1. Report No. TX-96/2919-2	2. Government Accession No.	3. Recipient's Catalog No.	
4. Title and Subtitle FORENSIC EVALUATION OF THE CEMENT TREATED BASE FAILURE ON SH 36 IN HOUSTON		5. Report Date October 1995	
		6. Performing Organization Code	
7. Author(s) Sanet Bredenkamp and Tom Scullion		8. Performing Organization Report No. Research Report 2919-2	
9. Performing Organization Name and Address Texas Transportation Institute The Texas A&M University System College Station, Texas 77843-3135		10. Work Unit No. (TRAIS)	
		11. Contract or Grant No. Study No. 7-2919	
12. Sponsoring Agency Name and Address Texas Department of Transportation Research and Technology Transfer Office P. O. Box 5080 Austin, Texas 78763-5080		13. Type of Report and Period Covered Interim: August 1994 - September 1995	
		14. Sponsoring Agency Code	

15. Supplementary Notes
Research performed in cooperation with the Texas Department of Transportation.
Research Study Title: Spall Repair, Base and Subgrade Stabilization, and Non Destructive Test (NDT) Service for the Houston District, Phase II.

16. Abstract

The Cement Treated Base (CTB) on SH 36 experienced rapid deterioration after only three years in service. This report presents the field and laboratory studies undertaken to identify the cause of failure. The main cause of the failure was attributed to the process of carbonation where the calcium component from the cement matrix is leached out, resulting in layer disintegration. Contributing factors to this process were:

a) the substantial clay contamination of the field sands used which resulted in high suction levels within the CTB, and
b) the pavement design which placed CTB's made from different aggregates on top of one another. This resulted in unusual crack patterns which permitted water to be trapped within the base.

Recommendations are given on how to avoid this problem in the future, including reducing the allowable PI of the fine materials from 10 to 8. A field carbonation test is also proposed and demonstrated on another project which was experiencing construction problems.

17. Key Words Cement Treated Base, Carbonation, Failure Analysis, Forensics		18. Distribution Statement No Restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161	
19. Security Classif.(of this report) Unclassified	20. Security Classif.(of this page) Unclassified	21. No. of Pages 44	22. Price

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FORENSIC EVALUATION OF THE CEMENT TREATED BASE FAILURE ON SH 36 IN HOUSTON

by

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October 1995

IMPLEMENTATION STATEMENT

This study concluded that the materials specification of Max PI = 10 should be reviewed for cement treated materials. TxDOT should consider reducing the MaxPI to 8 or less. The practice of placing different stabilized layers one on top of another should also be reviewed.

A field carbonation test is proposed to diagnose future problems.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the Texas Department of Transportation (TxDOT). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineer in charge of the project was Tom Scullion, P.E. #62683.

ACKNOWLEDGMENT

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SUMMARY

The Cement Treated Base (CTB) on SH 36 experienced rapid deterioration after only three years in service. This report presents the field and laboratory studies undertaken to identify the cause of failure. The main cause of the failure was attributed to the process of carbonation where the calcium component from the cement matrix is leached out, resulting in layer disintegration. Contributing factors to this process were:

a)   the substantial clay contamination of the field sands used which resulted in high suction levels within the CTB, and

b)   the pavement design which placed CTB's made from different aggregates on top of one another. This resulted in unusual crack patterns which permitted water to be trapped within the pavement.

Recommendations are given on how to avoid this problem in the future, including reducing the allowable PI of the fine materials from 10 to 8. A field carbonation test is also proposed and demonstrated on another project which was experiencing construction problems.

CHAPTER I

BACKGROUND

The section of interest in this study is on State Highway 36 in Fort Bend County. The project limits are from the Rosenburg city limits east to the Austin County line, a distance of 21 km.

SH 36 was totally reconstructed in 1990. The existing pavement of asphalt surfacing over a gravel base was milled up, stockpiled and stabilized with approximately 5% cement. This layer was placed over the entire new section as a 150 mm base layer. Another 150 mm base layer consisting of cement stabilized crushed limestone and sand was added giving a total cement treated base (CTB) thickness of 300 mm. A 75 mm thick hot mix surfacing was placed over the entire section. The traffic on the section is moderate; the Average Daily Traffic was reported to be 4360, and the design twenty year 80 kN loads were 3.4 million.

After three years in service, several sections on this highway were showing significant distress. The distress took the form of transverse depressions in the wheel paths. Figure 1 shows a typical transverse crack on this highway; evidence of pumping is clearly present. In the worst areas, the depressions were at about 6 meter spacings and about 0.3 m wide with depths ranging from 10 to 20 mm. The depressions were found initially in the wheel paths but eventually extended to cover the entire lane; they were centered around existing transverse reflection cracks. The depressions did not extend into the shoulder. The riding quality of the section was noticeably reduced in these areas. The section had a riding quality similar to that of a faulted jointed concrete pavement.

The purpose of this study was to:

1) identify the cause of the premature distress in the highway, and

2) if warranted, to recommend changes to construction practices and material specifications.

Figure 1. Transverse Depressions in the Wheel Path (Note Pumping of Fine Material).

As the initial deterioration was confined to the wheel path, it was assumed that the primary cause of distress was load related durability problems. Samples were extracted from the highway, and durability tests were performed. Comparison report 2919-1 describes these tests. During this testing, it became clear that load/durability effects were not the primary problem. Consequently, another approach was taken to investigate this failure, focusing on the chemical deterioration of the CTB layer. As described in this report, this involved the use of the following procedures:

1)   Ground Penetrating Radar Analysis.
2)   Laboratory Analysis of Base Material. Sieve analysis and X-Ray diffraction analysis on the clay in the layer.
3)   Thermal properties of the two cement treated base layers.
4)   Soak test to determine the suction properties of the cement treated base (CTB).

The goals of this research were to identify the cause of the deterioration, recommend a pavement rehabilitation strategy, and recommend modifications to TxDOT specifications can be modified to avoid a re-occurrence of this problem.

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CHAPTER II
CONDITION OF THE PAVEMENT

Before discussing the actual results from the specific tests performed on SH 36, it is necessary to show and discuss the general condition of the pavement as it was found after cross sectional cutting and coring. Figure 2 shows a cross sectional cut made through the shoulder of the pavement. This figure shows that the upper CTB has disintegrated in spots. Figure 3 shows a core hole that clearly exhibits how the top layer of the CTB disintegrated while the bottom layer was still intact. Figure 4a shows a core that was taken through a transverse crack in the shoulder of the pavement. The crack runs through the hot mix asphalt and the top 150 mm of the CTB but stops at the top of the lower CTB layer. This suggests that the CTB layers are debonded. Figure 4b shows a core that was taken from an area where the surface depressions were found in the wheel paths. It also shows that the top 150 mm of the CTB is completely disintegrated, and all the fine materials have been pumped out.

From these observations, it was concluded that the problems with the pavement were caused by disintegration of the upper CTB layer. This was unexpected since these materials had been used previously in the Houston area.

As the distresses were concentrated in the wheel paths, it was initially assumed that the problems were primarily "durability under wheel load" related. The durability aspect of these materials was investigated by means of a South African wheel tracking machine. Companion report 2919-1 includes the results of those tests. Once these durability tests were underway, it became clear that other factors were involved in the deterioration of this layer. This report describes these "other factors". They include the investigation of the expansive nature of the clays found in the upper CTB layer and the chemical deterioration of the layer through the process of carbonation (leaching of calcium from cementitious matrix).

Figure 2. A Cross Section of SH 36.

Figure 3. Core Hole with Disintegration of the Top 150 mm of the CTB Layer.

