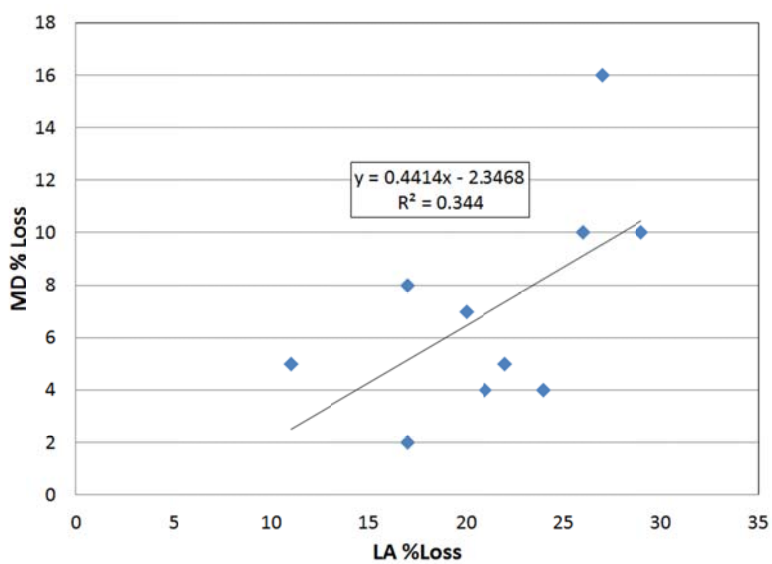


Figure 3.5 Comparison Between Micro-Deval and L.A. Abrasion

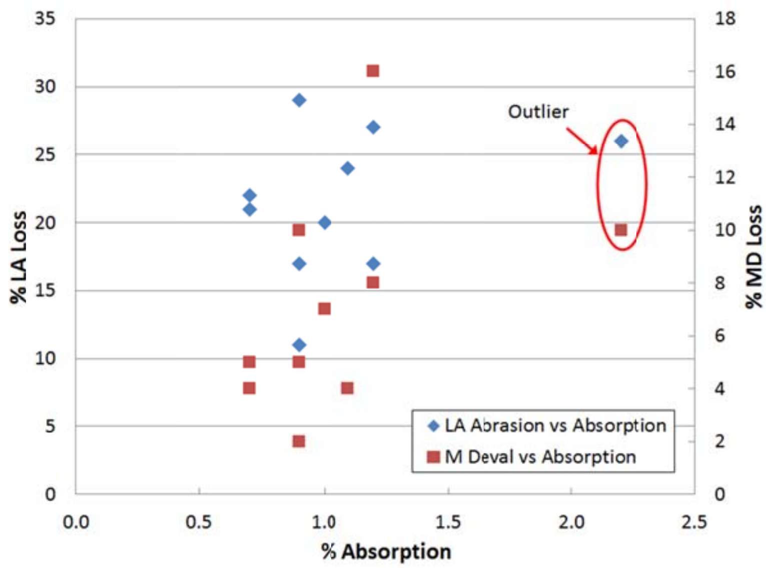


Figure 3.6 Relation Between Absorption vs. %LA and %MD Losses

Angularity and texture of aggregate was measured before Micro-Deval (BMD) and after Micro-Deval (AMD) by the Aggregate Image Measurement System (AIMS). Aggregates retained on the ½-in., 3/8-in., 1/4–in. and #4 sieves were used in AIMS. Average angularity and texture presented in this report are the average values for different particle sizes. Note that B-TX and I-OK did not have all the needed aggregate sizes and were crushed to get the missing particle sizes. It should be noted that angularity, texture values and shapes are all affected by the crushing process in both of these aggregates.

Figure 3.7 shows average angularity before and after Micro-Deval. Aggregates can be divided into four groups based on their angularity: Low (≤ 2100), Moderate (2100 - 3975), High (3975 - 5400) and Extreme (5400 - 10000). G-OK, H-OK and J-OK are in the moderate angularity range BMD and AMD. C-AR is in the moderate range BMD and low range AMD. The remaining aggregates are in the low angularity range for both BMD and AMD. Note that all the river gravels are in low angularity range, as expected.

Figure 3.8 represents the percent change in angularity BMD and AMD. F-TX showed the highest change of 23.5%, and B-TX showed the lowest of 4.3%. This change is consistent with the MD loss, which is shown in Figure 3.9.

Figure 3.10 represents the average texture value BMD and AMD. Aggregates are classified into four groups based on the texture: Low (≤ 200), Moderate (200 - 500), High (500 - 750) and Extreme (750 - 1000). F-TX, G-OK, H-OK, and J-OK are in the moderate range BMD and AMD. The remaining aggregates are in the low texture range. Although B-TX and I-OK were crushed, that did not significantly improve the texture of these aggregates. This is probably because the texture of the failure plane is governed by the mineralogy of the aggregates. Figure 3.11 shows the percent change in texture BMD and AMD. AMD texture was supposed to decrease, but three aggregates indicated a significant increase in texture AMD. This may be due to an inability of AIMS to actually measure texture rather than contrast, but such determinations are outside the scope of this research project.

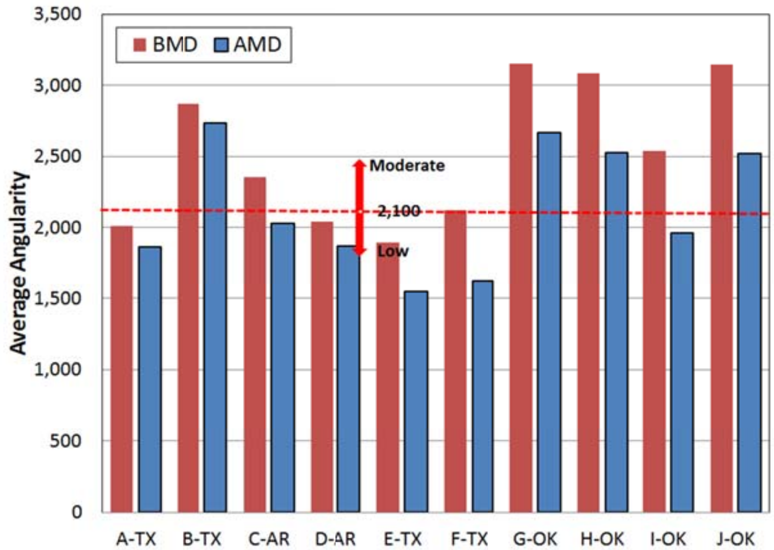


Figure 3.7 Average Angularity Before and After Micro-Deval

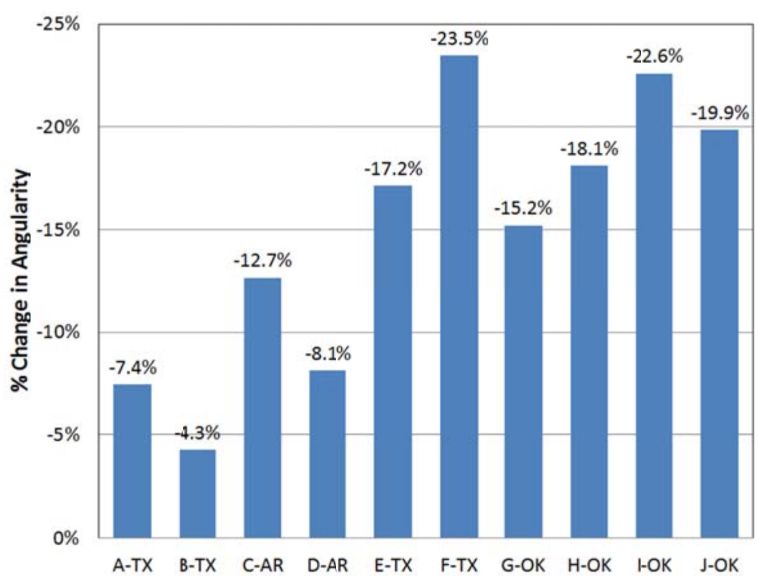


Figure 3.8 Percent Change in Texture Before and After Micro-Deval

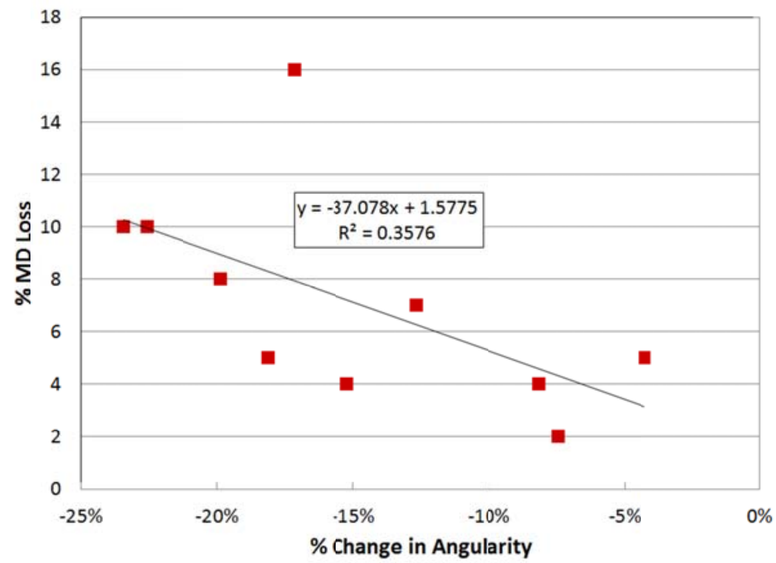


Figure 3.9 Comparison Between % Change in Angularity and % Micro-Deval Loss

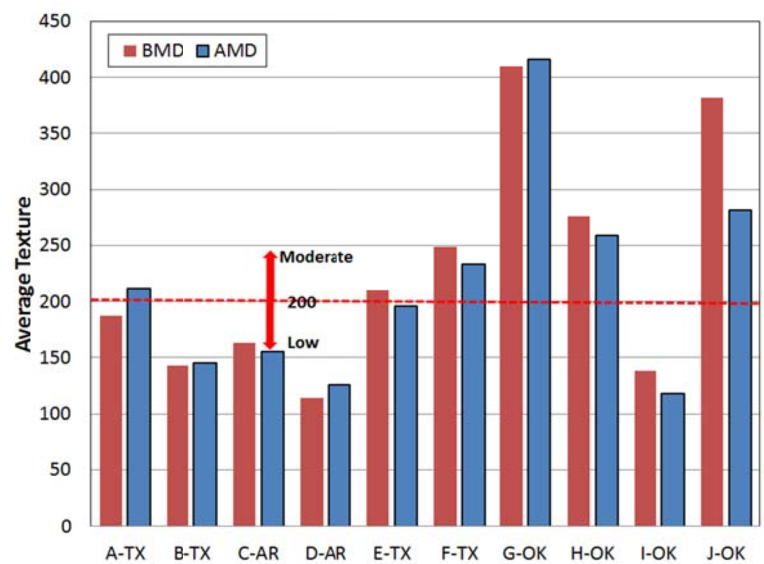


Figure 3.10 Average Texture Before and After Micro-Deval

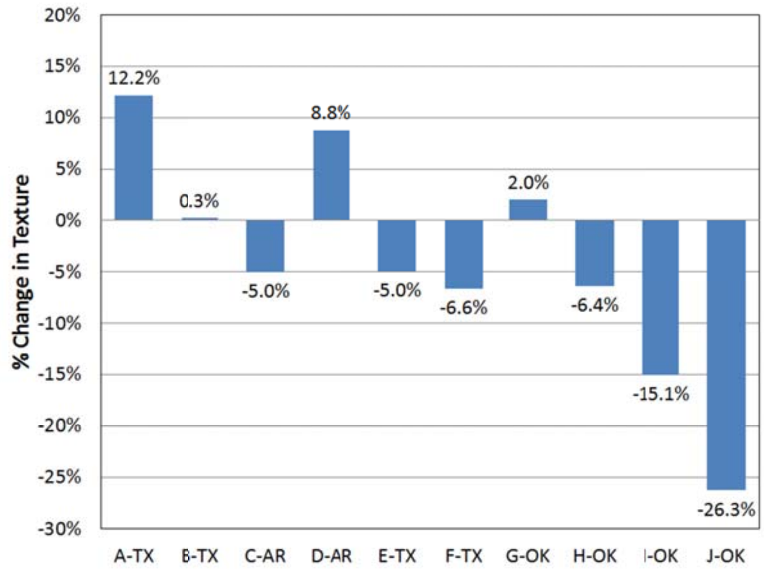


Figure 3.11 Change in Texture Before and After Micro-Deval

Figure 3.12 shows the particle shape distribution of the specified aggregates. The longer-to-shorter-dimension ratio (L/S) of 2:1 was selected, because a particle is considered "flaky" when L/S>1.66. All the aggregates show at least 40% of their particles having L/S>2.

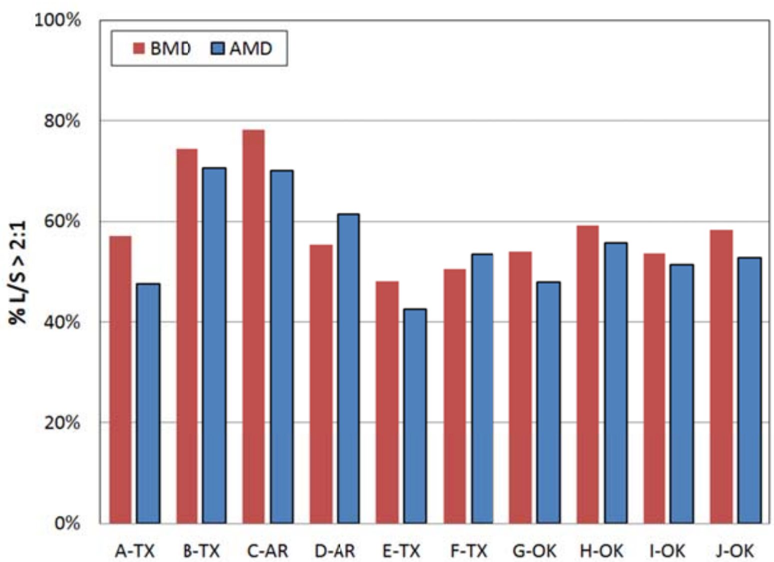


Figure 3.12 Particle Shape Before and After Micro-Deval

Unconfined freezing and thawing (also known as Canadian freeze-thaw) loss was determined according to CSA A23.2-24A. (2004). Unbound aggregates went through five cycles of freezing and thawing. Figure 3.13 shows the unconfined freezing-and -thawing loss. I-OK experienced the highest loss of 17.8%, whereas B-TX has the lowest at 1.2%. MTO requires unconfined loss of 6% or lower for coarse aggregates to be used in concrete pavements. MD loss combined with unconfined freezing and thawing loss can predict field performance with 95% accuracy (Rogers et al. 2003).

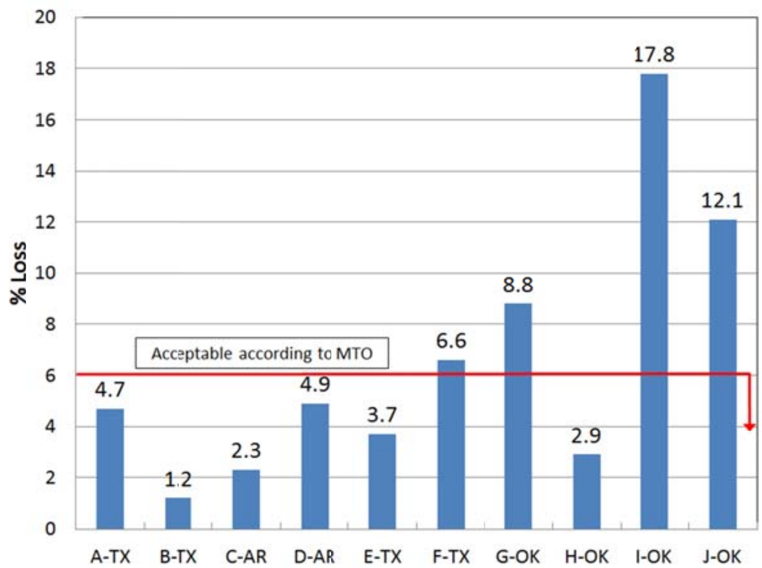


Figure 3.13 Unconfined (Canadian) Freeze and Thaw Loss of Aggregates

Aggregate crushing value (ACV) was determined according to TxDOT recommended method Tex-1xx-E (Unpublished). According to this method, there are three ACVs. ACV4, ACD40 and ACV200 are percent ACV loss based on sieving at #4, #40 and #200 sieves, respectively. ACV4 is presented in this report. Figure 3.14 shows the ACV4 for the project aggregates. F-TX showed the highest loss of 41% and H-OK showed the least at 28%. There is no known acceptable limit for ACV.

