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Each research project will have an advisory committee appointed by the LTRC Director. The Project Review Committee is responsible for assisting the LTRC Administrator or Manager in the development of acceptable research problem statements, requests for proposals, review of research proposals, oversight of approved research projects, and implementation of findings.

The dedication and work effort of the following Project Review Committee members to guide this research study to fruition are acknowledged and appreciated.

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16. Abstract

Conduit structures dealing with hydraulic drainage needs in the Louisiana highway system include pipe culverts, pipe arch culverts, storm drains, sewers, etc. Although the Louisiana Department of Transportation and Development (LADOTD) has standard specifications for furnishing and installing these conduit structures to guarantee their proper functions, unexpected pavement surface dips still occur at some locations of highway cross-drain culverts and cause the deterioration of pavement ride comfort. The goal of this study was to develop recommendations for design and construction procedures to eliminate such pavement surface dips above highway culvert crossing structures.

Researchers conducted a literature search and field investigation on existing pavements at various cross-drain locations with and without the pavement surface "dip" problem. In addition to conventional laboratory tests, full-scale trench backfill tests at the Louisiana Transportation Research Center's (LTRC) Pavement Research Facility (PRF) site evaluated different backfill materials in a controlled environment. Four construction projects accommodated field trench backfill testing sections with various backfill materials to further verify the findings obtained previously. The field testing sections used concrete pipes varied in size from 36 to 54 inches. Using different field compaction equipment and methods, the study explored and evaluated factors that influence the quality of highway cross-drain trench backfill. Relevant cost information is also included for future reference.

The results from this study indicate that pavement surface dips at highway cross-drains on Louisiana highways involve many complex factors. The field probing tests revealed that the occurrence of the pavement dip depended largely on the relative stiffness of trench backfill materials with respect to their adjacent natural soils. The occurrence and magnitude of pavement surface dips depended also on other factors such as the stiffness of the pavement structure and truck traffic loading, etc. When a dip occurred at the surface, the trench backfill underneath was weaker than adjacent subgrade soils. Construction environment, contractors' workmanship, backfill materials, and compaction are the major factors controlling the quality of trench backfill compaction. Sand used in Louisiana is not a good backfill for highway cross-drains due to its very poor gradation and difficulty in compaction. Alternatives such as crushed limestone and flowable fill should be used for highway cross-drains because of their good performance after placement. The DCP device can be useful in evaluating the quality of trench backfills. LADOTD has implemented the results from this study by modifying the current specifications and standard design detail plans to accommodate the complicated field construction conditions.

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Alternative Methods to Trench Backfill

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Louisiana Transportation Research Center

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ABSTRACT

Conduit structures dealing with hydraulic drainage needs in the Louisiana highway system include pipe culverts, pipe arch culverts, storm drains, sewers, etc. Although the Louisiana Department of Transportation and Development (LADOTD) has standard specifications for furnishing and installing these conduit structures to guarantee their proper functions, unexpected pavement surface dips still occur at some locations of highway cross-drain culverts and cause the deterioration of pavement ride comfort. The goal of this study was to develop recommendations for design and construction procedures to eliminate such pavement surface dips above highway culvert crossing structures.

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ACKNOWLEDGMENTS

The Louisiana Department of Transportation and Development (LADOTD) financially supported this research through Louisiana Transportation Research Center (LTRC) with State Project Number 736-99-1128 and LTRC Project Number 03-3GT. Colleagues in the geotechnical, pavement, and bituminous laboratories of LTRC conducted the actual research work. The principal investigator also acknowledges the direction of the Project Review Committee and the Project Manager of this research and the help and support from LA DOTD maintenance and construction engineers, project engineers, and inspectors of different districts for the field test activities. The student workers from Louisiana State University also play an important role in processing of laboratory and field test data and preparing this report.

IMPLEMENTATION STATEMENT

The conclusions and recommendations from this research are intended to improve LADOTD's current highway cross-drain trench backfill construction. The major results from this study have been implemented by modifying the current LADOTD specifications and standard design detail plans to accommodate the complicated field construction conditions. The new specification stipulates that stone aggregate or recycled Portland cement concrete that meets the LADOTD specifications be required as backfill materials for all cross drain pipes and side drain pipes under paved areas of travel lanes, shoulders, and turnouts subject to traffic. Also, each district is expected to validate the option of using RAP as trench backfill before RAP is officially specified as backfill material. Four workshops on pipe installation and inspection were conducted throughout the state to disseminate the results from this study and discuss the modified LADOTD construction specifications and details of highway trench backfills. These workshops included both presentation and field demonstration and were cosponsored by LTRC, Concrete and Aggregates Association of Louisiana, and the Louisiana Local Technical Assistance Program.

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INTRODUCTION

Conduit structures are commonly used in the Louisiana highway system for dealing with hydraulic drainage needs. These structures include pipe culverts, pipe arch culverts, storm drains, sewers, etc. Although LADOTD has standard specifications for furnishing and installing these conduit structures to guarantee their proper functions, unexpected settlements still occur at some locations of cross drain culverts under highway pavements. These settlements cause the deterioration of pavement ride comfort by forming "dips" in newly-constructed pavement riding surfaces. Figure 1 shows one example of such situations.