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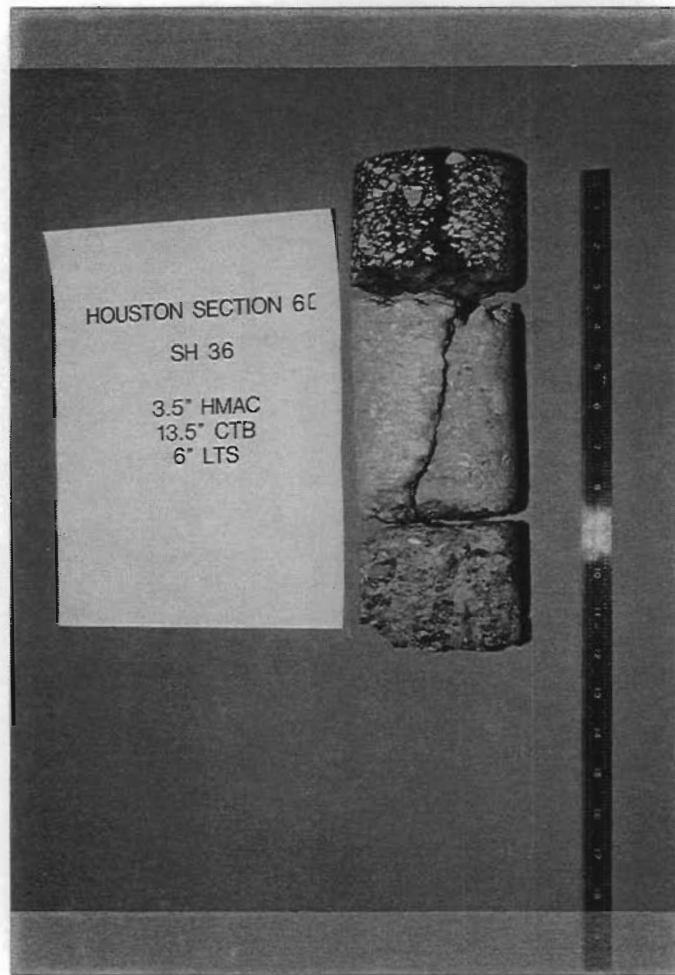
Figure 4a. Core Sample Taken through Crack in the Pavement.
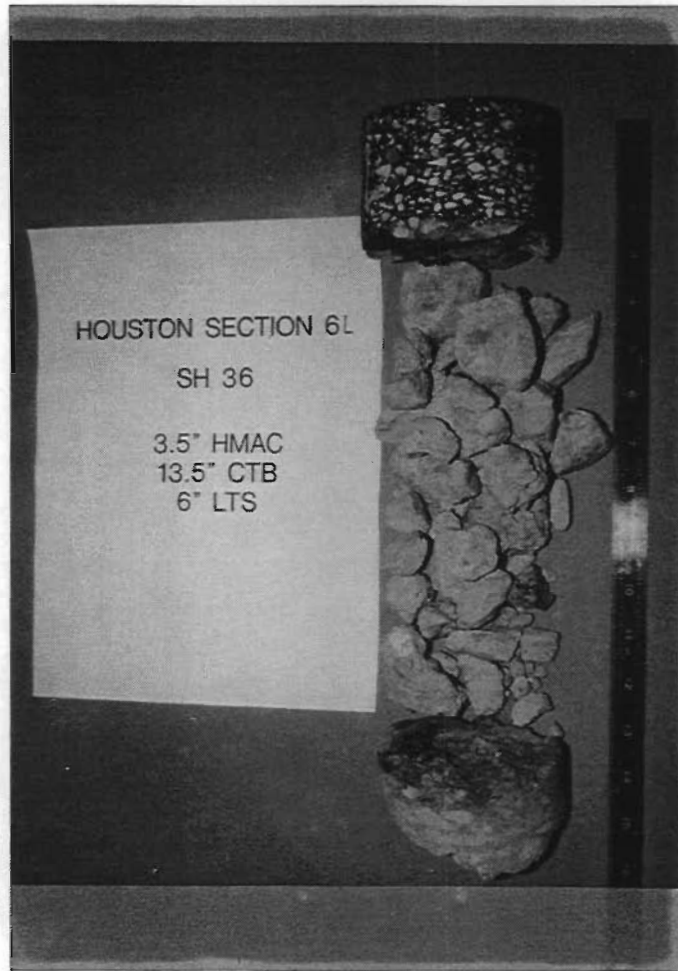
Figure 4b. Top 150 mm of the CTB Completely Disintegrated.

Field observation supports the conclusion that factors other than "durability under load" were responsible for the rapid deterioration. Figure 2 shows a cut made in the shoulder 3 m away from the wheelpath. The beginning stages of deterioration were apparent in the shoulder; the aggregates at the bottom of the upper CTB layer could be easily dislodged by hand.

CHAPTER III
TEST RESULTS

Ground Penetrating Radar Analysis

The basics of Ground Penetration Radar are shown in Figures 5 and 6. The radar antenna transmits a pulse of electromagnetic energy (radar wave) which is either reflected or transmitted at each layer interface. Figure 6 shows a typical return signal from a single pulse. The amplitude and time delays between peaks are used to estimate layer properties and thicknesses. The amplitude of the reflected signal is a function of the dielectric property of the pavement layer; in pavement materials, this property is strongly related to the moisture content of the layer.

The dielectric properties of typical pavement materials are shown below:

	Dielectric
Asphalt	4.5 to 6.5
Concrete	7.0 to 9.0
Dry Flex Base	4.0 to 6.0
Typical Flex Base	7.0 to 9.0
Wet Flex Base	12 to 15.0
Air	1
Water	81

Clearly, the addition of water to any pavement layer will considerably increase its dielectric properties; in the case of a wet base an increase in base dielectric would result in a significant increase in the amplitude of peak B in Figure 6. The addition of air to a base layer (low density) would result in a decrease in dielectric and a decrease in peak B amplitude.

GPR traces from an undistressed "good" and "bad" area of SH 36 are shown in Figure 7. The good area has three distinct peaks, A, B, and C being reflections from the