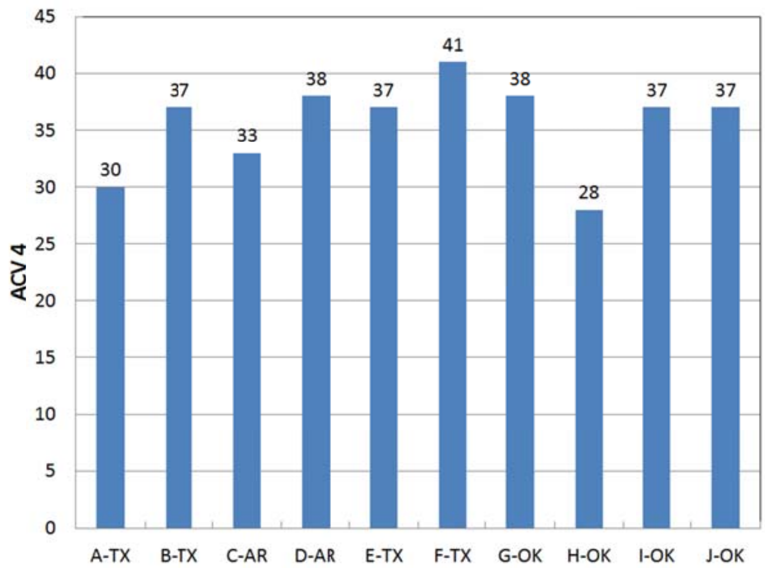


Figure 3.14 Aggregate Crushing Value

3.4 Concrete Testing

The following concrete testing was performed on concrete cylinder specimens made from each of the project aggregates.

- Seven-day and twenty-eight day compressive strength.
- Twenty-eight day modulus of elasticity.
- Coefficient of thermal expansion (CoTE).

The mixture design for all ten concrete mixes conformed to Tex-428-A (2011). Before mixing, coarse aggregates were regraded according to Tex-428-A (2011). The 4-in. X 8-in. cylinders were made according to ASTM C192 (2007). Compressive strength and modulus of elasticity were measured according to Tex-418-A (2008) and ASTM C469 (2010), respectively. The coefficient of thermal expansion for concrete cylinders made with the selected aggregates was determined in accordance to Tex-428-A (2011). Compressive strengths of the concrete at 7 and 28 days are shown in Figure 3.15.

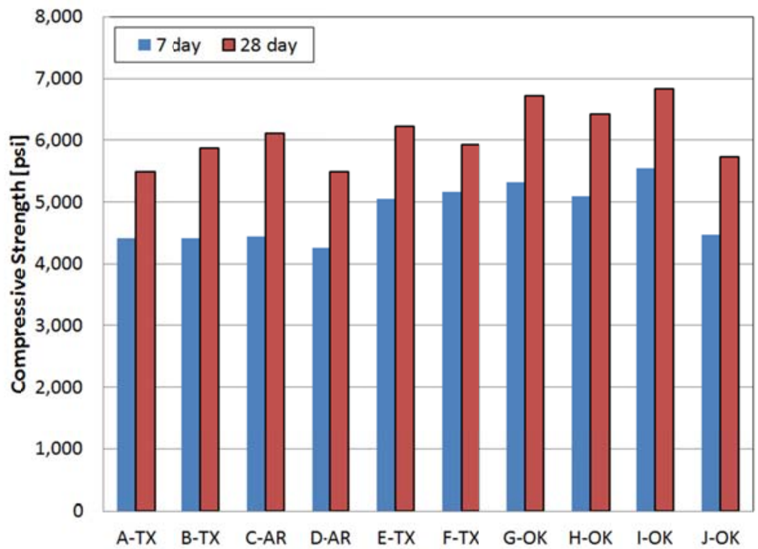


Figure 3.15 Seven-Day and 28-Day Compressive Strength of Concrete

Since concrete strengths for mix designs in the average strength range are primarily controlled by the water-cement ratio (W/C) and all 10 mixes were controlled to the same W/C, all specimens fell into the 4,000 to 5,000 psi range at 7 days and 5,500 to 7,000 psi range at 28 days. All the concrete mixes satisfied the class P concrete strength requirements according to item 360. Figure 3.16 shows the 28-day modulus of elasticity (E) of the ten concrete mixes. Modulus of elasticity varied from 5.2 million to 3.5 million psi. A-TX had the highest E and I-OK had the lowest. River gravels showed relatively higher E than other types of aggregates. Sand stone showed the lowest E, probably because of the porous structure of the aggregates. Figure 3.17 shows the relationship between 28-day E and absorption of aggregate. Higher aggregate absorption showed lower E. Higher absorption represents more porous structure and results in lower E.

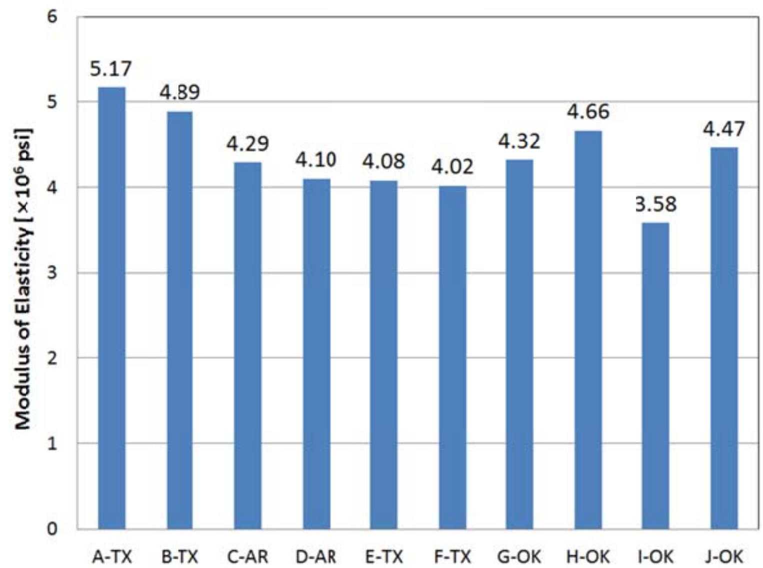


Figure 3.16 Modulus of Elasticity for Concrete Specimens at 28 Days of Age

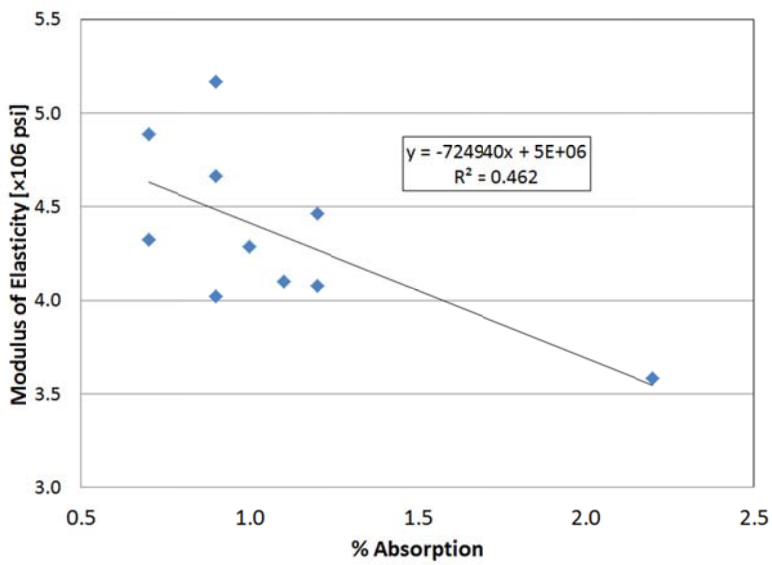


Figure 3.17 Comparison of 28-day Modulus of Elasticity of Concrete and % Absorption of Aggregate

For the CoTE testing, a new protocol developed by TxDOT in 2012 was used. As described in the previous chapter, a DVRT was used as the displacement measuring device instead of an LVDT. Figure 3.18 shows the various components of the CoTE setup at CTR. A programmable water bath was used to change the temperature of the concrete cylinder at the range of 10°C to

50°C. An RTD was used to measure the water temperature. Each RTD was calibrated using an ice bath. Water temperature and length change of the concrete cylinder were measured every minute. 4-in. X 7-in. cylinders were used to determine the CoTE of the concrete mix. A 4-in. X 8-in. cylinder was trimmed at one end to achieve the 7-in. length. A 3-in. X 7-in. stainless steel reference cylinder was first calibrated by TxDOT and then used for determining the correction factor of the frame. Figure 3.20 shows the frame and reference stainless steel cylinder. Before starting the test, the RVDT was physically nulled. Then each cylinder was subjected to three cycles of heating and cooling. Figure 3.21 shows the three cycles of heating and cooling along with the length change of the concrete cylinder. The CoTE of the concrete cylinder was calculated according to Tex-428-A (2011).

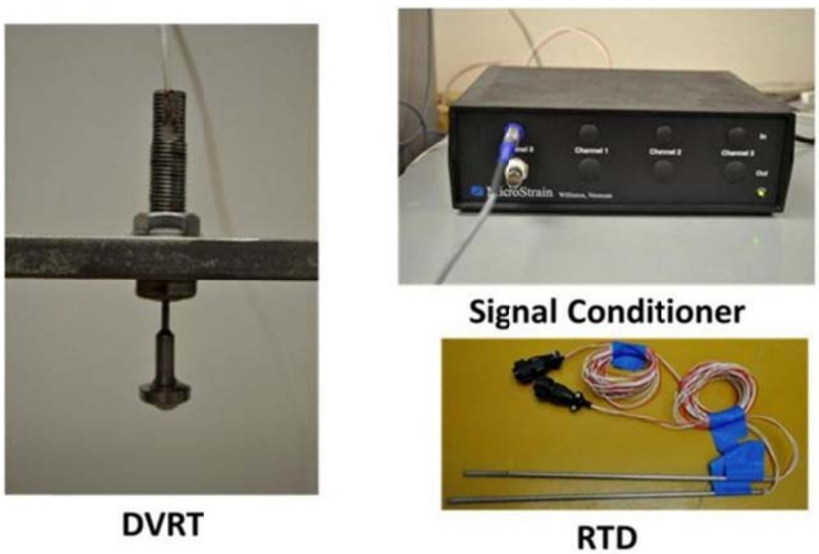


Figure 3.18 Various Components of CoTE Setup

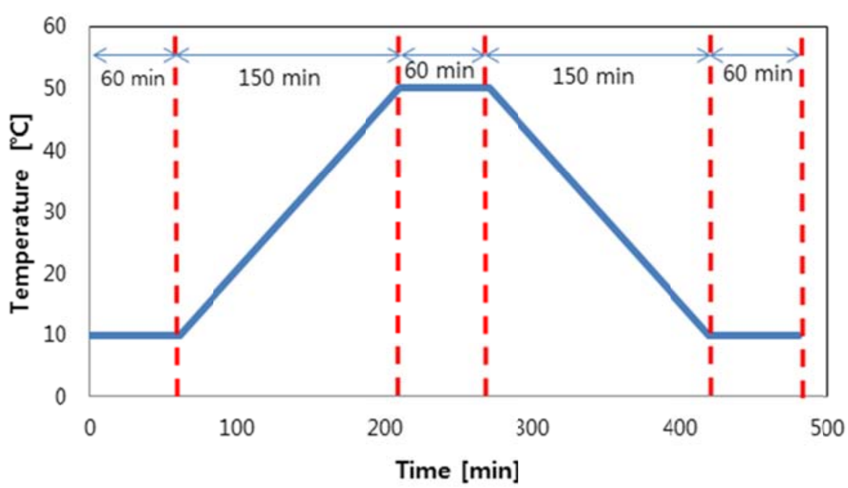


Figure 3.19 One Cycle of Heating and Cooling

Figure 3.20 CoTE Frame and Reference Stainless Steel Cylinder

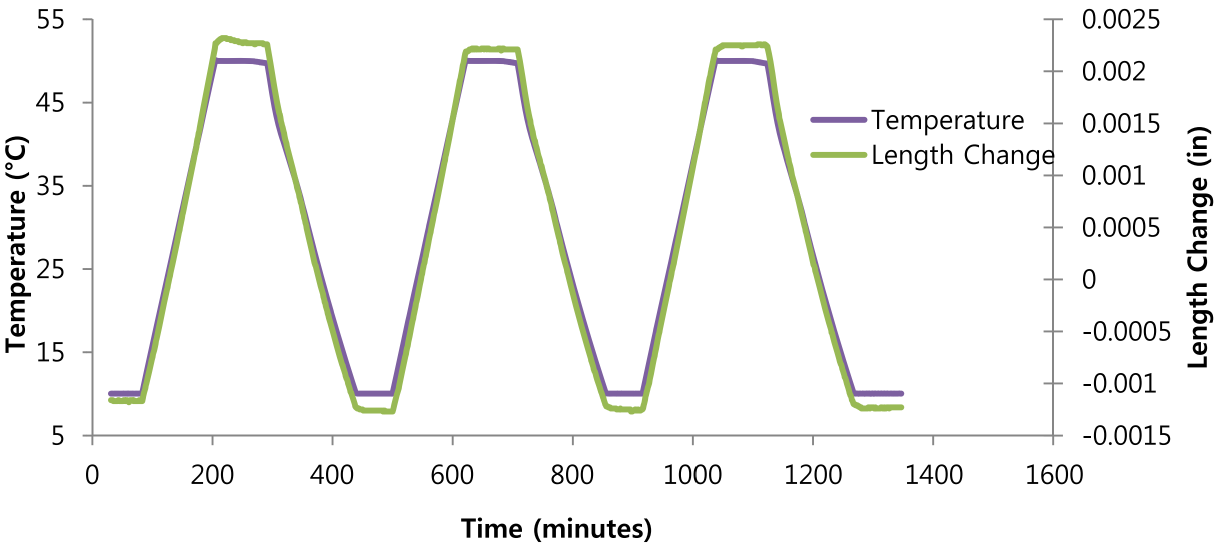


Figure 3.21 Three Cycles of Heating and Cooling and Change in Length for 4-in. x 7-in. Concrete Cylinder of B-TX

Figure 3.22 represents the CoTE of ten concrete mixes. J-OK (Slate) had the highest CoTE and H-OK (Rhyolite) had the lowest CoTE. Igneous rock showed the lowest CoTE among all the aggregates type. River gravel (A-TX, B-TX, C-AR, and D-AR) showed higher CoTE than the blend of river gravel and limestone (E-TX and F-TX). This is an indication of a possible way to reduce the CoTE of concrete by blending lower CoTE aggregates with higher CoTE aggregates.

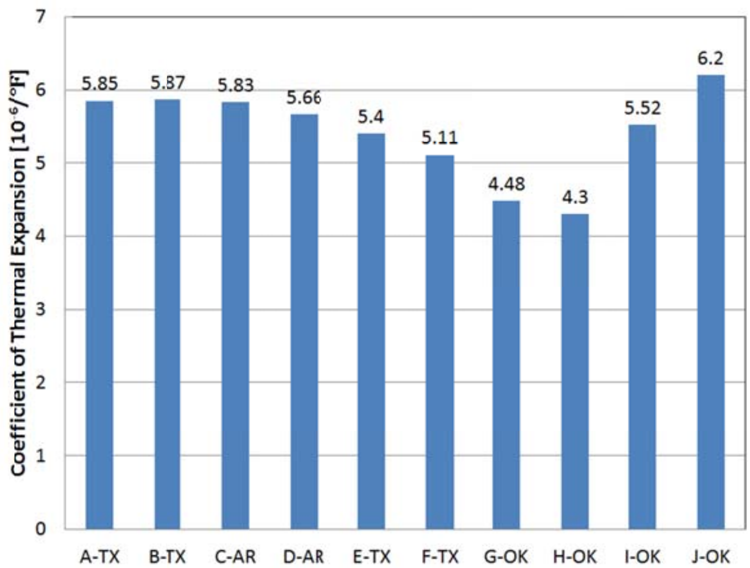


Figure 3.22 CoTE for Concrete Specimens Older than 28 Days

3.5 Conclusions

The following conclusions can be drawn from the above discussion:

1) All ten aggregate sources qualified according to Item 421 requirements of Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.
2) Three sources failed to meet MTO's unconfined freezing and thawing requirement, and one source did not meet MTO's MD requirement. When combining unconfined freezing and thawing results and MD results, four sources do not qualify.
3) All the concrete mixes satisfied the class P concrete strength requirements according to item 360.
4) River gravel showed the highest 28-day modulus of elasticity. Aggregate with higher absorption showed lower 28-day modulus of elasticity.
5) Slate showed the highest CoTE and igneous rock had the lowest CoTE. River gravel showed higher CoTE than the river gravel and limestone blend, justifying the potential of reducing concrete CoTE by blending low CoTE aggregate with high CoTE aggregate.

Chapter 4: Cost Analysis of Coarse Aggregate

4.1 Aggregate Cost Analysis

The material properties of coarse aggregate are considered to be an important factor when designing and constructing concrete pavement. In many cases, however, the cost and availability of coarse aggregate can also have a substantial influence on project outcomes. For this reason, the following cost analysis has been completed in order to provide TxDOT with a better understanding of the current coarse aggregate market. In addition to presenting cost information from acceptable aggregate sources, another objective of this task was to compare construction costs of continuously reinforced concrete pavement (CRCP) with concrete pavement contraction design (CPCD). Cost effective recommendations for pavement type in the districts of Amarillo, Atlanta, Houston, Paris and Wichita Falls are provided accordingly. In evaluating cost effectiveness of alternative materials or methods, it should be recognized that material costs and other values can easily become dated and the most current values should be obtained and used.