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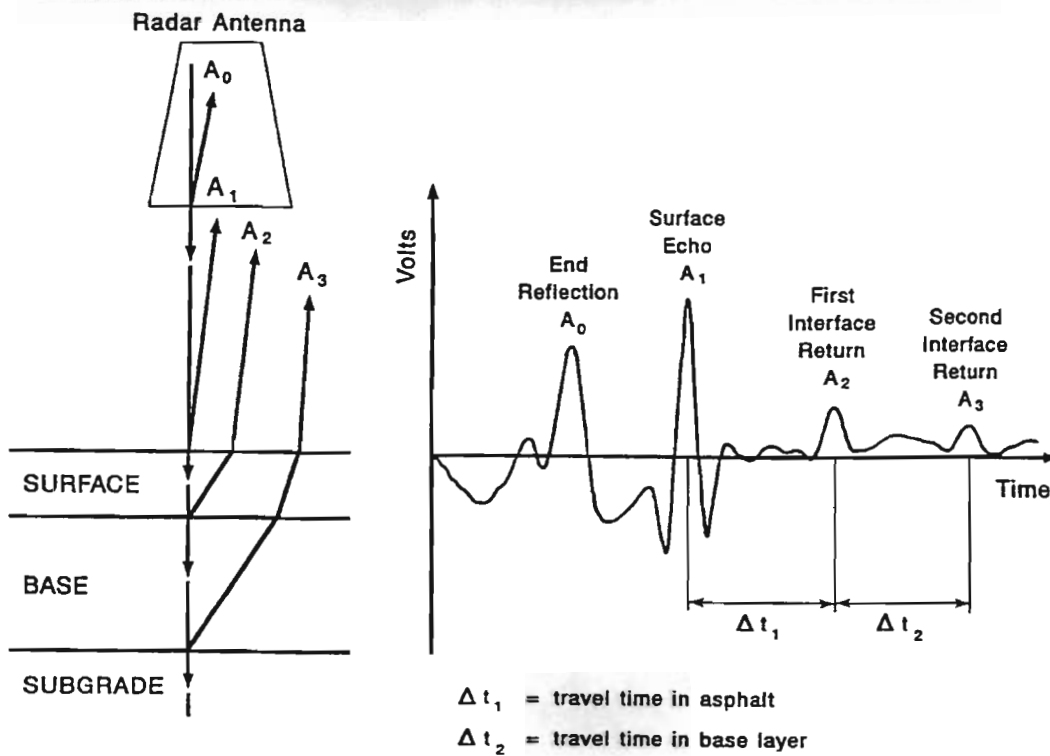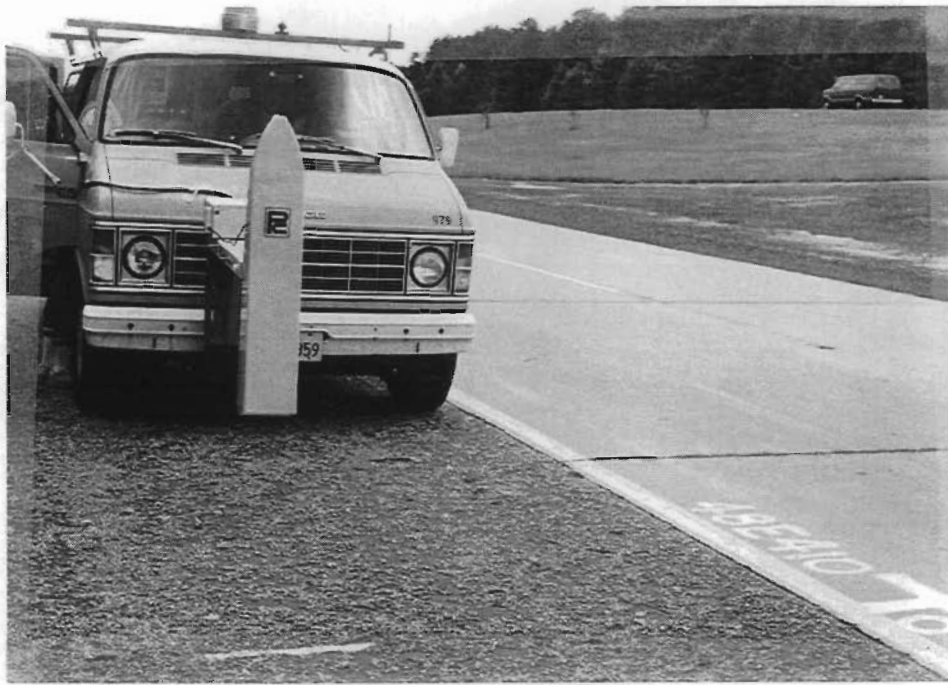Figure 5. Principles of Ground Penetrating Radar. The Incident Wave is Reflected at Each Layer Interface and Plotted as Return Voltage Against Time of Arrival in Nanoseconds.