4.1.1 Methodology

4.1.1.1 Coarse Aggregate

Sources of coarse aggregate were identified by obtaining a list of active quarries from TxDOT. Additional sources were obtained by collaborating with TxDOT district engineers, concrete pavement contractors and ready-mixed concrete suppliers. The following information for coarse aggregate conforming to the grade requirements of a Class P concrete pavement mix design was collected via phone interviews with 28 aggregate quarry representatives (including 10 quarries selected for material property testing as shown in Chapter 3):

- Aggregate Type (e.g., Gravel)
- Aggregate Cost ($/ton)
- Transportation Cost ($/ton/mile)

The transportation distance from the quarry location to the district center was used to calculate the delivery cost. Then the total cost of aggregate was calculated by summing the material and delivery costs. Similar cost information was also acquired from seven rail yards that supply aggregate for the Houston district.

4.1.1.2 Ready-Mixed Concrete

Sources of ready-mixed concrete were identified by collaborating with TxDOT district engineers, aggregate quarry representatives and concrete pavement contractors. Additional sources were obtained via the World Wide Web. The following information for ready-mixed concrete conforming to the requirements of a Class P concrete pavement mix design was collected via phone interviews with eight concrete supplier representatives:

- Concrete Cost Including Delivery ($\$/yd^3$)
- Aggregate Source (e.g., B1'-AR)
- Aggregate Type (e.g., Gravel)
- Aggregate Cost Including Delivery ($/ton)
- Concrete Density (lb/yd^3)
- Aggregate Content by Weight (%)

4.1.1.3 Concrete Pavement

Sources of concrete pavement construction were identified by collaborating with TxDOT district engineers, aggregate quarry representatives and ready-mixed concrete suppliers. Additional sources were obtained via the World Wide Web. The following historical information for CRCP and CPCD projects including stipulations for a Class P concrete pavement mix design was collected via phone interview with a pavement contractor representative:

- Width, Length, Depth, Area (ft, ft, in, yd^2)
- Total Pavement Cost ($\$/yd^2$)
- Concrete Cost Including Delivery ($\$/yd^3$)
- Concrete Quantity (yd^3)
- Aggregate Cost Including Delivery ($/ton)

Qualitative information was also collected to provide insight from the experience and perspective of the concrete pavement contractor regarding the construction of CRCP and CPCD. Finally, the TxDOT Twelve Month Average Low Bid Unit Prices for Statewide Construction as of February 23, 2012, were investigated to gain an understanding of historical concrete pavement costs in Texas.

4.1.2 Results

4.1.2.1 Coarse Aggregate

Figure 4.1 illustrates where the quarries in each district fall within this range. The availability and cost information collected via phone interviews with aggregate quarry representatives is presented in Table 4.1. The average material cost of coarse aggregate is $11.17/ton, ranging from $6.90 to $21.00. Also, the average transportation cost for delivery is $0.17/ton/mile, ranging from $0.14 to $0.22. For those quarries that do not offer delivery services, a rate of $0.20/ton/mile was assumed to calculate total cost. If the quote received from a representative seemed to be above the typical price range, another phone interview was conducted to confirm the cost information. It is also important to note that 75% of the quarries surveyed are located more than 50 miles from their respective district centers. Consequently, the average total cost of coarse aggregate including delivery is \$23.75/ton, ranging from \$13.13 to \$39.40. Figure 4.2 indicates locations of aggregate venders included in the study.

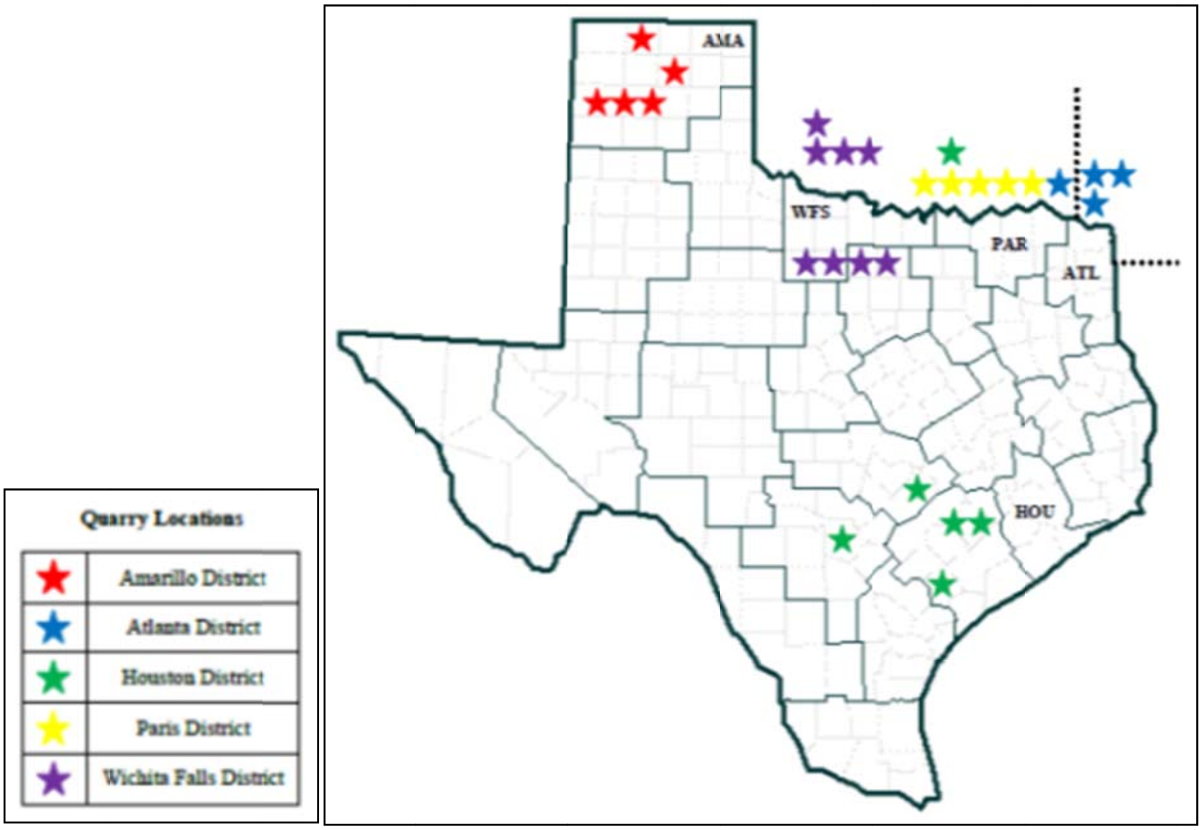
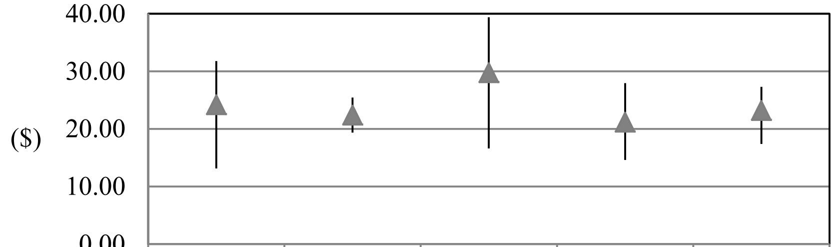


Figure 4.1 Location of Aggregate Vendors Used in This Study

Table 4.1 Availability and Cost of Coarse Aggregate

District	Vendor	Type	Material Cost ($/ton)	Transportation Cost ($/ton/mile)	Total Cost ($/ton)
Amarillo	A1'-TX	Gravel	21.00	0.22	31.78
	A2'-TX	Limestone	21.00	0.17	22.19
	A3'-TX	Gravel	12.50	0.21	13.13
	A4'-TX	Limestone	14.00	0.17	27.77
	A5'-TX	Gravel	16.50	0.21	26.16
Atlanta	B1'-AR	Gravel	9.25	**	22.65
	B2'-AR	Gravel	10.00	0.17	19.35
	B3'-AR	Gravel	8.25	**	22.65
	B4'-OK	Gravel	8.50	0.14	21.94
	B5'-OK	Sandstone	8.50	0.14	25.44
Houston	C1'-TX	Gravel	11.50	**	37.10
	C2'-TX	Gravel	13.00	**	16.60
	C3'-TX	Gravel	12.00	**	39.40
	C4'-TX	Gravel	13.00	0.14	26.02
Paris	D1'-OK	Limestone	7.55	0.15	27.95
	D2'-OK	Limestone	7.55	0.15	22.40
	D3'-OK	Sandstone	12.00	0.17	17.78
	D4'-OK	Limestone	7.00	**	24.00
	D5'-OK	Gravel	8.50	0.14	20.26
	D6'-OK	Sandstone	8.50	0.18	14.62
Wichita Falls	E1'-OK	Limestone	7.40	0.15	22.85
	E2'-OK	Limestone	6.90	0.15	17.40
	E3'-TX	Gravel	15.00	0.18	25.62
	E4'-TX	Limestone	12.00	0.18	22.98
	E5'-TX	Limestone	10.00	0.18	23.50
	E6'-OK	Granite	13.35	0.15	25.50
	E7'-TX	Limestone	10.50	0.15	20.55
	E8'-OK	Granite	7.50	0.18	27.30

* Quarry selected for material property testing in Subtask 2.1.
** Quarry does not offer delivery services (assumed $0.20/ton/mile).



	Amarillo	Atlanta	Houston	Paris	Wichita Falls
High	31.78	25.44	39.40	27.95	27.30
Low	13.13	19.35	16.60	14.62	17.40
▲Average	24.21	22.41	29.78	21.17	23.21

Figure 4.2 Total Cost of Coarse Aggregate Including Delivery Per Ton

In many cases, rail may be more efficient at moving large volumes of materials to market. The availability and cost information collected from rail yards from vendor F1' that supply aggregate for the Houston district is presented in Table 4.2. The average cost of the aggregate is $26.71/ton, which includes the material cost and the transportation cost incurred from transporting the aggregate from its source quarry. However, similar delivery charges still apply to transport aggregate from the yard to a jobsite. Despite the increased distance from its source, the cost of aggregate obtained from a rail yard in Houston is comparable to the cost of aggregate obtained from a quarry when transportation costs are considered.

The cost of aggregate at the F1' San Antonio quarry is $9.75/ton. San Antonio is roughly 200 miles from Houston, and the aggregate is sold for $26.50/ton in the rail yards. This equates to a rail transportation cost of approximately $0.08/ton/mile. The cost of aggregate at the F1' Mill Creek quarry is $12.75/ton. Mill Creek is roughly 400 miles from Houston, and the aggregate is sold for $27.00/ton in the rail yards. This equates to a rail transportation cost of approximately $0.04/ton/mile. The rail transportation costs are substantially lower when compared to the delivery truck transportation costs of $0.14 to $0.22/ton/mile.

Table 4.2 Availability and Cost of Coarse Aggregate from Rail Yards in Houston

Vendor	Rail Yard Location	Type	Material Cost ($/ton)	Transportation Cost ($/ton/mile)	Source Quarry Location	Material Cost at Source Quarry ($/ton)
F1'	Houston, TX	Limestone	26.50	0.16	San Antonio, TX	9.75
	Baytown, TX	Limestone	27.00	0.16	San Antonio, TX	9.75
	Houston, TX	Limestone	26.50	0.16	San Antonio, TX	9.75
	Humble, TX	Limestone	26.50	0.16	San Antonio, TX	9.75
	Rosenburg, TX	Limestone	26.50	0.16	San Antonio, TX	9.75
	Rosharon, TX	Granite	27.00	0.16	Mill Creek, OK	12.75
	Tomball, TX	Granite	27.00	0.16	Mill Creek, OK	12.75

Most of the surveyed quarries included in this study are in close proximity to rail (0-10 miles). The only exceptions found were the three Zack Burkett quarries, which are 30-60 miles from the nearest rail. Thus, considerable project cost savings could be realized by utilizing rail if aggregate is required to be transported from a distant source.

A quarry representative acknowledged that discounts for aggregate are given at the discretion of the salesman on a per project basis. Typically, account customers receive better rates than cash or credit card customers. The size and scope of the project also affect the cost as well as the aggregate type and availability.

Another quarry representative provided cost information in relation to project size. Table 4.3 presents the effect of project size on aggregate cost. According to the representative, due to the small profit margin that is characteristic of the aggregate industry, only a minor percentage of unit cost savings can be realized by purchasing aggregate in large quantities.

Table 4.3 Effect of Project Size on Coarse Aggregate Cost

Project Size	Pavement Area (yd^2)	Pavement Depth (in)	Pavement Volume (yd^3)	Aggregate Volume (yd^3)	Aggregate Mass (ton)	Aggregate Cost ($/ton)	Unit Cost Savings (%)
Small	5000	11	4583	3208	4492	13.00	0.00
Medium	50000	11	45833	32083	44917	12.75	1.92
Large	100000	11	91667	64167	89833	12.50	3.85

4.1.2.2 Ready-Mixed Concrete

The cost information collected via phone interviews with ready-mixed concrete supplier representatives is presented in Table 4.4 The average cost of Class P concrete is $86.31/$yd^3$, ranging from $75.00 to $95.00. Also, the average cost of coarse aggregate including delivery is comparable to quotes received directly from the aggregate quarries, ranging from $15.30 to

$20.00/ton.

Table 4.4 Cost and Density of Ready Mixed Concrete Including Coarse Aggregate Data

District	Vendor	Concrete Cost ($/yd^3)	Aggregate Source	Aggregate Type	Aggregate Cost ($/ton)	Concrete Density (lb/yd^3)	Aggregate Content by Weight (%)
Amarillo	A1”	95.00	Texas S & G	Gravel	16.50	4010	42
	A2”	95.00	Texas S & G	Gravel	16.50	4010	42
Atlanta	B1”	90.00	Trinity Materials	Gravel	20.00	4000	42
	B2”	94.50	Hanson	Gravel	15.30	4000	43
Houston	C1”	80.00	Martin Marietta	Limestone	Proprietary	3900	45
Paris	D1”	86.00	Smith-Buster	Sandstone	16.00	4000	39
Wichita Falls	E1”	75.00	Dolese / E&A	Lim / Gra	19.00 / 17.25	3986	42
	E2”	75.00	Dolese / E&A	Lim / Gra	19.00 / 17.25	3986	42

The average density of the concrete is 3987 lb/yd^3, and if the average aggregate content of concrete by weight is 42%, then it can be concluded that an average of 1675 lb of coarse aggregate is used for a single yd^3 of Class P concrete. Based on this conclusion and the information obtained from aggregate quarries, the average total cost of coarse aggregate including delivery per yd^3 of Class P concrete is $20.34.

4.1.2.3 Concrete Pavement

The cost information collected via phone interview with the concrete pavement contractor representative is presented in Table 4.5, including the cost and dimensions of CRCP and CPCD pavement types utilizing a Class P concrete mix design. Proprietary information has been withheld to protect the anonymity of the surveyed contractor. An initial assessment of this information indicates that the contractor receives preferred pricing from its vendors for both coarse aggregate and ready-mixed concrete. Although the aggregate costs are still within the range of the previously surveyed information, the concrete costs are actually below the surveyed range. This difference in pricing is most likely due to discounts that the contractor receives from vendors for large purchase orders within a sustained business relationship. The contractor also indicated that installation of CPCD was preferred due to the lack of steel and the ease of installing dowels; however, installation of steel reinforcement for CRCP was difficult and time consuming.

Table 4.5 Dimensions and Cost of Concrete Pavement Projects Including Ready Mixed Concrete and Coarse Aggregate Data

Pavement Type:	CRCP	CPCD
Width (ft):	68	38
Length (ft):	9069	17026
Depth (in):	9	12
Area (yd^2):	80953	102415
Total Pavement Cost ($\$/yd^2$):	42.50	45.25
Concrete Cost ($\$/yd^3$):	68.50	61.50
Concrete Quantity (yd^3):	21655	36163
Aggregate Cost ($/ton):	20.34	17.50

To standardize the comparison between the concrete pavement projects, the total pavement cost was converted to $\$/yd^3$ while accounting for the differing pavement depths. This conversion yielded total costs of $\$170.00/yd^3$ for CRCP and $\$135.75/yd^3$ for CPCD. If the average total cost of coarse aggregate (including delivery) per yd^3 of Class P concrete is $20.34, and similar CRCP and CPCD projects yield similar converted costs, then it can be concluded that coarse aggregate accounts for approximately 12% and 15% of the total cost of CRCP and CPCD, respectively.
To form a baseline for comparison between CRCP and CPCD historical costs in Texas, a cost analysis was conducted utilizing the TxDOT Twelve Month Average Low Bid Unit Prices for Statewide Construction as of February 23, 2012. The results of the analysis are presented in Table 4.6. Depths, quantities and unit costs are based on the twelve month averages. A total quantity of 8,426,951 yd^2 of CRCP was constructed in the past twelve months compared to 311,878 yd^2 of CPCD. To determine the average unit cost for each pavement type, the sum of the unit costs divided by their depths and multiplied by their respective quantities was divided by the total quantity. CRCP has an average unit cost of $\$3.27/yd^2/in$ compared to $\$3.63/yd^2/in$ for CPCD. The additional material and labor costs associated with the steel reinforcement required for CRCP construction have a substantial influence on total project cost, while construction costs associated specifically with CPCD include saw cutting and doweling. The most influential factor effecting average units costs, however, is the difference in the total quantities of pavement constructed. The lower quantity of CPCD construction implies that the projects are both smaller in scale and in numbers. The consequence of this limiting factor is that contractors often charge a premium when performing work on a smaller scale.

Table 4.6 Cost Analysis of CRCP and CPCD Construction Based on TxDOT 12-month Average Low Bid Unit Prices

Description	Depth (in)	Quantity (yd^2)	Unit Cost ($/$yd^2$)	Total Quantity (yd^2)	Unit Cost/Depth ($/$yd^2$/in)	Quantity×Unit Cost/Depth ($/in)	Average Unit Cost/Depth ($/$yd^2$/in)
CRCP Construction	6	2,496	50.10	8,426,951	8.35	20,843.55	3.27
	7	399,942	26.08		3.73	1,490,270.74	
	7.5	37,325	49.16		6.55	244,665.03	
	8	838,475	32.23		4.03	3,378,424.71	
	8.5	10,284	45.00		5.29	54,444.71	
	9	520,980	31.19		3.47	1,805,723.02	
	10	1,607,702	36.50		3.65	5,868,651.46	
	10.5	7,523	68.00		6.48	48,720.38	
	11	784,453	38.03		3.46	2,712,033.73	
	11.5	46,531	41.00		3.57	165,893.13	
	12	895,254	37.35		3.11	2,786,721.19	
	13	2,430,660	38.72		2.98	7,239,976.96	
	14	603,416	30.45		2.17	1,312,250.50	
	15	241,910	25.57		1.70	412,435.26	
CPCD Construction	8	160,227	24.62	311,878	3.08	493,175.90	3.63
	9	19,967	42.36		4.71	93,982.36	
	10	5,699	43.36		4.34	24,711.20	
	11	19,203	63.00		5.73	109,980.82	
	12	102,440	45.25		3.77	386,284.17	
	13	4,342	72.00		5.54	24,048.00	

4.2 Life-Cycle Cost Analysis

There will be many factors to consider when determining pavement type for a road, and it is necessary to examine the initial cost as well as costs associated with the performance of the road over time (i.e. – maintenance costs). Software that performs a life-cycle cost analysis (LCCA), allowing for the comparison of two alternative options, can be a valuable tool in assisting with the decision making process. For this project, the purpose of an LCCA is to apply the costs obtained for coarse aggregate to determine what the best pavement alternative for a particular road may be based solely on the price of the coarse aggregate.

4.2.1 Methodology

Running an LCCA can be done several ways, but the most widely accepted method is using a computer-based software program. The Federal Highway Administration's (FHWA) method of performing life-cycle cost analysis is the computer program RealCost. The RealCost interface requires the user to enter inputs in various screens, which it then applies a series of algorithms to, in order to determine which of the two given alternatives is the superior choice based on the inputs. To be most accurate, an LCCA requires precise information pertaining to the specific job being assessed. For the purposes of this research, two hypothetical scenarios were imagined. The process used to determine the pricing for both scenarios was the same. The determination of the variables will be discussed in the context and order in which they appear in RealCost. After the description of how the inputs were determined, both case studies are presented to show how the LCCA can be applied to examine the effect of coarse aggregate selection on overall pavement cost.

Due to the complex nature of the inputs required, and the limited time allotted for this project, and in order to obtain the best representative numbers, inputs were gathered from several sources to perform the LCCAs for the case studies contained below. The inputs will be discussed in the order in which they appear in the RealCost program. Due to the specific nature of the LCCA program, there were two sets of LCCAs run: one for a hypothetical project in Mount Pleasant, in the Atlanta district and one for a hypothetical project in Houston, in the Houston district. Both sets of LCCAs were run using primarily national and Texas data, supplemented with comparable data from the State of California Department of Transportation (CalTRANS).

For the Atlanta and the Houston district each, a hypothetical project was imagined where the Concrete Pavement Contraction Design (CPCD) utilized a local aggregate, and the Continuously Reinforced Concrete Pavement (CRCP) utilized an imported aggregate. This would allow for the comparison of the fairly constant locally sourced coarse aggregate and the escalating costs of imported coarse aggregate. The project was set as a portion of roadway two lanes wide (24 feet) and one mile long (5280 feet). For the purposes of evaluation, the concrete thickness for the CRCP was defined as 9 inches, and the concrete thickness for the CPCD was defined as 11 inches.

This project definition is what all calculations were based on, for both the examples in the Atlanta district and the Houston district.

4.2.1.1 Determination of LCCA Inputs

The LCCA inputs will be presented in the initial order in which they are required to run the analysis in *RealCost*. After the general discussion of inputs that apply to both case studies, the specific case studies and calculation of inputs will be discussed as they are specific to both districts.

1. Project Details

The Project Details contain the physical details of the project being analyzed. It is general information, generally project specific. These inputs include:

- State Route – Identifies the road or highway.
- Project Name – Identifies the particular project.
- Region – Identifies the region of the state.

- County – Identifies the county in the state.
- Analyzed By – Identifies the person performing the analysis
- Beginning Milepost
- Ending Milepost
- Comments – Space to add any notes or comments

For the purposes of this project, the research team defined these inputs while the LCCAs were being run. An example of the screen is given in Figure 4.3, along with the general data that applied to each of the LCCAs run in Table 4.7.

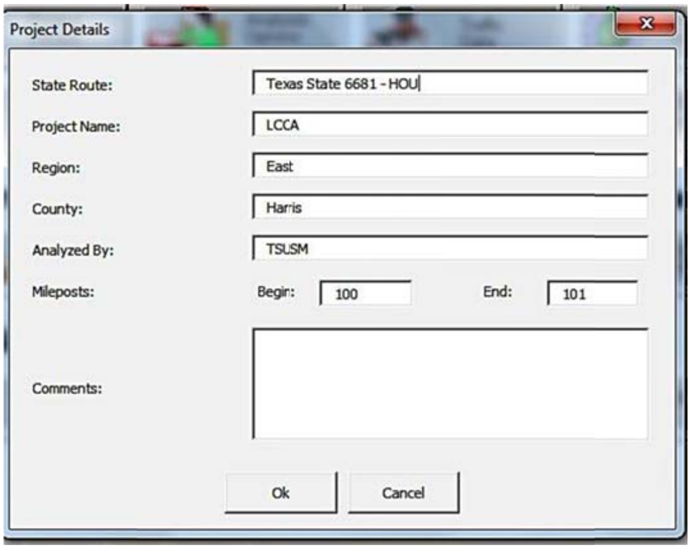


Figure 4.3 Example of Project Details Screen

Table 4.7 Specific Inputs for Case Studies for Project Details

	Atlanta	Houston
State Route	Texas State 6681 - ATL	Texas State 6681 - HOU
Project Name	LCCA	LCCA
Region	North East	East
County	Titus	Harris
Analyzed By	TSUSM	TSUSM
Mileposts	100;101	100;101
Comments	(none)	(none)

2. Analysis Options

This panel allows the user to set the analysis options for the Alternatives. These inputs include:

- Analysis Units – Choose English or Metric. All LCCAs run used English.
- Analysis Period (years) – The number of years for which the program will run the analysis. TxDOT, supported by the research team, defined this number as 50, the expected service life of the concrete roads being analyzed.
- Discount Rate (%) – The discount rate the program will apply to the costs for the analysis period. This number is generally between 2-4% nationally. A discount rate of 4% was used on all LCCAs in this project to cover all potential angles.
- Beginning of Analysis Period – The year the user wants the analysis to begin. All LCCAs in this project were run beginning in 2012.
- Include Agency Cost Remaining Service Life Value – Check box. This box was left “checked” in all LCCAs run.
- Include User Costs in Analysis – Check box. This box was left checked in all LCCAs run.
- User Cost Computation Method – Select “Calculated” or “Specified.” “Calculated” was selected for all LCCAs run.
- Traffic Direction – Select “One-Way” or “Both.” “Both” was specified for all LCCAs in this project.
- Include User Cost Remaining Value – Check box. This box was left “checked” for all LCCAs run in this project.
- Number of Alternatives – Select 1 or 2. The number “2” was selected for all LCCAs run, as it is a comparison program, but is currently only able to compare two alternatives at one time.

Figure 4.4 below shows an example of an Analysis Options screen from this project. The same analysis options were used for the LCCAs run for both the Atlanta and the Houston districts.

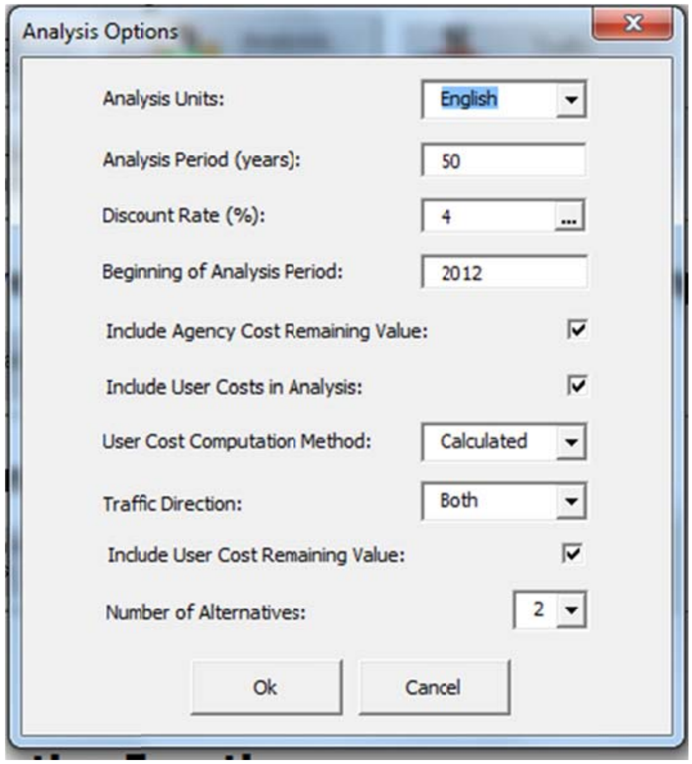


Figure 4.4 Analysis Options Screen Example

The two alternatives chosen for the life-cycle cost analyses in this project are defined in Table 4.8. These alternatives were determined by the research team in response to the findings of the CoTE testing of coarse aggregate.

Table 4.8 Two Alternative Situations

	CRCP	**CPCD**
Aggregate Type	Gravel	Limestone
Location	Local	Imported

By comparing CPCD and CRCP from initial construction through the design life of the pavement using a program such as *RealCost*, it is possible to examine ways to mitigate cost over time by making construction and maintenance decisions before the roads are built. *RealCost* applies predetermined algorithms to the inputs to establish cost over time, allowing the user to examine the costs associated with construction and maintenance in conjunction with the anticipated wearing on the roadway as applied by the software.

3. Traffic Data

To calculate user costs, as chosen in the Analysis Options screen, the program uses work zone traffic data. User costs can add a significant amount of money to the overall life-cycle cost of the road, so the program factors them in based on the traffic information provided, including:

- AADT at Beginning of Analysis Period (total both directions) – The annual average daily traffic level for the year in which the analysis period is set to begin. Based on an assumption that a 4-lane highway can accommodate an AADT of 150,000, and with the general assumption of the situation to be a one-lane highway, an AADT of 37,500 (or one fourth the AADT of a 4-lane highway) was used.
- Single Unit Trucks as Percentage of AADT – Based on both national and local information (CalTRANS; Bronzini, 2008), the single unit truck percentage was set at 7%.
- Combination Trucks as Percentage of AADT – Based on both national and local information (CalTRANS; Bronzini, 2008), the combination unit truck percentage was set at 8%
- Annual Growth Rate of Traffic – An average annual growth rate of 1.2% was used, as supported by national and local information (CalTRANS; Qu, Lee, Huang, 1997; Bronzini, 2008).
- Speed Limit Under Normal Operating Conditions – This input was defined as 70, as that is a common speed limit in Texas on two-lane State Highways.
- Lanes Open in Each Direction under Normal Conditions – As the example was set as a two-lane mile, the input here was defined as "1" to indicate one mile open in each direction.
- Free Flow Capacity (vphpl) – *RealCost* has a built in Free Flow Capacity calculator, which was used here to calculate the Free Flow Capacity. An example of the calculator is shown below the Traffic Data screen example (Figure 4.5) in Figure 4.6.
- Queue Dissipation Capacity (QC) – CalTRANS provides a formula to calculate the queue dissipation capacity, which was used to calculate this input.

$$\mathrm{QC} = \frac{\mathrm{Q} \times 100}{100 + \mathrm{P} \times (\mathrm{E} - 1)}$$

Q is equal to the base capacity of the lane, which is generally 1,800 passenger cars per hour per lane, or "pcphpl," which is the number used in this case.

  - P is the percentage of heavy vehicles at the project location. Based on Bronzini's (2008) study involving percentage of heavy commercial traffic, which specifically examines Houston (among other locations) this input was set to "9." It was left as "9" for the Atlanta district for general comparison purposes.
  - E is an equivalency factor developed to represent the physical features of the road. For example, "Level" has a value of 1.5, "Rolling" has a value of 2.5, and "Mountainous" has a value of 4.5. For this project, the inputs for the Atlanta district were determined using a 2.5 ("Rolling"), while the inputs for the Houston district were calculated using a 1.5 ("Level").

- Maximum AADT (both directions) – The max AADT was also determined using a formula provided by CalTRANS in their *RealCost* handbook.

$$\mathrm{AAST_{max}} = \frac{\mathrm{M \times N \times 100}}{\mathrm{100 + P \times (E - 1)}}$$

  - The value for M is given as "43,000" for two-lane highways. This was the number used in the calculations.
  - N is the number of total lanes, which for all LCCAs in this project, is "2."
  - P is again the percentage of heavy truck traffic, which is "9" for all LCCAs run for this project. This is based on the assumption that not all truck traffic is necessarily "heavy" truck traffic, and that the levels of heavy truck traffic can fluctuate from 2-20% based on time of day and time of year.
- Maximum Queue Length – Research suggests that seven miles is the maximum acceptable queue length, so this number was used to imagine a "worst case" scenario for both case studies.
- Rural or Urban Hourly Traffic Distribution – Choose "Rural" or "Urban." "Urban" was chosen for all LCCAs run for this project.

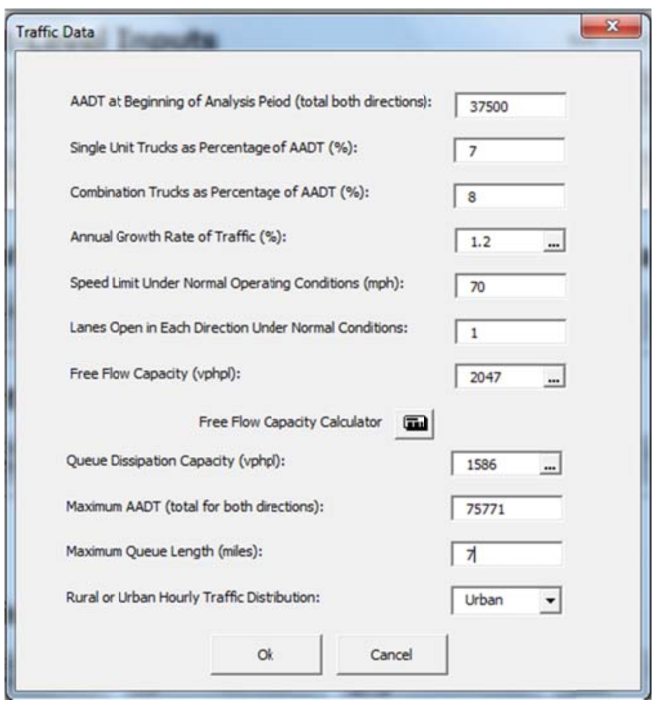


Figure 4.5 Traffic Data Screen Example

Table 4.9 Inputs for Traffic Data

	Atlanta	Houston
AADT at Beginning of Analysis Period	37500	37500
Single Unit Trucks as Percentage of ADT (%)	7	7
Combination Trucks as Percentage of ADT (%)	8	8
Annual Growth Rate of Traffic (%)	1.2	1.2
Speed Limit Under Normal Operating Conditions (mph)	70	70
Lanes Open In Each Direction Under Normal Conditions	1	1
Free Flow Capacity (vhphpl)	1883	1883
Queue Dissipation Capacity (vhphpl)	1586	1722
Maximum AADT (total for both directions)	75771	82297
Maximum Queue Length (miles)	7	7
Rural or Urban Hourly Traffic Distribution	Urban	Urban

It should be noted that the traffic data applies to the calculation of user cost, and this LCCA focuses only on the agency cost. Due to the non-impact of the traffic data on the agency cost, this data was not considered a key focus.

4. Value of User Time

The purpose of these LCCAs was to evaluate agency costs, and the Value of User Time is used to calculate user costs. The program assesses user costs based on calculations it makes using values for user time input by the person doing the analysis. There are many factors to consider when calculating user cost, and it can become very complicated. For the LCCAs run for this project, calculations were based on predetermined average highway user costs from CalTRANS, and approved by the Project Director. The inputs required are:

- Value of Time for Passenger Cars ($/hour) – $11.51 for all LCCAs run for this project.
- Value of Time for Single Unit trucks ($/hour) – $27.83 for all LCCAs run for this project.
- Value of Time for Combination Trucks ($/hour) - $27.83 for all LCCAs run for this project.