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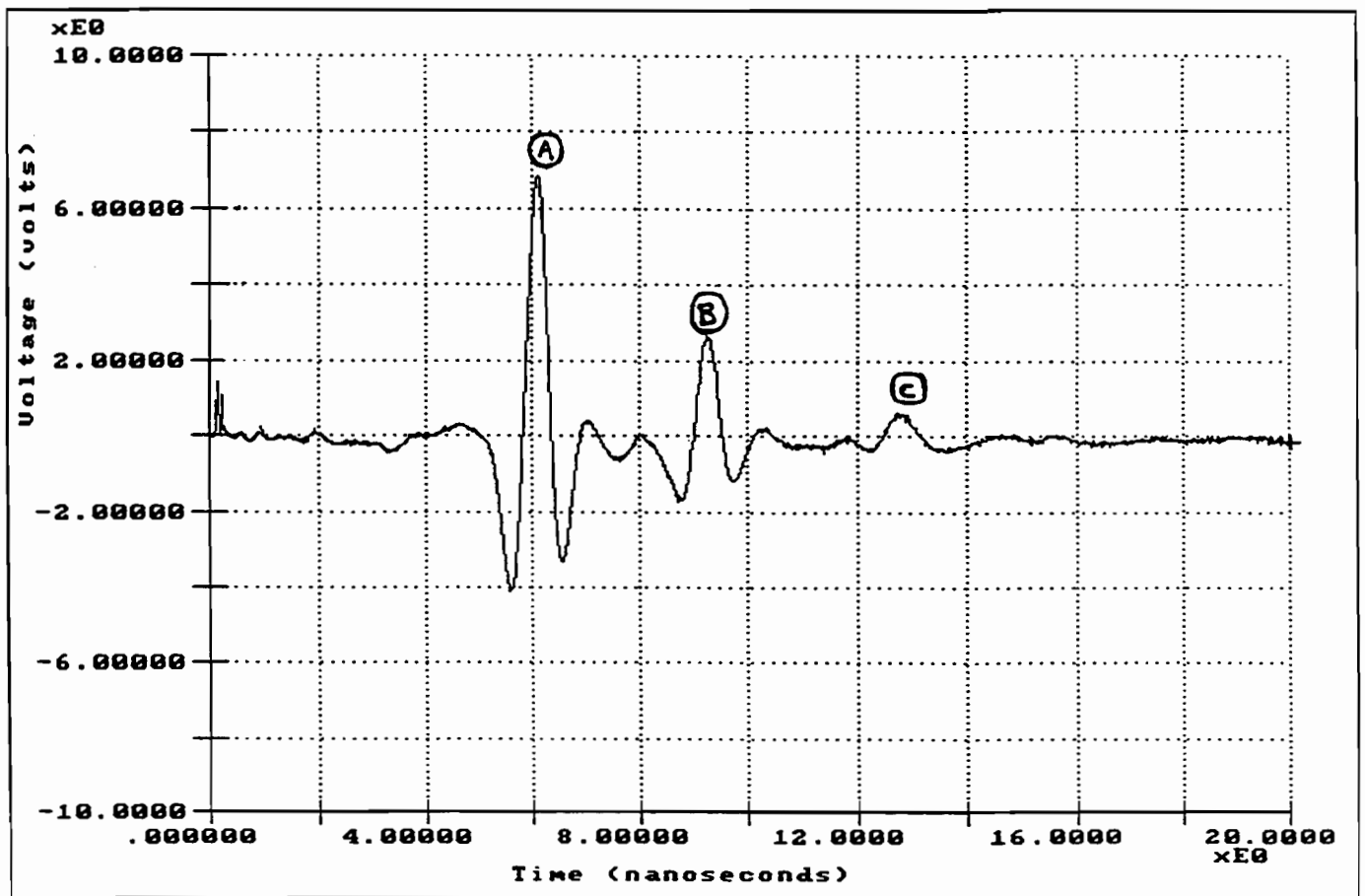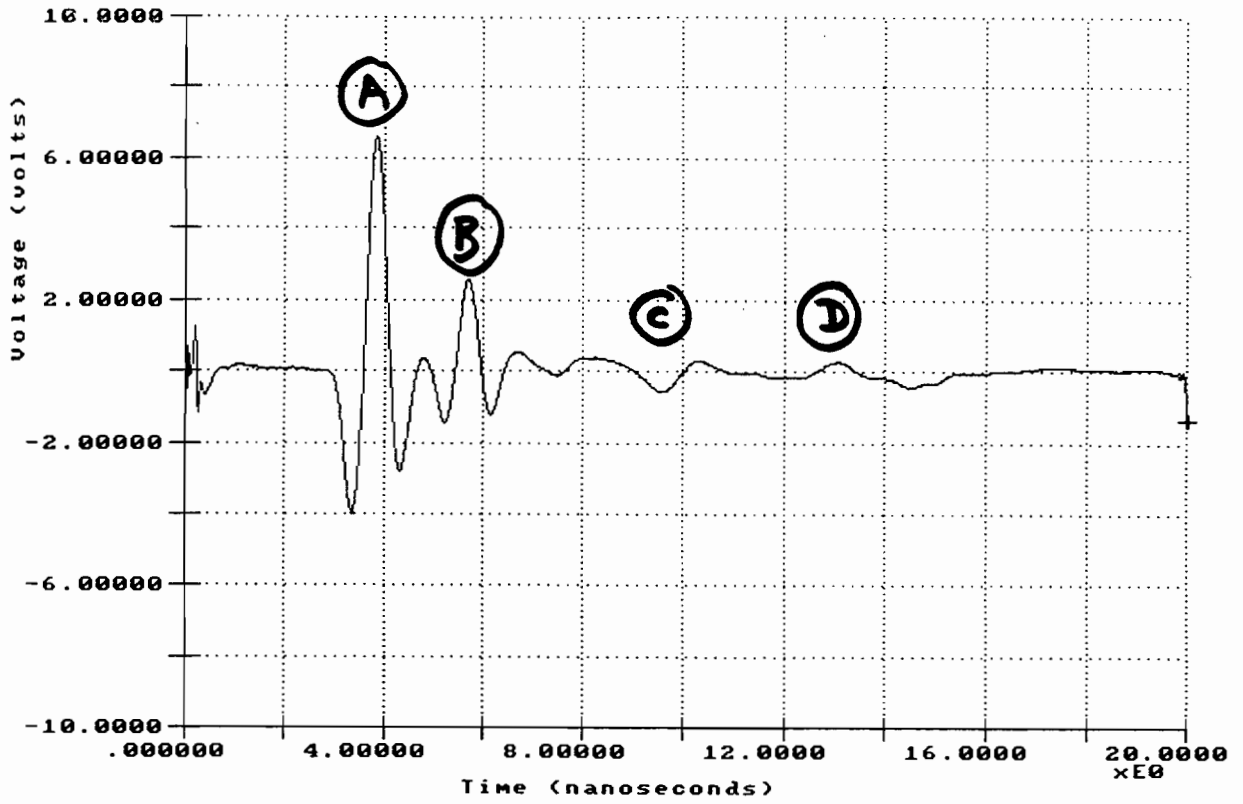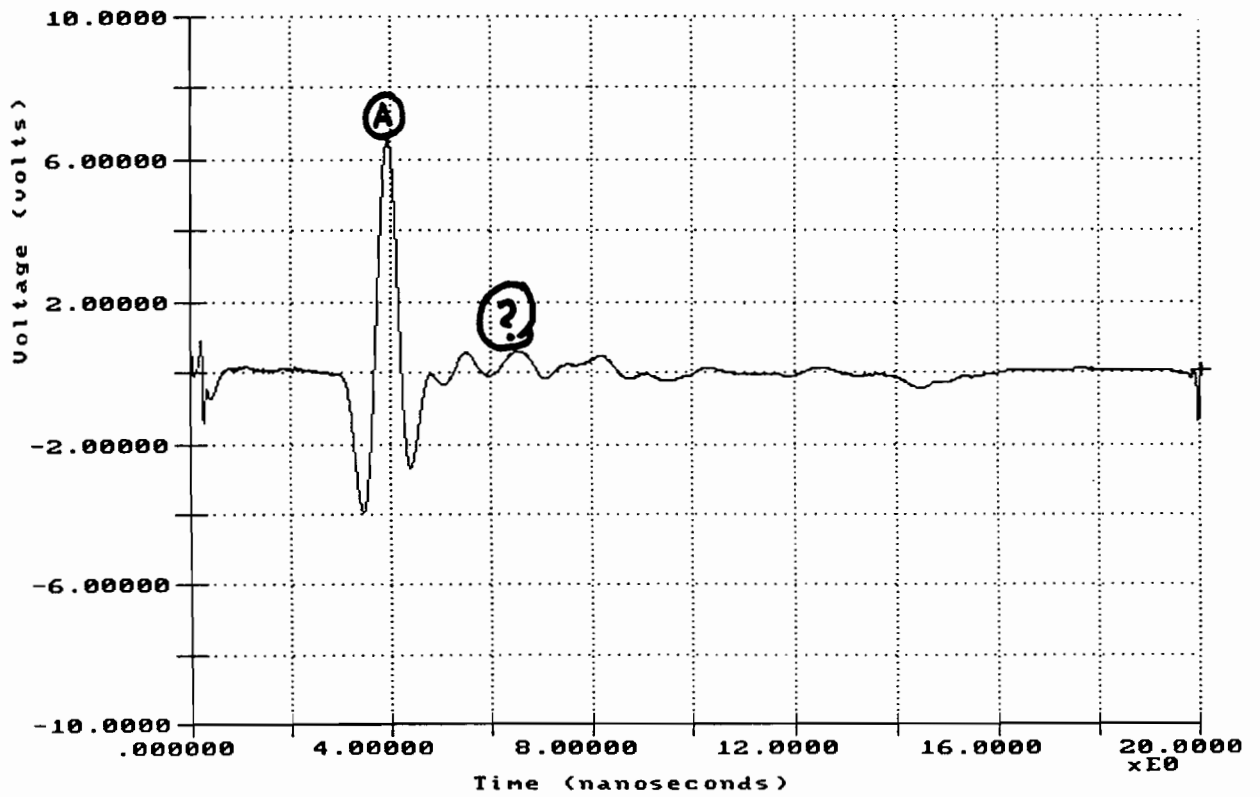
Figure 6.   Typical GPR Waveform. Peaks A, B, and C are Reflections from the Surface, Top of the Base, and Top of the Subgrade, Respectively.

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a) In Typical Sound Area.

b) In Badly Distressed Area.

Figure 7. GPR Waveforms from SH 36.

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top of the AC, top of the upper CTB layer, and top of second CTB layer. Peaks A and B are positive peaks, and peak C is a negative peak. The negative peak indicates that the lower CTB has a lower dielectric than the upper CTB layer. The trace from the distressed area only has one clear peak, that being the surface reflection A. The major difference between these traces is the decrease in amplitude of peak B. This decrease is associated with a change in moisture content and density of the top CTB layer. The lower peak in the distressed area is associated with a low density location. It appears that GPR may be used to discriminate good from bad areas, although more GPR testing and coring should be performed to confirm this.

The amplitudes and time separation between peaks A, B, and C can be used to estimate the thicknesses of the asphalt and top CTB layer. A plot of calculated layer thickness for a short section around the core location is given in Figure 8. The core was taken at 27 m from the start of the section; the actual core thicknesses are marked with a + at this location. The actual thicknesses compare well with the predicted thicknesses.

One other point of some interest from the GPR traces is the calculated dielectric of the top CTB layer. The value in the sound area was calculated to be in the 11 to 13 range; this is substantially higher than previously measured on other CTB sections in Houston. Normally, a value in the range 8 to 9 is found. This high value on SH 36 can only be attributed to the presence of moisture in the top layer of CTB. It rained the day before testing, and it is apparent that some of this moisture was trapped within the top CTB layer. The dielectric value of the asphalt was 6 which is in the expected range. How the moisture enters the CTB is unknown. One possibility is that moisture enters surface cracks and through internal suction saturates the upper CTB layer.