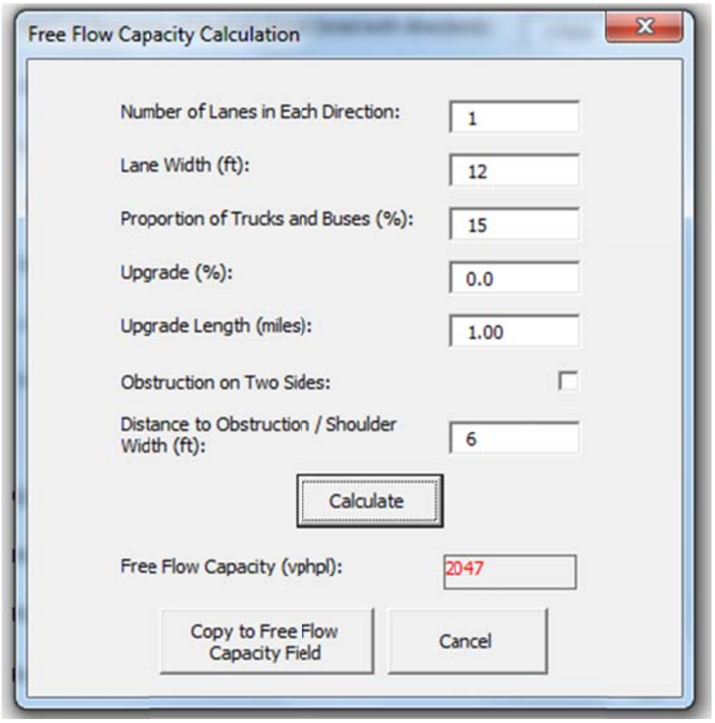


Figure 4.6 Free Flow Capacity Screen Example

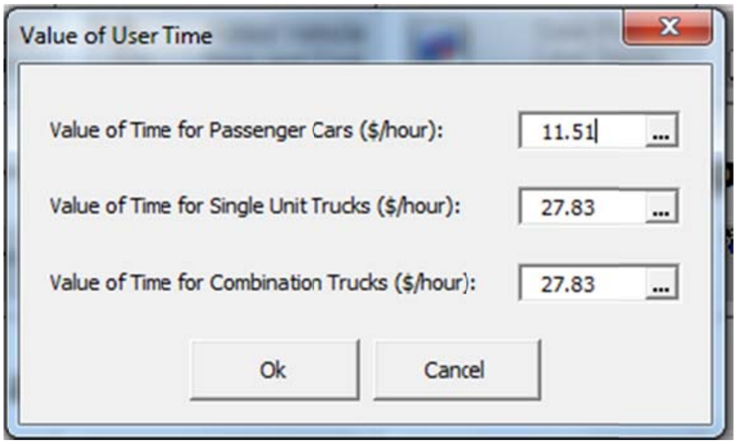


Figure 4.7 Value of User Time Screen Example

Activity 1: Construction

Alternatives. As discussed above, alternatives are the two options being compared via the *RealCost* program. For this project, that meant comparing locally sourced gravel for use in CPCD and imported limestone for use in CRCP. For each alternative, an initial construction situation was developed in order to address whether it is more economical over the long term to use local or imported aggregate. Hypothetical jobsites were imagined that would allow for at least a 200-mile transportation factor for the imported aggregate in the initial construction cost. Additionally, the research team worked with the Project Director to establish two major rehabilitations, and an annual maintenance cost was calculated for CPCD in addition to those rehabs.

Table 4.10 Hypothetical Situation Definition

	CPCD	**CRCP**
Aggregate	Gravel	Limestone
Source	Local	Imported
Width	24’	24’
Thickness	11”	9”
Length	5280’	5280’
Reinforcement	Dowel Bars and Tie Bars	Reinforcement and Tie Bars
Coarse Aggregate Content of Concrete	42%	42%

Before it is possible to calculate initial construction costs in general, some assumptions have to be made. Generally, initial construction costs are made up of three major components: labor, equipment, and materials. Initial attempts were made to gather cost data on materials from contractors, but this proved unsuccessful, as most contractors will not give out their cost information, which is confidential. Therefore, it was generally assumed that materials comprised a third of the cost of the total cost of the project. Since exact pricing was unavailable, it was decided that information would be used from the published average bid prices on the TxDOT website.

However, the research team ran into issues while going through the average bid pricing. First, there is a very minimal amount of CPCD done in Texas, which skews the pricing, as seen in Table 4.11. Nationally, it is generally cheaper to use CPCD than CRCP, but in Texas, it is notably higher:

Table 4.11 Texas Statewide Average Bid Prices (Construction) for CPCD and CRCP as of July 31, 2012

ITEM NO	DESCRIPTION	UNITS	12 MO QTY	12 MO AVG BID	12 MO Usage
360 2001	CONC PVMT (CONT REINF-CRCP)(8")	SY	423788.51	$38.60809	23
360 2002	CONC PVMT (CONT REINF-CRCP)(9")	SY	261963.16	$43.3752	13
360 2003	CONC PVMT (CONT REINF-CRCP)(10")	SY	870931.16	$38.86892	29
360 2004	CONC PVMT (CONT REINF-CRCP)(11")	SY	223455.82	$42.30707	3
360 2005	CONC PVMT (CONT REINF-CRCP)(12")	SY	393702.11	$35.25701	7
360 2006	CONC PVMT (CONT REINF-CRCP)(13")	SY	1798036	$40.13256	10
360 2008	CONC PVMT (CONT REINF-CRCP)(15")	SY	10262	$60.00	1
360 2009	CONC PVMT (JOINTED-CPCD)(8")	SY	109437	$22.67847	3
360 2011	CONC PVMT (JOINTED-CPCD)(10")	SY	1592	$55.40704	2
360 2012	CONC PVMT (JOINTED-CPCD)(11")	SY	19203	$63.00	1
360 2013	CONC PVMT (JOINTED-CPCD)(12")	SY	11547	$41.00	1
360 2014	CONC PVMT (JOINTED-CPCD)(13")	SY	1098	$95.00	1

In order to account for this, it was decided that national pricing would be used and adjusted with area multipliers. The pricing was developed using *RS Means Heavy Civil Construction Cost Data 2012*, which also provided the multipliers for the districts. Additionally, using unit pricing from RS Means allowed for the research team to account for only the differences in the CRCP and CPCD. In other words, things that are the same for both types of pavement were cancelled out, so the costs presented represent a very basic look at the cost of pavement, without factors such as ramps, bridges, and other items that raise the price of roadways considerably, but are incidental and job specific. In this way, the research team was able to compare the effect of coarse aggregate choice on the overall cost of the pavement more cleanly than if there were hundreds of factors involved.

Table 4.12 Initial Construction Costs Considered

CPCD	CRCP
Paving Equipment and Labor	Paving Equipment and Labor
Concrete (with Coarse Aggregate)	Concrete (with Coarse Aggregate)
Coarse Aggregate	Coarse Aggregate
Dowel Bars	Steel Rebar (#5, #6 rebar, including Tie Bars)
Saw Cutting	
Joint Clean and Seal	
Tie Bars	

Table 4.13 Area Multipliers for National Costs – RS Means Heavy Civil 2012

Area	Multiplier
Texarkana	68.6%
Longview	76.0%
Atlanta (avg.)	72.3%
Houston	80.3%

Calculating Initial Agency Construction Cost. Using the situation and items noted above, the area of a roadway 24' wide and 5280' long was calculated. This provided the total number of square yards in one two-lane mile: 14,080 SY. Then, the volume for 9" CRCP and 11" thick CPCD was calculated, resulting in the total number of cubic yards of pavement per mile for each type. The result was 3520 CY of CRCP, and 4302.2 CY of CPCD per two-lane mile.

Using a standard Class P mix design for concrete pavement and standard plans for the construction of concrete pavement (TxDOT standard plan sheets CPCD-94 and CRCP (1)-11), a standard concrete density of 3987 lbs/CY, and assuming the density of steel is 490 lbs/CF, the percent volume of the concrete that was coarse aggregate was determined, as was the number and volume of reinforcements for each type of concrete, and the amount of non-coarse aggregate materials.

Assuming a slab size of 12' x 15', per CPCD-94, one square two-lane mile is 352 slabs long by two slabs wide, or 704 slabs total. The dowel bars can then be calculated based on joint spacing of 15', which in one mile creates 351 transverse joints. The transverse joints go from one side of the pavement to the other, so are 24' long each. This means each transverse joint requires 22 dowel bars, or 7,722 dowel bars per mile, and requires 8,424 LF of transverse joint cutting. The longitudinal joint adds another 5280' LF of joint installation, for a total 13,704 LF of joints that need to be sawed, cleaned, and sealed per two-lane mile.

According to CPCD-94, the dowel bars that would be used in 11" CPCD are 1 3/8" x 18" dowel bars. After calculating the volume of one dowel bar (26.72808125 cubic inches), and multiplying by the total number of dowel bars in one two-lane mile, then dividing by the number of slabs (704), we find that there are roughly 11 dowel bars per slab, or 2.041728429 cubic feet per cubic yard, about 0.1% of the volume of the pavement. Subtracting the volume of the dowel bars (only about 4.3 CY per mile) out of the volume of concrete leaves a total concrete volume of 4,297.89 CY of concrete. Converting that to tons CY multiplied by the density 3987 lbs/CY) and assuming that 42% of the volume is coarse aggregate, the result is that 3,598.49 tons of coarse aggregate are needed for one two-lane mile for CPCD pavement. In order to apply this number to a wide range of job sizes, the amount of coarse aggregate was broken down further into 0.256 tons (511lbs.) of coarse aggregate per square yard.

Table 4.14 Quantities Needed for One Two-Lane Mile of CPCD

Item	Unit	Quantity
Paving Equipment and Labor	SY	14080
Dowel Bars	Ea.	7722
Saw Cutting	LF	13704
Joint Clean and Seal	LF	13704
Coarse Aggregate	Tons	3598.49
Other Concrete Materials (Less Coarse Aggregate)	Tons	8,567.84

Cost was then applied to these quantities. First, the unit price for paving, dowel bars, saw cutting, and joint cleaning and sealing was pulled from RS Means. This was necessary because these items are not broken out in TxDOT average bid pricing. The pricing for each included labor, overhead, and profit so that the truest cost could be examined. As the objective of the project is to assess the impact of coarse aggregate cost, a price for coarse aggregate per ton was calculated and adjusted for each of the districts in the case studies, discussed below. However, a price for the rest of the pavement had to be determined as well.

Table 4.15 General CPCD Cost per Mile

	Unit	QTY	Cost	Total
Concrete + Paving Equipment and Labors	SY	14080	$41.00	$577,280.00
Dowel Bars	Ea	7722	$10.10	$77,992.20
Saw Cutting	LF	13704	$4.96	$67,971.84
Joints	LF	13704	$1.97	$26,996.88
Tie Bars	Tons	5.49	$2,125.00	$11,665.86
			Construction Cost per Mile:	$761,906.78

In order to determine the price of non-coarse aggregate materials for each district, the total construction cost was calculated using national data to estimate the total cost of the job, as shown in Table 4.15. The multipliers in Table 4.5 are applied to the total in each of the case studies to establish specific pricing for the districts being evaluated.

For CRCP, TxDOT provided the percent volume of reinforcement as 0.6% steel in CRCP, which is a standard amount that is supported by Federal Highway Administration documents. The total volume of steel then was calculated as 21.12 CY, (0.6% of 3520 CY). Once subtracted from the total, the remaining concrete is 3,498.88 CY, and 42% coarse aggregate content means that 2,929.51 tons of coarse aggregate are needed for one two-lane mile of CRCP. This was then broken down further to 0.208061595 tons, or 416.12319 lbs, of coarse aggregate per square yard of CRCP. The amount of steel was estimated according the quantities called for on CRCP (1)-11.

Table 4.16 General CRCP Cost per Mile

	Unit	QTY	Cost	Total
Concrete + Paving Equipment and Labors	SY	14080	$34.50	$485,760.00
Steel (#5, #6, including Tie Bars)	Tons	193.88	$2,032.00	$393,964.16
			Construction Cost per Mile:	$879,724.16

Table 4.17 Average Calculated Construction Cost per Mile for Case Studies Adjusted with RS Means Heavy Civil 2012 Area Multiplier

	Atlanta	Houston
CPCD	$551,019.25	$611,989.57
CRCP	$636,040.57	$706,418.50

Once those average costs had been established, and since the material cost was unknown, multipliers had to be established to adjust the overall cost based solely on the coarse aggregate cost per ton so that comparisons could be made in *RealCost*. In order to do this, a workbook was developed that broke down the known costs as outlined above. The paving cost was paving only, and did not include reinforcement.

First, for both CPCD and CRCP, a material cost of $65/CY of concrete was assumed. The average price per cubic yard quoted to the research team by the quarries was $86.31/CY. However, the research team was advised by several people (who asked not to be identified) that their companies or contractors they knew were making money charging $45/CY. Since prices can vary so widely and quantities in the average bid prices may not be representative of true cost, it was decided to take the average and use $65/CY of concrete for both CRCP and CPCD. Once that was set, the average aggregate price per ton for each the Atlanta and Houston districts was examined to determine how much of that price would need to be adjusted to accommodate for coarse aggregate pricing. A consistent average from both the quarries interviewed and the ready-mix concrete providers indicated that $20/CY (or about $20/ton) was a reasonable amount to expect to spend for coarse aggregate. Based on this, it was assumed that the “other” materials in the concrete totaled $45/CY. Setting this allowed for the price per ton to be adjusted independently in the workbook, so that the total price per CY is always the price per ton plus $45/CY for the other materials.

Second, the paving labor and equipment needed to be estimated per square yard. In order to do this, a simple formula was applied:

$$\text{Paving Labor and Equipment} = \frac{\mathrm{R} - \mathrm{C}}{14080}$$

- R = The total adjusted construction cost per two-lane mile as calculated using *RS Means* (Table 18).
- C = The total number of CY of concrete (4,297.89 for CPCD, 3,498.88 for CRCP) times the average price of $65/CY.
- 14080 = The total number of square yards in one two-lane mile.

This results in the following paving equipment and labor prices per square yard:

Table 4.18 Paving Equipment and Labor Prices per Square Yard for LCCA Input Calculation

	Atlanta	Houston
CPCD	$15.30	$27.31
CRCP	$16.16	$30.33

It is important to note that these numbers are estimates based on national pricing with area multipliers, and an assumed average price per cubic yard. They may or may not be representative of actual costs, but as this information is highly guarded by contractors, it is difficult to break it out accurately. Once these numbers were calculated, along with the unit pricing (which included labor and equipment as it was included in the costs that were pulled from *RS Means Heavy Civil 2012*) and the ability to change the unit price of coarse aggregate, rough estimates of the construction cost of both types of pavement could be generated quickly.

To calculate the multipliers for the overall construction price, the research team determined the average per ton price for each of the districts and set them as 100%. To make a conservative estimate, it was assumed that there are 1900 lbs. of coarse aggregate per cubic yard of concrete, or almost one ton. Atlanta's average price per ton of coarse aggregate was $22.41, with a low of $19.35 and a high of $25.44. Therefore, the average price per ton for the Atlanta district was defined as $20/ton, with the remainder of the materials making up the $65/CY. Houston has a higher average, $29.78/ton, with a low of $16.60/ton and a high of $39.40/ton. Upon closer review, the low is a skewed number, that material is $16.60 without transportation from the quarry to the jobsite, and of the quarries surveyed who supplied the Houston district, the average distance from the district center is 94 miles. Therefore, it made sense to place the average for the Houston aggregate at $30/ton.

Using the workbook, the construction costs for the given situation of roadway were calculated for both CPCD and CRCP with aggregate pricing at $10, $15, $20, $25, $30, $35, $40, $45, and $50 per ton. For Atlanta, as determined, the standard was set at $20/ton. The costs of both CRCP and CPCD at the stated intervals were compared to the value at $20/ton to find percent difference.

Table 4.19 CPCD and CRCP Multipliers for Atlanta District

Price per Ton	$10	$15	$20	$25	$30	$35	$40	$45	$50
CPCD	-6%	-3%	0%	3%	6%	8%	11%	14%	17%
CRCP	-5%	-2%	0%	2%	5%	7%	10%	12%	14%

Once the multipliers were established, they were applied to the construction cost calculated by using *RS Means*. The same process was used to establish the multipliers for Houston. Those are given below in Table 21.

Table 4.20 CPCD and CRCP Multipliers for Houston District

Price per Ton	$10	$15	$20	$25	$30	$35	$40	$45	$50
CPCD	-9%	-6%	-4%	-2%	0%	2%	4%	6%	9%
CRCP	-9%	-7%	-4%	-2%	0%	2%	4%	7%	9%

This LCCA is to help aid in the decision to use local aggregate or imported aggregate, and it was determined that in the Atlanta and Houston districts it is not likely that aggregate pricing will be less than $20/ton. Therefore, when choosing inputs for the construction costs to run in the LCCAs for each district, the $10/ton and $15/ton were not included in the analysis. In order to best compare at what point one option outweighs the other, for all LCCAs in the Atlanta district, the CPCD construction price was set to $551,019.25, with an assumed coarse aggregate price of $20/ton. For all LCCAs in the Houston district, the initial construction price was set to $611,989.57 with an assumed coarse aggregate price of $30/ton. LCCAs were run for construction costs with the following inputs:

Table 4.21 Agency Construction Cost Inputs ($1000)

Atlanta		
$/ton	CPCD/Local	CRCP/Imported
$20.00	$551.02	$636.04
$25.00		$651.18
$30.00		$666.32
$35.00		$681.45
$40.00		$696.59
$45.00		$711.73
$50.00		$726.86
Houston		
$25.00	$611.99	$690.64
$30.00		$706.42
$35.00		$722.20
$40.00		$737.97
$45.00		$753.75
$50.00		$769.53
$100.00		$927.30

Other Activity 1 Inputs. The other inputs were determined based on various factors, discussed below.

- User Work Zone Costs – This was left as "Calculated" in the Analysis Options screen, so the user is not able to enter any input in this box.
- Work Zone Duration – This is the number of days lanes will be closed, and is assigned a value of "0" for initial construction.
- Number of Lanes Open in Each Direction During Work Zone – as this is a two-lane highway, traffic has to be able to move even when there is work going on, so there was assumed to be one lane open in each direction, whether by diversion to a frontage road or other means.
- Activity Service Life – This is the amount of time the activity is intended to survive with minimal maintenance until another activity is needed. According to the rehabilitation schedule assumed for this project, Rehabilitation One will occur 15 years after initial construction.
- Activity Structural Life – The activity service life of the first activity is the anticipated service life of the pavement. For concrete roads, this is assumed to be 50 years, and was confirmed by TxDOT as the correct value for this input.
- Maintenance Frequency – The number of years maintenance is performed. For CPCD, it is assumed that joints will need to be cleaned and sealed every 10 years. Assuming all 13704 LF of joints need to be cleaned and sealed, at $1.97/LF, that is $26,996.88 every

10 years. Spread out annually, that cost is $2,699.688 per year. In a 50-year analysis, there is the initial cut, clean and seal, which is accounted for in the initial construction cost. After this, it can be expected at year 10, 20, 30, 40, and 50, and so it makes sense to spread it out as an annual maintenance cost over the life of the pavement. In meetings, TxDOT personnel indicated only responsive patchwork for CRCP, and no scheduled maintenance, so its input value is left at "0".

- Work Zone Length (miles) - The work zone length is the length of the lane closure. This was left at 1, as the size of the projects vary across analyses.
- Work Zone Speed Limit (mph) – Typically 5-10 miles less than the posted speed limit. "65" was used as the input here, 5 mph less than the normal posted speed of 70 on most State Highways.
- Work Zone Capacity (WC) – Calculated based on a formula provided by CalTRANS

$$\mathrm{WC} = \frac{\mathrm{W} \times 100}{100 + \mathrm{P} \times (\mathrm{E} - 1)}$$

W is the base work zone capacity, or passenger cars per hour per lane, and is given as 1,100 pcphpl for two-lane highways.

  - P is the percentage of heavy vehicles, or 9%.
  - E is the passenger car equivalent, defined by CalTRANS as 1.5 for "Level" (Houston) and 2.5 for "Rolling" (Atlanta).
- Traffic Hourly Distribution – Choose Weekday 1, Weekend 1, or Weekend 2. "Weekday 1" was chosen for all LCCAs run for this project.
- Time of Day Lane Closure – "0" for initial construction.

Table 4.22 (a) Activity 1 Inputs for Both Alternatives – Atlanta District (without agency construction cost, Inputs found in Table 4.21)

	CPCD		CRCP	
Activity 1	Construction		Construction	
Agency Construction Cost ($1000)				
User Work Zone Costs ($1000)				
Work Zone Duration (days)	0		0	
No of Lanes Open in Each Direction During Work Zone	1		1	
Activity Service Life (years)	15		15	
Activity Structural Life (years)	50		50	
Maintenance Frequency (years)	1		1	
Agency Maintenance Cost ($1000)	2.69969		0	
Work Zone Length (miles)	1		1	
Work Zone Speed Limit (mph)	65		65	
Work Zone Capacity (vphpl)	969		969	
Traffic Hourly Distribution	Week Day 1		Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)				
Inbound	Start	End	Start	End
First period of lane closure	0	0	0	0
Second period of lane closure				
Third period of lane closure				
Outbound	Start	End	Start	End
First period of lane closure	0	0	0	0
Second period of lane closure				
Third period of lane closure				

Table 4.22 (b) Activity 1 Inputs for Both Alternatives – Houston District (without agency construction cost, Inputs found in Table 4.21)

	CPCD		CRCP	
Activity 1	Construction		Construction	
Agency Construction Cost ($1000)				
User Work Zone Costs ($1000)				
Work Zone Duration (days)	0		0	
No of Lanes Open in Each Direction During Work Zone	1		1	
Activity Service Life (years)	15		15	
Activity Structural Life (years)	50		50	
Maintenance Frequency (years)	1		1	
Agency Maintenance Cost ($1000)	2.69969		0	
Work Zone Length (miles)	1		1	
Work Zone Speed Limit (mph)	65		65	
Work Zone Capacity (vphpl)	1053		1053	
Traffic Hourly Distribution	Week Day 1		Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)				
Inbound	Start	End	Start	End
First period of lane closure	0	0	0	0
Second period of lane closure				
Third period of lane closure				
Outbound	Start	End	Start	End
First period of lane closure	0	0	0	0
Second period of lane closure				
Third period of lane closure				

Activity 2: Rehabilitation 1

When moving into the second Activity, Rehabilitation 1 was assumed as a full depth repair on 1.5% of the total area of the CRCP pavement, and 5% of the total area of the CPCD pavement. This data was calculated in square yards, as that is how it is bid on and listed in the average bid prices listed on TxDOT's website. Only a few of the inputs change from the first Activity:

- Agency Construction Cost (see below)
- Work Zone Duration – Work zones durations have an impact on the user cost and are estimated using CalTRANS "Productivity Estimates of Typical Future Rehabilitation for Rigid and Composite Pavement" (CalTRANS, 2010, p.59)
- Activity Structural Life – Each activity is meant to help carry the road through its original design-life, so as this activity takes place 15 years into an anticipated 50 year design-life, the input changes from "50" to "35".
-

Calculating Rehabilitation One Costs. Since the rehabilitation was defined as 1.5% and 5% of the surface area of CRCP and CPCD, respectively, simple multiplication was needed to find the total square yards being rehabilitated. There are 14,080 SY in one two-lane mile. Once the total square yards was determined, the TxDOT average bid price for Full Depth Reclamation of Concrete Pavement (Item 361) for the specific pavement thicknesses were found and applied to determine the cost for both districts to perform Rehabilitation 1. To represent Texas numbers for Rehabilitation 1 in Atlanta, the price of full depth repair of CRCP was found on the TxDOT average bid price website and is given as $291.1125/SY, as of August 10. Even though this is high, it was used as it was the most accurate number that could be found with supporting documentation. There was no price listed for full depth repair of CPCD specifically in the Atlanta district, so the Texas statewide average of $157.41/SY was used. Houston had prices listed in maintenance bid prices for full depth repair for both specified thicknesses of CRCP and CPCD. CRCP was given as $172.94/SY (which is more in line with normal pricing) and CPCD was given as $183.00/SY.

Table 4.23 Rehabilitation One Pricing

	Atlanta	Houston
CPCD	$110,816.64	$128,832.00
CRCP	$61,459.20	$36,524.93

Table 4.24 Activity 2 Inputs – Atlanta District

	CPCD		CRCP	
Activity 2	Rehabilitation 1		Rehabilitation 1	
Agency Construction Cost ($1000)	110.81664		36.64009325	
User Work Zone Costs ($1000)				
Work Zone Duration (days)	10		5	
No of Lanes Open in Each Direction During Work Zone	1		1	
Activity Service Life (years)	15		15	
Activity Structural Life (years)	35		35	
Maintenance Frequency (years)	1		1	
Agency Maintenance Cost ($1000)	2.69969		0	
Work Zone Length (miles)	1		1	
Work Zone Speed Limit (mph)	65		65	
Work Zone Capacity (vphpl)	969		969	
Traffic Hourly Distribution	Week Day 1		Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)				
Inbound	Start	End	Start	End
First period of lane closure	20	24	0	4
Second period of lane closure				
Third period of lane closure				
Outbound	Start	End	Start	End
First period of lane closure	22	24	0	6
Second period of lane closure				
Third period of lane closure				

Table 4.25 Activity 2 Inputs – Houston District

	CPCD		CRCP	
Activity 2	Rehabilitation 1		Rehabilitation 1	
Agency Construction Cost ($1000)	128.832		36.524928	
User Work Zone Costs ($1000)				
Work Zone Duration (days)	10		5	
No of Lanes Open in Each Direction During Work Zone	1		1	
Activity Service Life (years)	15		15	
Activity Structural Life (years)	35		35	
Maintenance Frequency (years)	1		1	
Agency Maintenance Cost ($1000)	2.69969		0	
Work Zone Length (miles)	1		1	
Work Zone Speed Limit (mph)	65		65	
Work Zone Capacity (vphpl)	1053		1053	
Traffic Hourly Distribution	Week Day 1		Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)				
Inbound	Start	End	Start	End
First period of lane closure	20	24	0	4
Second period of lane closure				
Third period of lane closure				
Outbound	Start	End	Start	End
First period of lane closure	22	24	0	6
Second period of lane closure				
Third period of lane closure				

Activity 3: Rehabilitation 2

The final activity is Activity 3, Rehabilitation 2. This activity was assumed to include a full depth repair identical to the one in Rehabilitation 1, as well as an asphaltic concrete (AC) overlay. The CRCP was assigned a 2-inch overlay and the CPCD was assigned a 4-inch overlay. The full depth repairs were calculated identically to the full depth repair in Rehabilitation 1. However, to calculate the pricing AC overlay, some assumptions had to be made. A Type D, PG64-22 asphalt mix was selected. The price given in the TxDOT statewide average bid prices for construction of Item 341, Series 2106, is $83.8356/ton. The asphalt density was defined as 180 lbs/cubic foot. A theoretical cubic foot of asphalt was imagined, and divided into twelve 1" squares. One cubic foot of asphalt then yields 1.3 square yards of 1" thick mat. Dividing the density per cubic foot by the number of square yards of 1" thick AC mat in a cubic foot then yields the weight per SY, at which the total tons for one two-lane mile are calculated. Once the total tons needed to place a one-inch mat on a two-lane mile were calculated, it was a matter of multiplying that price per SY by the thickness of the mat in inches (2 or 4) multiplied by the total number of square yards (14,080).

Table 4.26 Calculated Asphalt Overlay Pricing

Asphalt Price Per Ton	Asphalt - Compacted (lbs/CF)	Asphalt - SY (1" thick) per CF	Asphalt - Compacted (lbs/SY)	Tons Needed	Total for 1" mat per two-lane mile	1" AC Overlay Price (SY)
$83.84	180	1.33	135	950.4	$79,677.42	$5.66

However, when we presented this price to other members of the research group, it was widely stated that $5.66/inch/SY for an AC overlay was high, and that $2.50 was more in line with the pricing currently being utilized in the industry. Therefore, $2.50/inch/SY was used to calculate the input, but it is worth noting the price discrepancy between what was calculated based on statewide asphalt pricing and what is being seen in the field.

The totals for the full depth repair and the asphalt overlay are added together to estimate the cost of Rehabilitation 2.

Table 4.27 Rehabilitation Two Pricing

	Atlanta	Houston
CPCD	$251,616.64	$269,632.00
CRCP	$107,040.0932	$106,924.938

Table 4.28 Activity 3 Inputs – Atlanta District

	CPCD		CRCP	
Activity 3	Rehabilitation 2		Rehabilitation 2	
Agency Construction Cost ($1000)	251.61664		107.0400932	
User Work Zone Costs ($1000)				
Work Zone Duration (days)	15		10	
No of Lanes Open in Each Direction During Work Zone	1		1	
Activity Service Life (years)	15		15	
Activity Structural Life (years)	20		20	
Maintenance Frequency (years)	1		1	
Agency Maintenance Cost ($1000)	2.69969		0	
Work Zone Length (miles)	1		1	
Work Zone Speed Limit (mph)	65		65	
Work Zone Capacity (vphpl)	969		969	
Traffic Hourly Distribution	Week Day 1		Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)				
Inbound	Start	End	Start	End
First period of lane closure	20	24	0	4
Second period of lane closure				
Third period of lane closure				
Outbound	Start	End	Start	End
First period of lane closure	22	24	0	6
Second period of lane closure				
Third period of lane closure				

Table 4.29 Activity 3 Inputs – Houston District

	CPCD		CRCP	
Activity 3	Rehabilitation 2		Rehabilitation 2	
Agency Construction Cost ($1000)	269.632		106.924928	
User Work Zone Costs ($1000)				
Work Zone Duration (days)	15		10	
No of Lanes Open in Each Direction During Work Zone	1		1	
Activity Service Life (years)	15		15	
Activity Structural Life (years)	20		20	
Maintenance Frequency (years)	1		1	
Agency Maintenance Cost ($1000)	2.69969		0	
Work Zone Length (miles)	1		1	
Work Zone Speed Limit (mph)	65		65	
Work Zone Capacity (vphpl)	1053		1053	
Traffic Hourly Distribution	Week Day 1		Week Day 1	
Time of Day of Lane Closures (use whole numbers based on a 24-hour clock)				
Inbound	Start	End	Start	End
First period of lane closure	20	24	0	4
Second period of lane closure				
Third period of lane closure				
Outbound	Start	End	Start	End
First period of lane closure	22	24	0	6
Second period of lane closure				
Third period of lane closure				

Results

The cost of the coarse aggregate per ton did not have a large effect on the overall initial construction cost of the pavement. CRCP costs more initially because of the large amount of steel that must be placed into it, but that is fairly balanced by the low associated maintenance costs. The overall results, based on agency cost over the 50-year analysis period, are presented in Table 30.

Table 4.30 LCCA Results

Winning Alternative Atlanta – Average at $20/ton	Price per Ton of Coarse Aggregate	Winning Alternative Houston – Average at $30/ton
CRCP	$20	CRCP
CRCP	$25	CRCP
CRCP	$30	CRCP
CRCP	$35	CRCP
CPCD	$40	CRCP
CPCD	$45	CRCP
CPCD	$50	CPCD

Atlanta

As shown in Table 30, CRCP has the best long-term value for Atlanta up to between $35-40/ton when compared to local aggregate at $20/ton. In terms of initial construction price, CPCD is the less expensive option, but the LCCA takes into account 50 years' worth of maintenance, so this is likely due to the high maintenance prices associated with CPCD. There were issues in finding good maintenance numbers for concrete pavement for Atlanta, so it is likely that the price range for imported aggregates could change if those numbers are updated to reflect a more realistic pricing schedule.

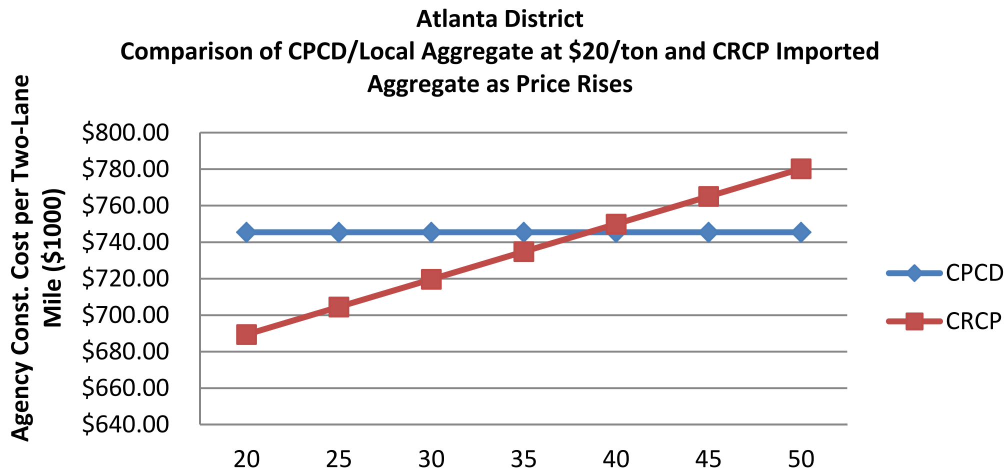


Figure 4.8 Agency Construction Cost by Present Value Result – Atlanta District

Houston

As shown in Table 30, CRCP is the best long-term value up until imported aggregate is between $45-50/ton. Houston's quarries are mostly located away from the center of the district, so the local aggregate, which is more expensive at $30/ton on average, is initially closer to the imported price.

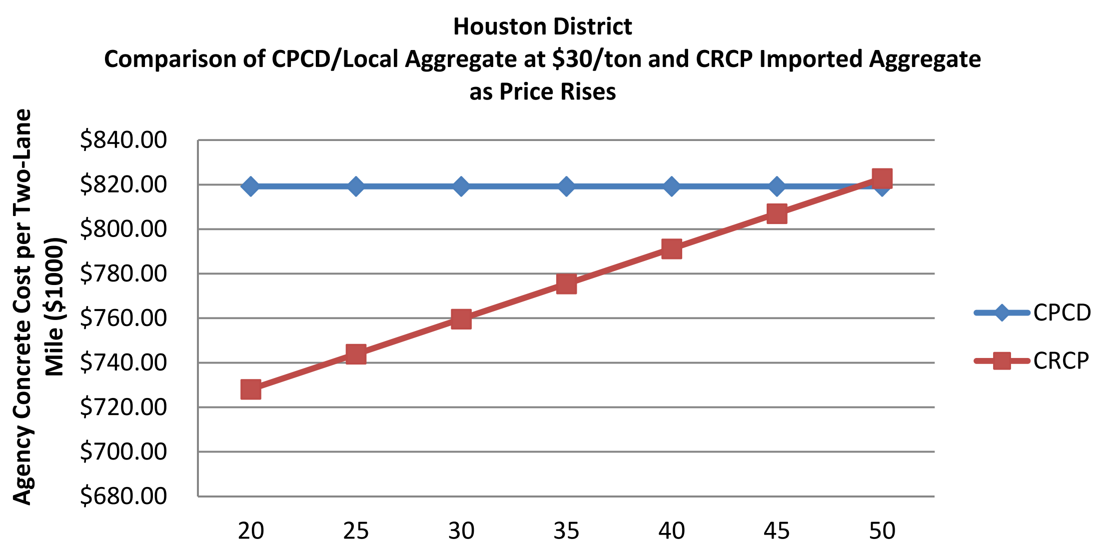


Figure 4.9 Agency Construction Cost by Present Value Result – Houston District

Based on the maintenance pricing and formulas run by RealCost, CRCP is a better option, even if it is more expensive per ton, to a certain point. At that point, the price of construction outweighs any maintenance benefit gained. This point is specific to individual areas, and would need to be found for each district to determine what that cost per ton break-even point is.