The important conclusion from the GPR testing is that the upper CTB layer appears to be attracting and trapping moisture. The causes and consequences of this phenomenon will be discussed later.
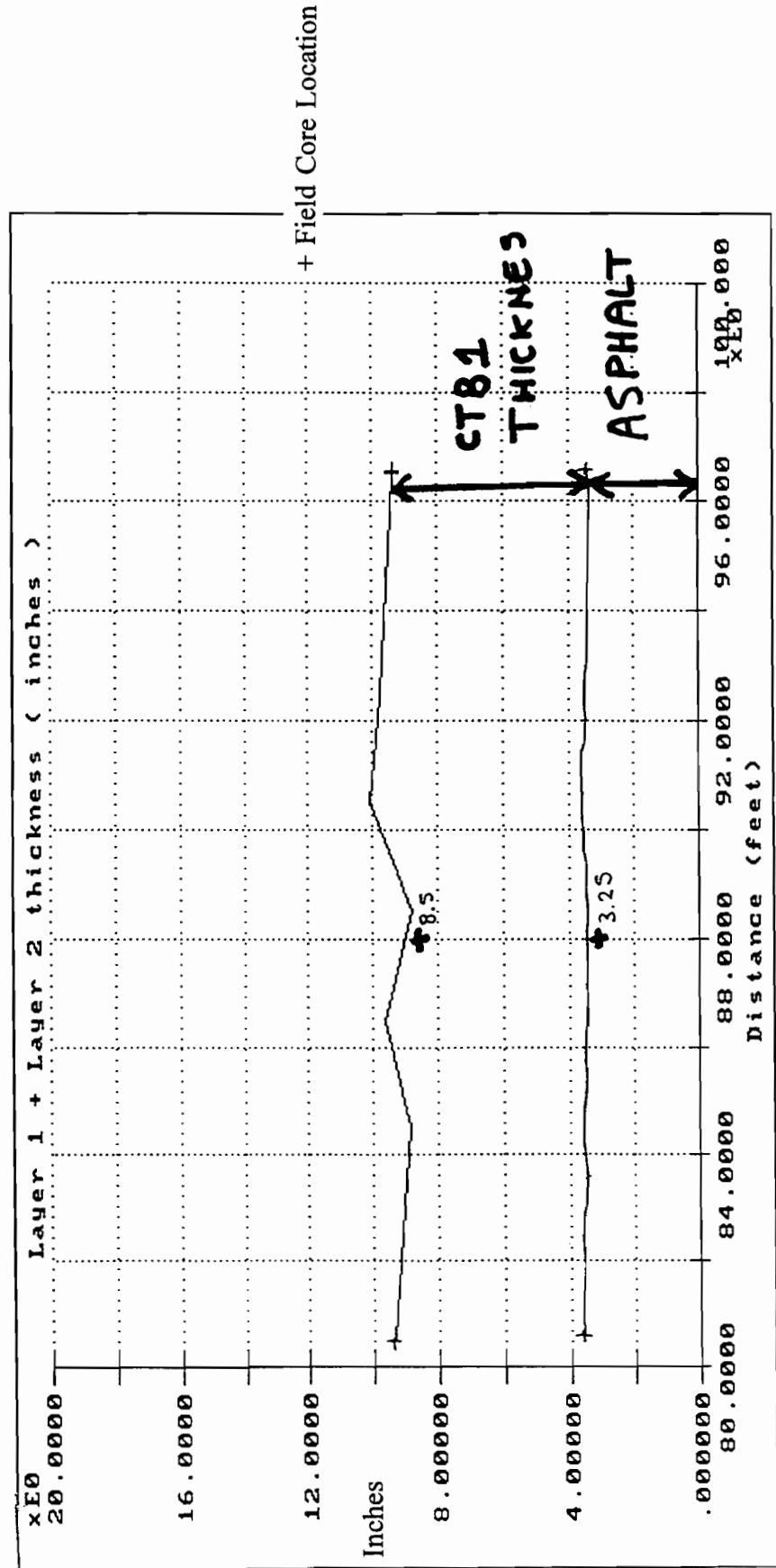
Figure 8. GPR Layer Thickness Estimates for SH 36.

(1 inch = 25.4 mm, 1 foot = 0.305 m)

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Laboratory Analysis of CTB Material

A sieve and subsequent hydrometer analysis were performed on the base material to determine the amount of clay present in the base. From the hydrometer analysis, it was found that the base contained as much as 15% clay. The clay lumps found in the base can be seen in Figure 9. It is believed that apart from the clay seen with the naked eye, the base material is dispersed with microscopic sized clay particles. Even with some severe clay contamination, the base material had a PI of 9 which passes TxDOT specifications for base materials (PI < 10).

The large amount of clay in the base causes a high suction value in this layer, which means the material is water susceptible and will soak up any free water. A proof for the excess amount of water in the CTB layer is the high dielectric constant values found during radar analysis (discussed in the previous section).

Apart from the water retention problem caused by clay in the CTB, another problem is introduced when the clay is of an expansive nature. A small clay

Figure 9. Clay Ball Found in the Top 150 mm of the Cement Treated Base.

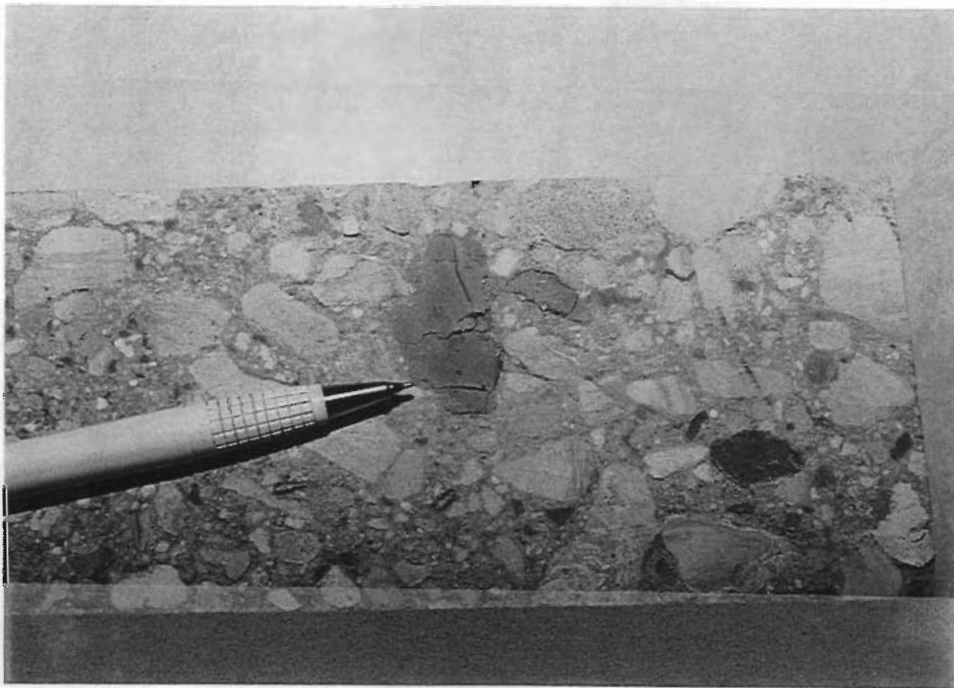sample was removed from the base material, and X-ray diffraction analysis was performed. The minerals in the clay sample were identified as quartz, calcite, mica, kaolinite, and smectite. Of the latter mentioned, kaolinite, mica, and smectite were present in significant amounts. Smectite and mica are highly active clays, and both are known for their expansive properties. The expansion of these clays upon wetting could develop internal forces in the stabilized base that could initiate or accelerate deterioration.