Chapter 5 Specific Requirements for CPCD with High CoTE

5.1 Introduction

As described earlier, the basic premise of CPCD design concept is that concrete volume changes due to temperature and moisture variations will be fully accommodated, which is accomplished by the use of dowels and plastic sheeting between concrete and base layers. Concrete with a high CoTE will experience larger volume changes compared with concrete with a low CoTE, in the form of greater warping and curling as well as larger joint movements.

Large warping and curling could result in a higher probability of transverse cracking. The best way to counter the higher probability of transverse cracking would be the use of smaller transverse joint spacing. TxDOT has used 15-ft joint spacing since the 1944, and the TxDOT PMIS shows that mid-slab cracking is quite rare in Texas. 15-ft joint spacing is the smallest value used in the nation. Based on the performance in Texas, as far as mid-slab cracking is concerned, it appears that 15-ft joint spacing is adequate for concrete with high CoTE.

Design for joint geometry is based on field observations and experience, not solely on mechanistic analysis. The objective of geometric design of joint is to provide an optimum performance of joint sealant, which also depends on the material properties of sealants. Currently, two types of sealants are used – silicone based materials and hot-pour asphalt materials.

Another issue with high CoTE concrete in CPCD is the joint saw cut timing and depth.

Before the issues related to high CoTE concrete in CPCD are discussed, general discussions on the distresses in CPCD in Texas are made.

5.2 CPCD Distresses in Texas

Mid-slab transverse cracking or joint faulting is quite rare in Texas. The reason for rare incidents of mid-slab cracking or faulting is that TxDOT design standards as early as 1944 required transverse joint spacing of 15 ft and the use of dowels at transverse joints. There have been some exceptions, where design engineers tried something deviant from TxDOT CPCD standards. For example, Figure 5.1 shows severe faulting in CPCD. In this project, dowels were not used. Instead, slab thickness was increased by using a larger value for load transfer coefficient (J) in the 1993 AASHTO Pavement Design Guide. This distress could have been prevented by the use of dowels. This example illustrates the fact that slab thickness does not necessarily make up for deficiencies in other design elements. A CPCD section on US 75 in the Paris District showed some faulting at the longitudinal construction joint between the outside lane and retrofitted outside shoulder. However, faulting at TCJs was almost negligible, even though a 10-in concrete slab was placed directly on subgrade and truck traffic has been quite heavy. Figure 5.2 illustrates transverse cracks observed in CPCD projects in Texas. If slab thickness was deficient or saw-cutting was delayed, transverse cracks are expected to develop in the middle area of the slab between transverse contraction joints (TCJs). In Figure 5.2, transverse cracks are rather close to a TCJ. These cracks are due to interactions through tie bars in the displacements of slabs placed at different times. This type of crack can be prevented by improved design standards and quality construction practices.

Figure 5.1 Faulting in Transverse Joint

Figure 5.2 Transverse Cracks Near Joint

Longitudinal cracks in CPCD are not frequent. The primary cause for longitudinal cracks in CPCD in Texas is the volume changes in the base and/or subgrade. Figure 5.3 shows longitudinal cracks in CPCD. When volume changes are excessive in the base and/or subgrade, longitudinal cracks occur in CRCP as well, as shown in Figure 5.4. Increasing slab thickness or modifications in joint layouts will not prevent longitudinal cracking. Reducing volume change potential in the base and/or subgrade is the best way to minimize longitudinal cracking potential in both CPCD and CRCP.

Figure 5.3 Longitudinal Cracking in CPCD **Figure 5.4 Longitudinal Cracking in CRCP**

The primary distress type in CPCD in Texas is failure near TCJs. Figure 5.5 illustrates typical distress at TCJ, and Figure 5.6 shows the repairs of distresses at TCJ. Distresses and repairs at TCJs negatively impact pavement condition score, and are one of the primary reasons for lower condition score of CPCD than that of CRCP or ACP.

Figure 5.5 Distress at TCJ **Figure 5.6 Repair of Distress at TCJ**

The mechanisms of the distress at TCJs in Texas appear to be quite different from those in other states, especially in northern states. One national study that investigated extensively the joint performance of CPCD (Taylor et al 2011) concluded that, even though not all the causes of joint deterioration are known, the primary mechanism of joint distress appeared to be freeze-thaw and a deficient air void system. In Texas, freeze-thaw is rare, and de-icing salt is rarely applied.

Accordingly, joint deterioration associated with freeze-thaw, the application of de-icing chemicals, and deficient air void system in concrete are not considered the cause for joint deterioration as shown in Figure 5.5.

Even though detailed investigations were not made in this study to identify the causes of the joint distress as shown in Figure 5.5, it appears that two potential causes exist. One is the saw cutting that could have caused micro-damages and the other might be related to dowel bar placement. Concrete with a high CoTE normally contain aggregates that are quite hard. Saw cutting at early ages when paste is still developing strength and is not as strong as coarse aggregates, could cause micro-damage to paste surrounding coarse aggregates that are under being cut. Saw cutting could dislodge the aggregates because aggregates are quite hard. Figure 5.7 shows the evidence of micro-damage near the transverse saw cut, and resulting spalling at the joint (Figure 5.8). In this project, a CRCP section on US 290 in the Houston District, an early-entry saw cut method was applied to induce a transverse crack at this location. The saw cut was applied at 4 to 5 hours after concrete placement. The coarse aggregate used was siliceous river gravel, which is quite hard. It took more than 10 years before the micro-damage developed into spalling. Since coarse aggregates with a high CoTE are usually siliceous aggregates, which are quite hard and durable, saw cutting timing should be adjusted not to cause micro-damage to paste surrounding coarse aggregate during the saw cut. Considering the short joint spacing used in Texas and the evidence of few mid-slab cracking in Texas due to environmental loading (temperature and moisture variations), saw cutting should be delayed until adequate concrete strength is achieved. Another reason for damage to the concrete at the joints could be wobbly action of the saw blades. This is an equipment issue, and adequate inspection of the sawing machine should address this issue.

Figure 5.7 Micro-Damage Due to Early Entry Saw Cut

Figure 5.8 Spalling at Saw Cut Joint Due to Micro-Damage in Concrete

The other cause for the distress shown in Figure 5.5 could be dowel misalignment. A number of studies were conducted over the years on this issue. However, disagreements exist regarding whether dowel misalignment actually causes distresses in CPCD. Some state DOTs, such as Caltrans, do not believe dowel misalignments cause distresses in CPCD, and do not have tolerances for dowel alignments. On the other hand, most state DOTs believe the importance of proper dowel alignment and have tolerances in their specifications. At this point, TxDOT Item 360 states “Tolerances for location and alignment of dowels will be shown on the plans.” However, the dowel bar tolerances are not included in the current CPCD design standards.

Sealing is another issue that state DOTs have different opinions on. Some states, such as Minnesota and Wisconsin DOTs, do not seal TCJs for certain highways, whereas most state DOTs make sealing of TCJs their standard practice. The reason Minnesota and Wisconsin DOT do not seal TCJs is that water will get into the joints once the sealants become aged, and sealants will keep the water in the joints longer, compared with when joints are not sealed, increasing the potential for moisture and freeze-thaw damage. Also, one of the objectives of the sealing – keeping incompressible materials out of the joints – is achieved by high speed traffic, which causes negative pressure at the joints when vehicles pass at a high speed and brings the incompressible materials out of the joints. In Texas, freeze-thaw damage is rare, and there is little evidence that sealing joints negatively affects CPCD performance. Currently, national efforts are under way to find the best practice on sealing. It is recommended that TxDOT keep the current requirements of sealing TCJs until positive findings are made from the national effort.

Major characteristics of concrete containing high CoTE coarse aggregate – larger volume changes due to temperature variations and more heterogeneous nature of concrete, coarse aggregates are harder than paste – might have technical implications on the use of this material in CPCD. These include the following items:

1) Joint spacing
2) Joint saw cut depth
3) Joint saw cut timing
4) Joint width
5) Thickness design
6) Tolerances on dowel bar alignment
7) Joint sealant reservoir design

Among these, only joint saw cut timing may be affected by the use of a high CoTE coarse aggregate. Current joint spacing of 15 ft appears to be working well in Texas, as evidenced by the low rate of mid-slab cracking in CPCD due to environmental loading. The current requirement of joint saw cut depth of 1/3 of the slab thickness has worked well in both CPCD and CRCP, and there is no need to change this requirement. The only issue with the use of high CoTE aggregates is that some sawing operators do not want to cut as deeply as required since hard rocks in high CoTE concrete will abrade saw blades more. This is an issue of specification requirement enforcement. Some states use 1/8-in single cut joint, and the performance communications with Minnesota and Wisconsin DOT engineers reveal that the performance has been satisfactory. TxDOT might implement the single cut design as a trial basis. Even though at least one mechanistic pavement design method requires the use of thicker slabs for concrete with a high CoTE, it is believed that insufficient field evidence exists that supports the validity of the approach. It is recommended that no changes are made to the current slab thickness requirements for CoTE. Even though slab displacements will be a little bit larger at joints due to the use of concrete with higher CoTE compared with concrete with lower CoTE, the effects on tolerances of dowel bar alignment and joint reservoir design will be minimal. It is recommended that typical tolerance values of dowel bar alignment used by most state DOTs be adopted by TxDOT, and joint sealant reservoir designs in the current standards, JS-94, be kept.

5.3 Saw Cutting of Transverse Contraction Joint

Determining optimum saw cut timing during CPCD construction is a difficult task. TxDOT Item 360 used to require saw cutting within 12 hours after concrete finishing. This requirement was not enforced vigorously. As a result, saw cut operations were left up to contractors. In 2004 specifications, TxDOT abandoned the "12 hour" requirement, and instead included the following wording:

"Saw joints to the depth shown on the plans as soon as sawing can be accomplished without damage to the pavement regardless of time of day or weather conditions. Some minor raveling of the saw cut is acceptable."

This requirement might be appropriate for concrete with soft coarse aggregates. However, as discussed previously, when hard coarse aggregates are used in concrete, sawing too early might result in micro-damage to concrete and spalling in the long run. It is recommended that this wording is revised for sawing of TCJs.

Saw cut timing is a difficult issue in both stipulating the optimum time in the specifications and enforcing the requirement during construction. It is primarily due to the number of variables that affect concrete strength development and the difficulty of estimating concrete strength or maturity in the field. Identifying a simple and practical method that can be used in the field for the determination of the optimum saw cut timing was one of the objectives of this study. To this end, a slab was cast with high CoTE concrete and saw cuts were made at different times from two hours to 24 hours after concrete placement. During this period, concrete maturity was measured, with numerous evaluations of concrete modulus of elasticity at various times. Meanwhile, a Schmidt hammer was used to estimate concrete strength. It was considered that the Schmidt hammer method is quite simple to use in the field, compared with the maturity method or other methods. However, the Schmidt hammer that was used in this project was not adequate to estimate early-age concrete strength. No good method was developed in this project that can guide the contractor to determining the optimum time for sawing. It is recommended that the responsibility of determining optimum saw-cut timing be left to the contractors. Special provision developed in this study includes the wording that is most commonly used nationwide.

5.4 Dowel Alignment

As discussed previously, the current TxDOT Item 360 states the tolerances of dowel bar alignments, but the requirements are not in the standards. Newly developed design standards in this project include tolerances for the dowel bar alignment. These values were selected from the requirements specified by most state DOTs.

5.5 Summary

It appears that the use of high CoTE concrete in CPCD will not cause more damage and distresses compared with CPCD to low CoTE concrete. Typical CPCD distresses observed in Texas – longitudinal cracking and distresses at transverse contraction joints – are not necessarily related to high CoTE of concrete. Good concrete practice with good specifications and design standards will ensure the good performance of CPCD. To that end, a special provision to Item 360 was developed and included in Appendix B of this report. Current CPCD design standards were revised and are included in Appendix C.

Chapter 6 Conclusions and Recommendations

Design concept and structural responses of CPCD and CRCP are quite different. In CPCD, concrete volume changes are allowed to a full extent, and accommodations are made to ensure good load transfer at discontinuities, i.e., transverse contraction joints. On the other hand, concrete volume changes are restrained to a significant degree in CRCP by longitudinal reinforcement and base friction. Because of this vastly different behavior between the two pavement types, concrete with a high CoTE is not an ideal material for CRCP. In other words, the performance of CRCP with a high CoTE concrete will be compromised, with resulting spalling distresses. Concrete with a high CoTE should not be used in CRCP; instead, it should be used for CPCD if at all possible.

This study investigated the correlation between spalling and delamination distresses and concrete material properties. CRCP sections with severe spalling and delaminations were identified. Sections with no spalling and delamination distresses were also identified. A minimum of two cores were taken from those sections and CoTE and modulus of elasticity were evaluated. There was an excellent correlation. Concrete material properties of selected coarse aggregates were extensively evaluated in the laboratory. Also, in-depth analysis was made of the life-cycle cost of the pavement with coarse aggregates from different sources.

The findings from this effort can be summarized as follows:

1. Excellent correlation was observed between functional distresses in CRCP (severe spalling) and the CoTE of concrete. CRCP sections where concrete had a CoTE above 5.5 microstrain per ºF exhibited severe spalling. On the other hand, CRCP sections with a CoTE less than 5.5 microstrain per ºF did not show functional distresses.
2. Extensive laboratory evaluations of concrete with various coarse aggregate types revealed the following:

    a. All ten aggregate sources qualified according to Item 421 requirements of Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges.
    b. Three sources failed to meet MTO's unconfined freezing and thawing requirement, and one source did not meet MTO's MD requirement. When combining unconfined freezing and thawing results and MD results, four sources did not qualify.
    c. All the concrete mixes satisfied the class P concrete strength requirements according to item 360.
    d. River gravel showed the highest 28-day modulus of elasticity. Aggregate with higher absorption showed lower 28-day modulus of elasticity.
    e. Slate showed the highest CoTE and igneous rock had the lowest CoTE. River gravel showed higher CoTE than the river gravel and limestone blend, justifying the potential of reducing concrete CoTE by blending low CoTE aggregate with high CoTE aggregate.

3. The cost of the coarse aggregate per ton does not have a large effect on the overall initial construction cost of the pavement.

4. CRCP costs more initially because of the large amount of steel that must be placed into it, but that is fairly balanced by the low associated maintenance costs.

5. Two case studies were made using life-cycle cost analysis – one for the Atlanta District and the other for the Houston District. The findings are as follows:

    a. For the Atlanta District, CRCP has the best long-term value up to between $35-40/ton when compared to local aggregate at $20/ton. In terms of initial construction price, CPCD is the less expensive option, but the LCCA takes into account 50 years' worth of maintenance, so this is likely due to the high maintenance prices associated with CPCD. There were issues in finding good maintenance numbers for concrete pavement for Atlanta, so it is likely that the price range for imported aggregates could change if those numbers are updated to reflect a more realistic pricing schedule.
    b. For the Houston District, CRCP is the best long-term value until imported aggregate reaches a price of between $45-50/ton. Houston's quarries are mostly located away from the center of the district, so the local aggregate, which is more expensive at $30/ton on average, is initially closer to the imported price. Based on the maintenance pricing and formulas run by RealCost, CRCP is a better option, even if it is more expensive per ton, to a certain point. At that point, the price of construction outweighs any maintenance benefit gained. This point is specific to individual areas, and would need to be found for each district to determine what that cost per ton break-even point is.

The findings from this study indicate that, if concrete with a CoTE greater than 5.5 microstrain/°F is used in CRCP, the potential for severe spalling increases substantially. Accordingly, if the only coarse aggregate type available locally produces concrete with a CoTE greater than 5.5 microstrain/°F, it is strongly recommended that this aggregate is not used in CRCP. Instead, the use of CPCD should be considered. Whether low CoTE aggregates need to be brought in for use in CRCP, or locally available high CoTE aggregates will be utilized in CPCD, should be based on the local experience with CPCD performance.

References

1. ASTM C 192. (2007). “Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory.” ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.
2. ASTM Standard C 215-08 (2008). "Standard Test Method for Fundamental Transverse, Longitudinal, and Torsional Frequencies of Concrete Specimens." *ASTM International*West Conshohocken, PA,.
3. ASTM C469. (2010). “Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression.” ASTM International, 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959, United States.
4. Choi, S., Ha, S., and Won, M. C. (2011). “Horizontal cracking of continuously reinforced concrete pavement under environmental loadings.” *Construction and Building Materials*, 25, 4250–4262.
5. CSA A23.2-24A. (2004). “Test method for the resistance of unconfined coarse aggregate to freezing and thawing.” Canadian Standards Association, Toronto, Canada.
6. Rogers, C. A. (1998). “Canadian Experience with the Micro-Deval Test for Aggregates.” *Geological Society, London, Engineering Geology Special Publications*, 13(1), 139–147.
7. Rogers, C. A., Lane, B. C., and Senior, S. A. (2003). “The Micro-Deval Abrasion Test for Coarse and Fine Aggregate in Asphalt Pavement.” Austin, Texas.
8. Tex-1xx-E.(unpublished).“Aggregate Crushing Value.” Texas Department of Transportation, Austin, Texas.
9. Tex-403-A. (1999). “Saturated Surface-Dry Specific Gravity and Absorption of Aggregates.” Texas Department of Transportation, Austin, Texas.
10. Tex-410-A. (1999). “Abrasion of Coarse Aggregate Using the Los Angeles Machine.” Texas Department of Transportation, Austin, Texas.
11. Tex-411-A. (2004). “Soundness of Aggregate using Sodium Sulfate or Magnesium Sulfate.” Texas Department of Transportation, Austin, Texas.
12. Tex-418-A. (2008). “Compressive Strength of Cylindrical Concrete Specimens.” Texas Department of Transportation, Austin, Texas.
13. Tex-428-A. (2011). “Determining the Coefficient of Thermal Expansion of Concrete.” Texas Department of Transportation, Austin, Texas.
14. Zollinger, D.G., and Barenberg, E.J. (1990). "Continuously reinforced pavements: punchouts and other distresses and implications for design." Research Report No. FHWA/IL/UI 227, Illinois Department of Transportation, Department of Civil Engineering, Engineering Experiment Station, University of Illinois, Urbana, Illinois

15. Gree, W.J, Carrasquillo, R.L., McCullough, B.F., and Saraf, C. L. (1987) "Coarse Aggregate for PCC - Pilot Study Evaluation." Research Report FHWA/TX-88+422-1, Center for Transportation Research, The University of Texas at Austin, Austin, Texas

16. Epps, J.A., Meyer, A.H., Larrimore, I.E., and Jones, H.L. (1974). "Roadway maintenance evaluation user's manual." Research Report 151-2, Texas Highway Department, Texas Transportation Institute, College Station, Texas.

17. Texas State Department of Highways and Public Transportation. (1985). Rater's Manual, Austin, Texas.

18. SHRP (1994). "Sensitivity analysis for selected pavement distresses." Research Report, Strategic Highway Research Program, National Research Council, Washington, D.C.

19. NCHRP (1979). "Failure and repair of continuously reinforced concrete pavement." NCHRP Synthesis 60, National Cooperative Highway Research Program, National Research Council, Washington, D.C.

20. Gutierriz de Velasco, M., and McCullough, B.F. (1981). "Summary report for 1978 CRCP condition survey in Texas." Research Report 177-20, Center for Transportation Research, University of Texas at Austin, Austin, Texas.

21. McCullough, B.F., Ma, J.C.M., and Noble, C.S. (1979). "Limiting criteria for the design of CRCP." Research Report 177-17, Center for Transportation Research, The University of Texas at Austin, Austin, Texas.

22. Shelby, M.D., and McCullough, B.F. (1960). "Experience in Texas with continuously reinforced concrete pavement." Highway Research Board Bulletin 274, Highway Research Board, National Research Council, Washington, D.C., 1-29.

23. McCullough, B.F., Abou-Ayyash, A., Hudson, W.R., and Randall, J.P. (1975). "Design of continuously reinforced concrete pavements for highways." NCHRP Report 1-15, Center for Highway Research, The University of Texas at Austin, Austin, Texas.

24. Tayabji, S.D., and Colley, B.E. (1986). "Improved rigid pavement joints." Research Report FHWA/RD-86/040, Construction Technology Laboratories, A Division of Portland Cement Association, Skokie, Illinois.

25. Wright, C.C. (1981). "Experimental continuously reinforced Portland cement concrete pavement - Interstate 95, Cumberland, Maine, final report digest." Technical Paper 81-4, Materials and Research Division, Maine Department of Transportation, Portland, Maine.

26. Tracy, R.G. (1978). "Deterioration of continuously reinforced concrete pavement in Minnesota - An overview." Research Report, Research and Development Section, Minnesota department of Transportation, Minneapolis, Minnesota.

27. Smith, K.D., Mueller, A.L., Darter, M.I., and Peshkin, D.G. (1990). "Performance of jointed concrete pavements volume II - evaluation and modification of concrete pavement design and analysis models." Research Report FHWA-Rd-89-137, ERES Consultants, Savoy, Illinois.

28. Van Wijk, A.J. (1985). "Purdue economic analysis of rehabilitation and design alternatives for rigid pavements: A user's manual for PEARDARP." Final Report, FHWA Contract Number DTFH61-82-C-00035, Federal Highway Administration, Washington, D.C.

29. Kopperman, S., Tiller, G., and Tseng, M. (1986). "Interactive microcomputer version, user's manual: IBM-PC compatible version." Research Report FHWA/TS-87/205, Federal Highway Administration, Washington, D.C.

30. Dossey, T., and Hudson, W.R. (1993). "Distress curves for CRCP crack spalling." Technical Memorandum No. 1908-12, Center for Transportation Research, The University of Texas at Austin, Austin, Texas.

31. Dossey, T., and Weissmann, A. (1989). "A continuously reinforced concrete database." Research Report 472-6, Center for Transportation Research, The University of Texas at Austin, Austin, Texas.

32. Taylor, P., Rasmussen, R., Torres, H., Fick, G., Harrington, D., and Cackler, T. (2011) "Interim Guide for Optimum Joint Performance of Concrete Pavements" National Concrete Pavement Technology Center, Iowa State University, Iowa

Appendix A: Guidelines for Optimum Rigid Pavement Type Selection

Currently, two types of rigid pavement are used in TxDOT. One is jointed plain concrete pavement (CPCD) and the other is continuously reinforced concrete pavement (CRCP). Even though both pavement types use the same materials on the surface layer and have similar pavement structures, the behavior and structural responses of the two pavement types are vastly different. In short, concrete volume changes in CPCD due to temperature and moisture variations are allowed and provisions made to ensure good load transfers at discontinuities (transverse contraction joints). On the other hand, volume changes in CRCP are severely restrained by longitudinal reinforcement and base friction. Because of this difference in pavement behavior, concrete with high volume change potential, i.e., concrete with a high coefficient of thermal expansion (CoTE) is not suitable for CRCP. There is a compatibility issue between rigid pavement type and Portland cement concrete (PCC) material properties. Ignoring this compatibility issue would result in less than optimum rigid pavement type.

The TxDOT Administrative Circular developed in 2000 practically discouraged the use of CPCD, except for special situations where CPCD is more suitable, such as intersections. Since then, most of the rigid pavement built at TxDOT was CRCP. Even though CRCP performance in Texas has been quite satisfactory, distresses in the form of severe spalling and delaminations were observed in CRCP with high CoTE concrete. Repairs of CRCP distresses are difficult, time-consuming, and expensive, and their performance has not always been good. When selecting a rigid pavement type, it is advisable to consider this “compatibility” issue between PCC material properties – CoTE – and rigid pavement type.

Since the coarse aggregate occupies about 40 percent of concrete volume, and CoTE of mortar is almost constant, coarse aggregate type has the most significant effect on the CoTE of concrete. In Texas, different coarse aggregate types are produced at various locations. Extensive CoTE testing conducted at TxDOT-CSTMP reveals that a large variability exists in CoTE among coarse aggregates produced at various locations. From a purely technical standpoint without economic considerations, it would be easy to select an optimum rigid pavement type for a coarse aggregate type available locally. In other words, where the only locally available coarse aggregates have high CoTE, CPCD should be used. However, the findings from this research study indicate that the cost of coarse aggregate in relation to the total cost of paving projects is quite small, regardless of whether locally available coarse aggregate is used or it is imported from quarries away from the project. Accordingly, when considering life-cycle cost including repair and maintenance cost, the selection of an optimum rigid pavement type primarily depends on the performance of each pavement type.

It is not an easy task to compare the performance of CPCD and CRCP, because identifying CRCP and CPCD sections with comparable traffic and environmental conditions is difficult. In addition, the same pavement structures were not used in CPCD and CRCP in Texas. For example, many miles of CPCD were built without a stabilized base layer, whereas most of the CRCP in Texas were built with a stabilized base. Traditionally, an adequate amount of longitudinal steel was used in CRCP in Texas, providing excellent load transfer at transverse cracks, while dowels were not used in some CPCD sections where traffic volume was high even though TxDOT standards required the use of dowels. This resulted in faulting at transverse joints and low ride scores. Limited field data available show that the performance of CPCD could be comparable to the performance of CRCP, if the same base structure is used for both pavement types and design features known to improve pavement performance are properly provided.

The findings from this study indicate that, if concrete with a CoTE greater than 5.5 microstrain/°F is used in CRCP, the potential for severe spalling increases substantially. Accordingly, if the only coarse aggregate type available locally produces concrete with a CoTE greater than 5.5 microstrain/°F, it is strongly recommended that this aggregate is not used in CRCP. Instead, the use of CPCD should be considered. The decision of whether low CoTE aggregates need to be brought in for use in CRCP, or locally available high CoTE aggregates will be utilized in CPCD should be based on the local experience with CPCD performance.

Another factor to be considered for the selection of a rigid pavement type is the geometric nature of the pavement. If there are a number of leaveouts and intersections, such as frontage roads in urban or metropolitan areas, CPCD is the more reasonable option, since it is easier to build in those areas and CPCD looks better than CRCP for pedestrians and drivers in the slow moving vehicles.

If a rigid pavement type is to be selected by TxDOT, the program developed in this study could be utilized with appropriate input values. On the other hand, if a rigid pavement type selection is left to contractors, chances are that they will select the rigid pavement type with the lower initial construction cost, using local aggregates, regardless of CoTE values. TxDOT may develop policies that discourage the use of CRCP with a coarse aggregate that produce concrete of CoTE greater than 5.5 microstrain/°F.

Appendix B: Special Provision to Item 360

2004 Specifications

SPECIAL PROVISION
360---0xx
Concrete Pavement

For this project, Item 360, "5.5 Summary," 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 360.4. Construction, Section C. Reinforcing Steel and Joint Assemblies is voided and replaced by the following:

a. **Continuously Reinforced Concrete Pavement (CRCP).** Accurately place and secure in position all reinforcing steel as shown on the plans. Tolerances for the depths of both longitudinal and transverse reinforcement will be shown on the plans. Stagger the longitudinal reinforcement splices to avoid having more than 1/3 of the splices within 2-ft. longitudinal length of each lane of the pavement. Place tie bars or drill and epoxy grout tie bars at longitudinal construction joints as shown on the plans. Verify that tie bars that are drilled and epoxied into concrete at longitudinal construction joints develop a pullout resistance equal to a minimum of ¾ of the yield strength of the steel after epoxy manufacturer's recommended curing time. Test 15 bars using ASTM E 488, except that alternate approved equipment may be used. All 15 tested bars must meet the required pullout strength. If any of the test results do not meet the required minimum pullout strength, perform corrective measures to provide equivalent pullout resistance. Secure reinforcing bars at alternate intersections with wire ties or locking support chairs. Tie all splices with wire.

b. **Concrete Pavement Contraction Design (CPCD).** Place dowels at mid-depth of the pavement slab, parallel to the surface. Place dowels for transverse contraction joints parallel to the pavement edge. Tolerances for location and alignment of dowels will be shown on the plans. Place tie bars or drill and epoxy grout tie bars at longitudinal construction joints as shown on the plans. Verify that tie bars that are drilled and epoxied into concrete at longitudinal construction joints develop a pullout resistance equal to a minimum of ¾ of the yield strength of the steel after epoxy manufacturer's recommended curing time. Test 15 bars using ASTM E 488, except that alternate approved equipment may be used. All 15 tested bars must meet the required pullout strength. If any of the test results do not meet the required minimum pullout strength, perform corrective measures to provide equivalent pullout resistance.

Article 360.4. Construction, Section D. Joints, 2. Transverse Construction Joints, b. Concrete Pavement Contraction Design (CPCD) is voided and replaced by the following:

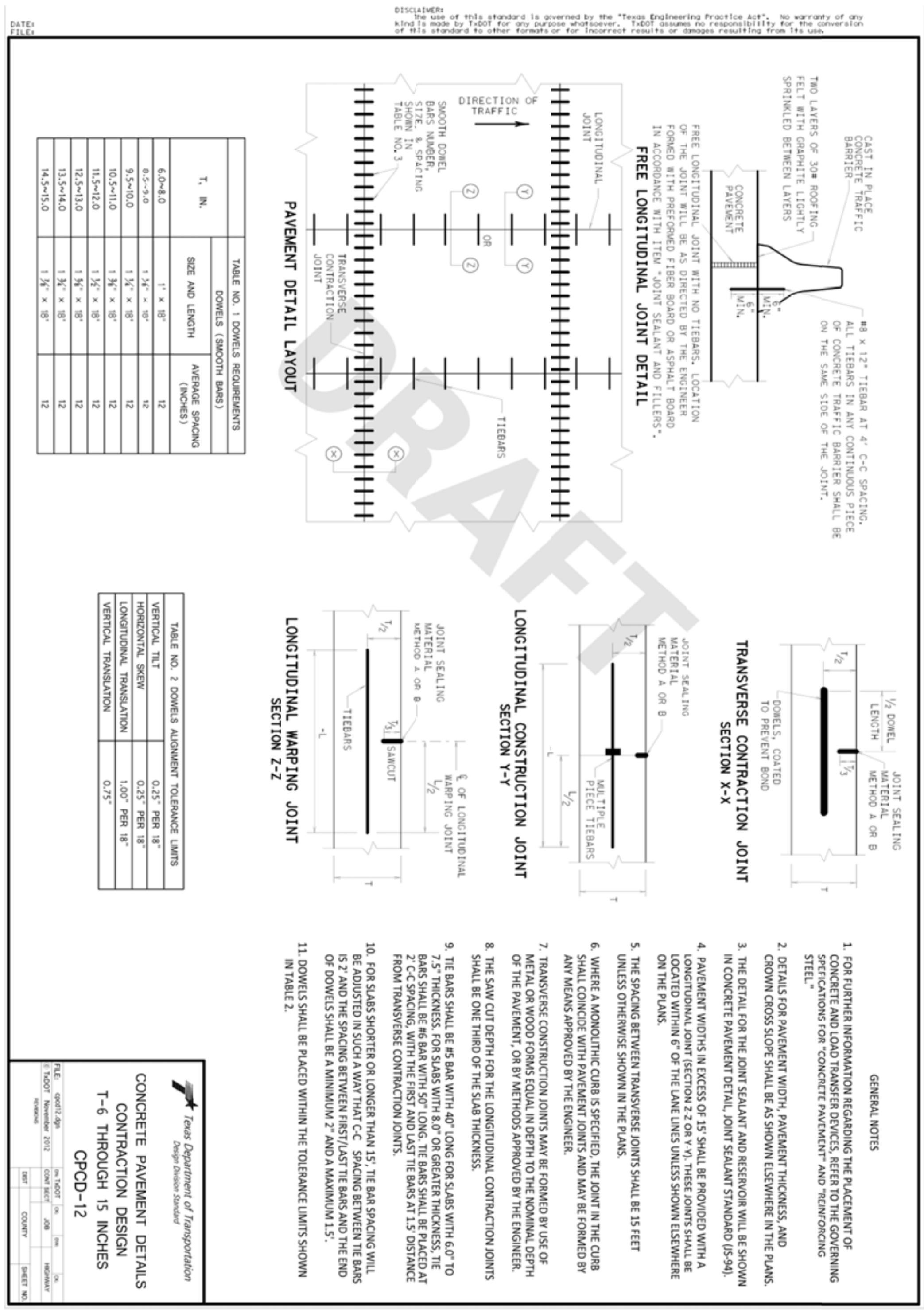
When the placing of concrete is intentionally stopped, install and rigidly secure a complete joint assembly and bulkhead in the planned transverse contraction joint location. When the placing of concrete is unintentionally stopped, install a transverse construction joint either at a planned transverse contraction joint location or mid-slab between planned transverse contraction joints. For mid-slab construction joints, install tie bars of the size and spacing as shown on the plans.

Article 360.4. Construction, Section J. Sawing Joints is voided and replaced by the following:

a. **Continuously Reinforced Concrete Pavement (CRCP).** Use a chalk line or string line to provide a true joint alignment. Saw longitudinal construction and warping joints to the depth and width shown on the plans within 24 hours of concrete finishing. Saw transverse construction joints to the depth and width shown on the plans at any time convenient to the contractor.

b. **Concrete Pavement Contraction Design (CPCD).** Use a chalk line or string line to provide a true joint alignment. Saw transverse contraction joints to the depth and width shown on the plans as soon as the condition of the concrete will permit without raveling and before random cracking occurs. Saw longitudinal construction and warping joints to the depth and width shown on the plans within 24 hours of concrete finishing.

Appendix C: Revised Design Standards for CPCD



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