The cement used for stabilization of the base tested to be a high alkali cement. Since the clay contains quartz, it is possible that an alkali-silica reaction was also taking place in the top 150 mm of the CTB. This is another potential cause of the rapid deterioration of the base.

Thermal Properties of the Two Cement Treated Base Layers

Laboratory testing showed that the thermal coefficient of linear expansion of the lower CTB layer was approximately twice that of the upper CTB layer. The thermal coefficient of expansion for the lower layer was $2 * 10^{-5}$ cm/cm/°C and for the top layer was $1.1 * 10^{-5}$ cm/cm/°C. The cement used for stabilization was the same for the two layers; however, the bottom layer consists of recycled asphalt base while crushed limestone and sand were used for the top layer. These two different materials have different shrinkage and thermal expansion properties. During curing and temperature cycling, these layers probably cracked at different locations and debonded. This is the probable reason for the transverse cracks in the two layers occurring at different locations, as observed during coring where cracks in the upper CTB did not extend into the lower CTB. This also means that water entering the pavement via surface cracks will be trapped between CTB layers.

Soak Test to Determine the Suction Properties of the Base Material

Dielectric constant is an unit-less electrical property that strongly correlates to the amount of moisture in a tested specimen. The dielectric constant of most dry solid materials in soils, sands, and clay vary between 2 and 4. Water has a dielectric constant

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of 78. By measuring changes in the dielectric constant of a material, the moisture content of the material may be determined.

A soak test was performed by placing various CTB core samples of 150 mm height in a water bath filled to a height of 25 mm. Six cores from the upper CTB layer of SH 36 were included in this study together with three additional CTB cores from different highways. A probe that measures surface dielectric constant was connected to the top of each of the core samples. Figure 10 shows a schematic view of the setup. The core samples were left to freely soak up the water in which they were standing. The dielectric constant measurements at the top of each core sample were monitored with time. The changes in surface dielectric constant over time are depicted in Figure 11. It is evident from this figure that the problem samples from SH 36 had a dielectric constant value greater than 12 after 2 days, while the sound core samples from other locations had a dielectric constant value that generally stayed below 10. It was
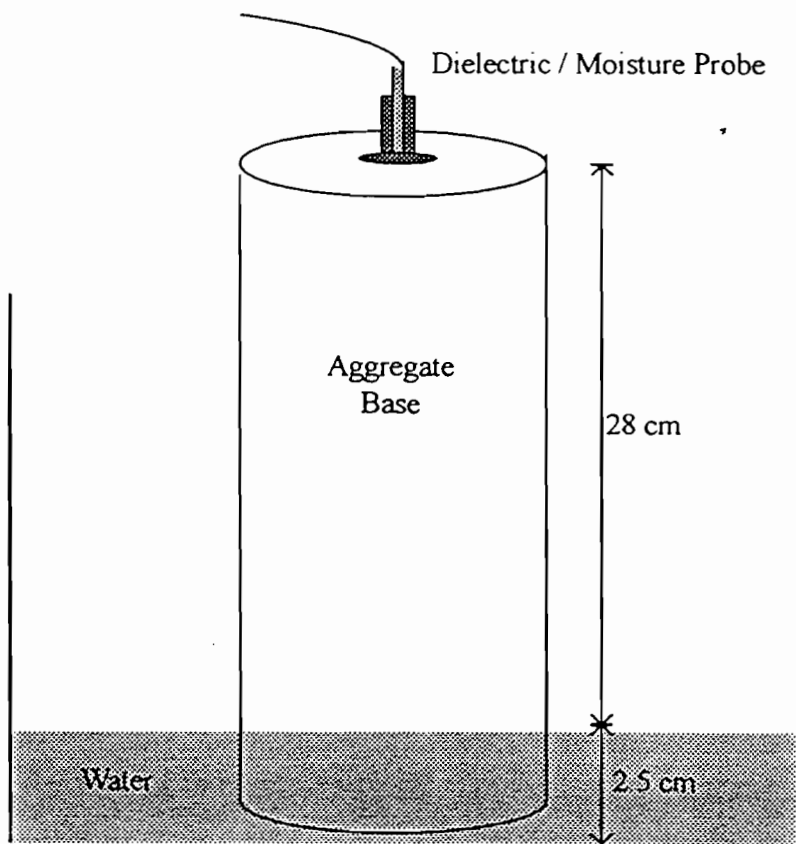

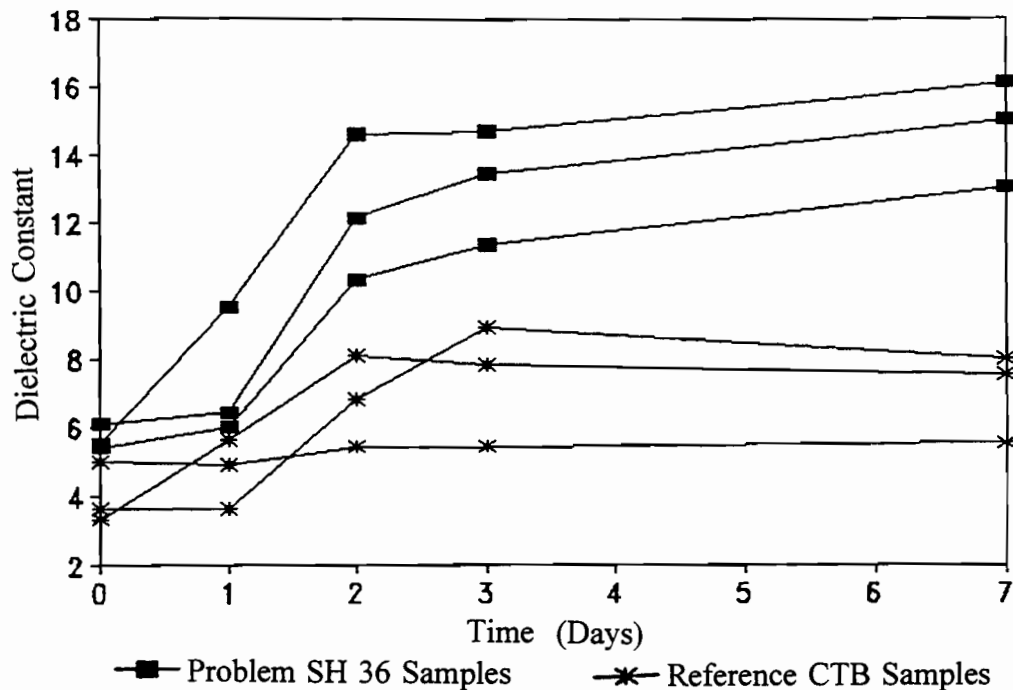Figure 10. Experimental Setup of the Soak Test.

Figure 11. Dielectric Values vs. Time of Samples During Soak Test.

apparent from visual observation that the SH 36 cores were saturated after two days. The moisture reached the surface through internal suction forces. Even after two weeks of soaking, none of the other cores reached saturation. This indicates that the rate water is absorbed by the samples containing clay lumps is much higher than the rate water is soaked up by the sound samples.

After about the fifth day of conducting the soak test, another phenomenon became apparent. The samples from SH 36 had white crystals forming on all the evaporation surfaces, especially at the top of the core samples. This phenomenon is known as efflorescence in concrete and carbonation in stabilized layers; this commonly occurs when air or water leaches through these materials. The white crystal residue is calcium carbonate that forms when calcium hydroxide leaches from the stabilized base and reacts with carbon dioxide from the air. The reduction of available calcium hydroxide in the stabilized material is accompanied by a dramatic strength loss and can lead to complete chemical decomposition. Efflorescence only occurred on the core samples from SH 36 with a high clay content. Two samples showing evidence of efflorescence are shown in Figure 12.

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Figure 12. Signs of Efflorescence on Core Samples from SH 36.

This carbonation process has been extensively researched by South African researchers. The work of Netterberg (1), Netterberg and Paige-Green (2), and Sampson, Netterberg, and Poolman (3) are excellent references for this phenomenon. Their work has focused on carbonation starting during the curing period and continuing rapidly in the early life of the pavement. Inadequate curing of the stabilized layer caused the initial problem. The secondary deterioration occurs with high permeability layers which continue to deteriorate with time.

A summary of the South African experience and description of their carbonation test is given in Appendix A. The typical deterioration as described by the South Africans was also found in a subsequent project on US 59 in Houston. Appendix B includes details of that project.

CHAPTER IV
CONCLUSIONS

- The deterioration of SH 36 is believed to be primarily related to leaching of calcium hydroxide from the cement stabilized base. Calcium is one of the main constituents in the hydration process of cement, and its removal will cause a rapid loss in strength.

- Although the first signs of deterioration were noticed in the wheel paths, deterioration was also observed in samples taken from the shoulder of the pavement. This observation supports the theory of chemical rather than physical causes as the primary reason for the failure. The vehicle loads simply accelerated the deterioration process.

- The presence of trapped moisture seems to be the major factor causing the deterioration of the cement treated layer. Two factors contribute to the presence of excess moisture and deterioration of the top 150 mm of the CTB. The first is the presence of excessive clay in the stabilized material. Not only does the presence of clay encourage the movement of water through the layer, but the expansive nature of clay upon wetting enhances the formation of cracks. The second factor leading to the deterioration of the CTB is the debonding caused by the mismatch of CTB properties between the two layers. Water entering through shrinkage cracks in the top 150 mm of the CTB will become trapped between the two layers since the cracks do not extend through the bottom layer. As discussed earlier in this report, debonding is mainly caused by the different thermal and shrinkage properties of the different materials used in the two CTB layers.

- Methods of ensuring that this problem does not occur again in future construction are, firstly, to prohibit the use of different materials for stabilized layers. A second precaution could be to lower the specified maximum PI of the fine material to a value of 8. The presence of excess amounts of clay would be detected with a lower PI specification. The top CTB layer was heavily contaminated with clay but still met specifications.

- The practice of laying different CTB layers one on top of another should be restricted to similar materials. Mismatch of materials can have serious consequences.

- The major cause of layer disintegration is the loss of calcium hydroxide from the stabilizing agent; this process is commonly known as carbonation. The carbonation test, described in Appendix A, is a quick field test that can be performed to determine whether a stabilized layer is carbonated, i.e., whether stabilization occurred or not. If problems are suspected, the carbonation test can detect the adequacy of stabilization.

APPENDIX A
CARBONATION OF LIME AND CEMENT TREATED LAYERS

Background

Carbonation is a process whereby calcium hydroxide is transformed to calcium carbonate in cement or lime treated base materials. This means that the lime reverts to limestone from which it was made and is no longer available to complete the stabilization process and to preserve the cementing compounds. Calcium hydroxide reacts with carbon dioxide. The source of the carbon dioxide could be either the atmosphere or water carrying dissolved carbon dioxide. Carbonation reduces the pH value of the system and is accompanied by shrinkage that amounts to one-third of the total allowable shrinkage. The development of a network of fine cracks, called crazing, may occur on the surface resulting from carbonation of the surface. This has a detrimental effect on the strength of the top layer of stabilized material. Carbonation of the surface can only be prevented by proper curing methods.

The Carbonation Test

The carbonation test is a quick field test developed by a South African by the name of Netterberg (1) and can be used to detect the carbonation of lime or cement treated materials. This appendix summarizes Netterberg's work. The basis of the test is to establish the pH value of the stabilized material and test for the presence of carbonates in the stabilized layer. The pH indicators used in this test are phenolphthalein and phenol red. Hydrochloric acid is used to test for the presence of calcium carbonate. Effervescence of hydrochloric acid indicates the presence of carbonates.

The test is performed on a newly exposed surface of the stabilized material. Although exposed surfaces of cement treated materials have a pH close to neutral (a pH of 7), high alkaline conditions are present within the treated material (a pH of about 12 to 12.5). To determine whether carbonation occurred, phenolphthalein and phenol red is sprayed on the cross section of the newly exposed surface. A surface sprayed by these

agents is shown in Figure 13. After spraying with the pH indicators, the surface is also sprayed with hydrochloric acid to test for the presence of carbonates. It is necessary to spray some of the untreated material with hydrochloric acid to determine whether carbonates are present in the material before stabilization. If it is found that carbonates were present in the parent material before stabilization, the hydrochloric acid test would not be valid to determine the presence of carbonates; however, the pH indicators should still be able to detect carbonation in most cases. The outcome of the test could be one of the following:

Carbonation did not occur:

a)   In the case where carbonates were absent from the original material:

Phenolphthalein turned red.

Phenol Red turned red.

No effervescence with acid.

b)   In the case where carbonates are present in the original material:

Phenolphthalein turned red.

Phenol Red turned red.

Effervescence with acid.

Complete carbonation occurred:

a)   In the case where carbonates were absent from the original material:

Phenolphthalein showed no color change (transparent).

Phenol Red turned red.

Effervescence with acid.

b)   In the case where carbonates are present in the original material:

Phenolphthalein showed no color change.

Phenol Red turned red.
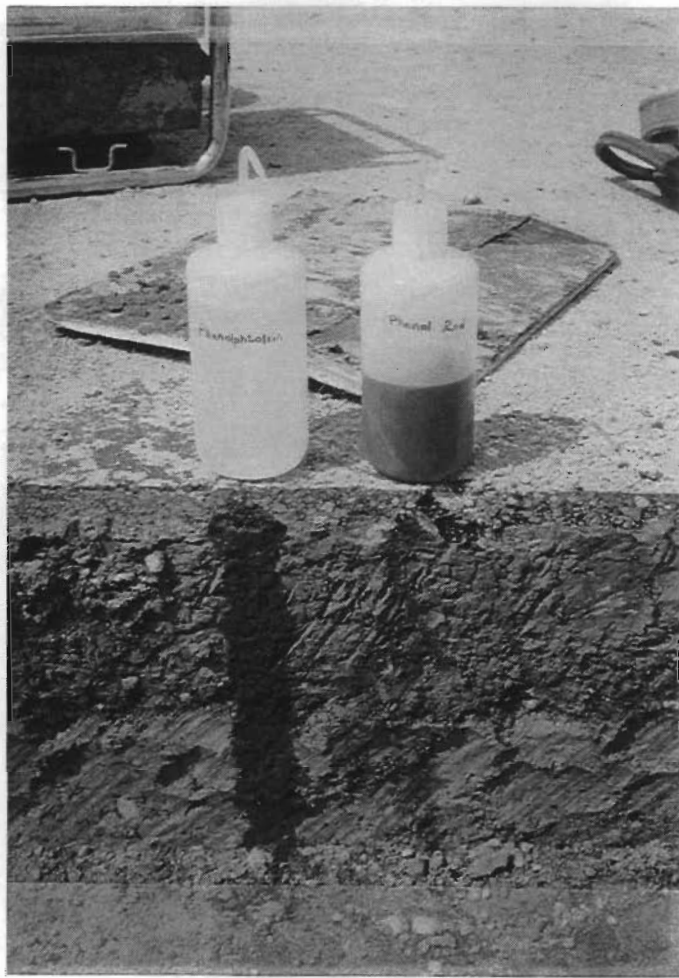
Effervescence with acid.

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Figure 13. Carbonation Test Performed on a Newly Exposed CTB.

In all of the above cases, a yellow color with phenol red probably indicates that the stabilizer was never added or was not properly mixed through the material.

Problems Associated with Carbonation

Most strength problems associated with cement and lime treated bases are due to carbonation of the surface layer during curing before sealing. This surface disintegration problem could possibly be prevented by better construction practices.

One key controlling factor is permeability of the layer. If the layer is impermeable, like concrete, the deterioration will only occur at the surface or close to cracks. However, if the layers are permeable, structural deterioration can occur. Deterioration because of the loss of stabilizer normally only presents itself after periods of months. However, the carbonation process can be very rapid, affecting a layer about 1 mm thickness per day. Laboratory tests have shown that carbonation normally weakens the material to one half of its original strength. These laboratory findings are supported by in-situ strength tests with a dynamic cone penetrometer (DCP).

The South African researchers found that carbonation of the top of the stabilized layer is not the fault of the lime, slag, or cement but is largely that of poor curing practices. The practice of curing stabilized layers by spraying them twice a day with water can be harmful as it may not retard, and may even accelerate, the carbonation process. It has also been found that application of a prime coat does not always result in effective curing. Sealing the stabilized layer by constructing the next layer as soon as possible appears to be the best practical solution. It is recommended that the next layer be applied two to three weeks after compaction.

Another more disturbing aspect of the carbonation problem is described by Netterberg as follows:

"Field and laboratory experiments have shown a marked decrease in the strength of some lime-stabilized material after exposure to carbon dioxide and even to the atmosphere, even after two years in service. This might lead to deterioration of the base course if it is subsequently exposed to the

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atmosphere, for example due to loss of the surfacing, cracking, or simply due to excessive permeability of the surfacing. By analogy with concrete technology, it appears that with the stabilizer and the reaction, products can become carbonated, leading to a loss of the cementing bonds. There is also some evidence that this also causes a loss of density in the case of stabilized materials so that the performance of the stabilized material might actually be worse than that of the untreated material. Although the PI might remain low, in other respects, the stabilized material reverts to an uncemented, granular material. Whether distress of the pavement will result presumably depends on the quality of this destabilized material, the density of the stabilized material, whether it had been densified, position of the pavement, traffic, ingress of water, and other factors."

"... some layers can be expected to become completely carbonated and to have lost cementation and density within 3 years. ..."

Recommendations

To prevent the problem of carbonation to some extent, use good quality material for stabilized bases in order to have maximum strength in the case of carbonation and loss of stabilizer. Also, do not economize on stabilizer, and obtain the highest possible density to limit the rate of ingress of carbon dioxide and water. Seal the pavement as quickly as possible after compacting the stabilized layer, and reseal in good time to prevent subsequent ingress of carbon dioxide and rainwater.

The method used for finishing CTB layers in Texas, "slushing," may cause more problems than it solves. Slushing involves wetting of the surface to achieve density. However, it may form a thin weak surface layer which will experience rapid deterioration under load.

Also, review the curing of CTB layers. Due to environmental and other issues, the methods of curing have changed in the past few years. Evaluate the effectiveness of curing techniques.

APPENDIX B
CASE STUDY OF CARBONATION

Description of the Problem

This case study was initiated to evaluate construction problems on US 59 in Houston. The CTB was constructed with aggregates obtained from a recycled asphalt pavement. The material was described as a combination of pea gravel, iron ore, oyster shell, and asphalt concrete. The recycled material met the PI, LL, and grading requirements and was mixed with 5% cement to form the base layer for the new highway. Other than the standard materials tested and the optimum density tests, no other tests were performed on the material; no strength or compatibility tests were performed.

During the initial stage of construction, it was found that the material was not meeting density requirements. The moisture content of the materials were adequate, but the densities were too low. Coring of the layer was attempted shortly after construction. Of a total of 20 cores attempted, only one core was recovered successfully.

Following the coring problems, the pavement was tested with a Falling Weight Deflectometer. It was found that the strength of the section was extremely variable. Dynamic cone penetrometer testing indicated that especially the top few inches of the layer exhibited low strengths.

Low strengths in the top layer of a stabilized base is probably one of the first and clearest signs of possible carbonation of the layer. It was decided to perform the rapid carbonation field test on a newly exposed cross section of the cement treated base as well as the lime treated subgrade.

Test Results Obtained from Carbonation Test

The newly cut trench through the cross section of the layers exposed a cement treated layer that was still moist inside, which indicated that complete hydration had not yet occurred. Spraying these layers with phenolphthalein and phenol red yielded the following results:

- The phenol red turned red throughout the length of the cross section of both layers.

- The phenolphthalein turned red through the lime and cement stabilized layers on the eastern-most side of the pavement. However, when moving to the inside of the pavement, the length of the red strip decreased. This is indicated in Figure 14 which is a schematic representation of the cross section of the pavement. At the pavement edge, the pH of both base and subgrade is high, indicating adequate stabilization. Moving towards the middle of the section, a layer of low pH is observed in the CTB. This carbonated layer increased in thickness when moving to the middle of the section.

Conclusions

The carbonation test indicated that the top few inches of the cement treated base layer was not stabilized. As no samples of the raw materials were available for laboratory testing, only limited conclusions can be made. It is acknowledged that using recycled materials in CTB layers is problematic, primarily due to the large variation in materials composition and gradation within stockpiles. More work is needed in this area. TxDOT should consider stabilizing recycled materials from flexible pavements with asphalt rather than cement. These bases are being constructed under thick concrete pavements where the uniformity of support is more critical than actual strength. It is thought that replacing cement with asphalt will have little or no influence on the ultimate pavement thickness designs.

The following are proposed as contributing factors on the failure of US 59:

a) No mix design specifies the percentage stabilizer to be used.

b) Inadequate curing of the surface.

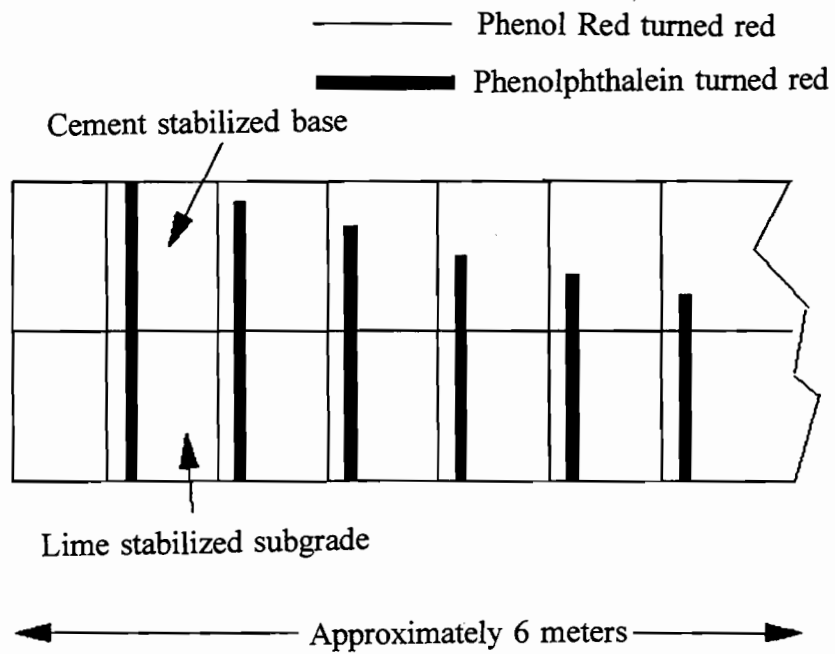c) Overwetting of sections of the base to achieve compaction.

Figure 14.   Carbonation Test Performed on the Eastern End of a Freshly Cut Trench
Through the Cross Section of US 59.