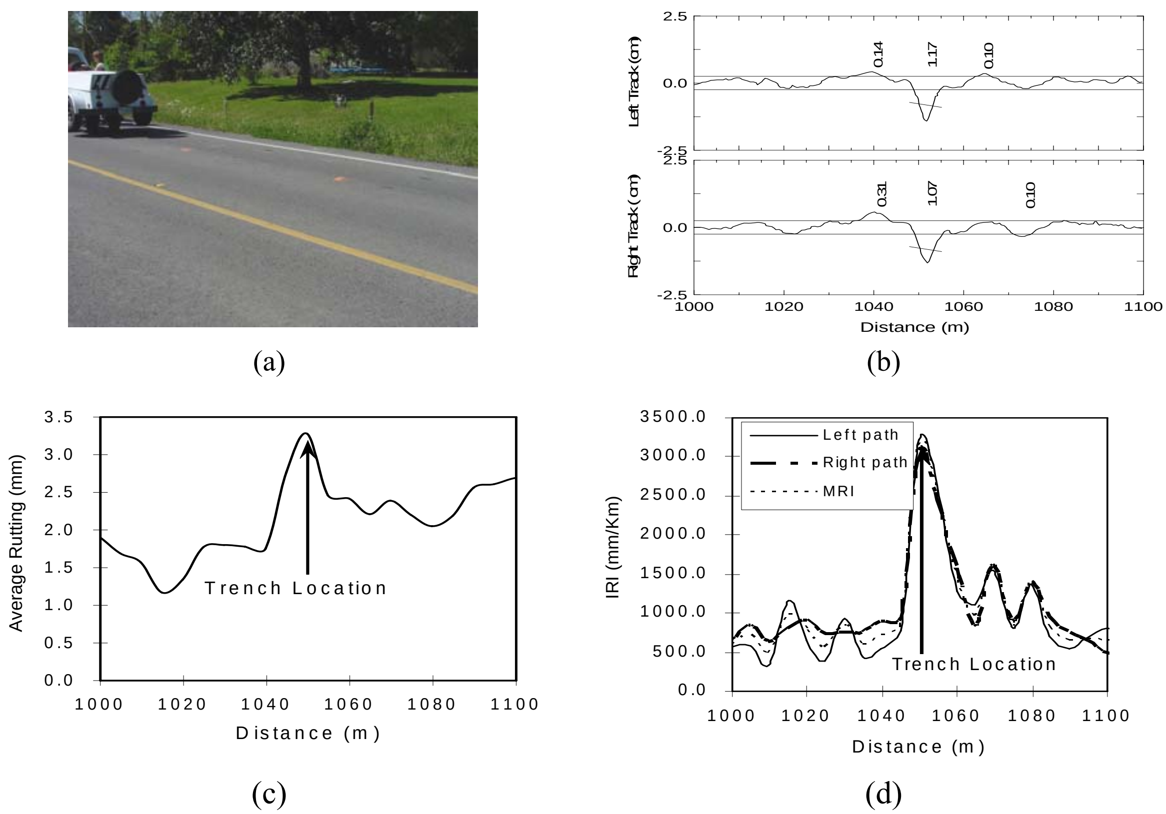


Figure 1
Pavement deterioration at a cross-drain pipe location (a) appearance of pavement surface; (b) surface profile; (c) rutting profile; and (d) IRI (International Roughness Index) profiles (MRI: mean roughness index)

The pavement surface profile in figure 1-b indicates a physical dip in the pavement surface. Figures 1-c and 1-d show the rutting profile and International Roughness Index (IRI) profile for the section of pavement. Both indicate that pavement smoothness and riding quality were damaged due to something related to the cross-drain pipe underneath. Because this is not an isolated incident in Louisiana, LADOTD's 2002 Research Project Identification Committee recommended a research project to find solutions to this problem.

To clarify the discussion that follows in this report, the following terminology is introduced as shown in figure 2.

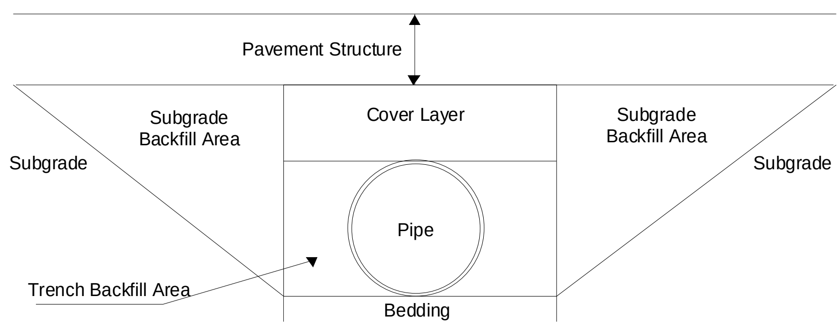


Figure 2
Cross section of highway cross-drain

OBJECTIVE

The goal of this study is to develop recommendations for design and construction procedures to eliminate pavement surface dips above culvert crossing structures. The final report will provide a guideline that will specify when, where, and how to use different backfill techniques and materials to install conduit structures in highway construction or reconstruction and related infrastructures. To achieve this goal, the following objectives were set for this research.

- Understand the cause(s) and mechanism(s) of the pavement surface dip problem over highway cross-drains;
- Examine and monitor the current crossing-pipe building practice specified by the LA DOTD specification;
- Evaluate different backfill materials and construction procedures to prevent the pavement surface dips from occurring;
- Identify factors influencing the quality control of cross-drain trench construction with respect to structural support to pavement.

SCOPE

This research was mainly a construction evaluation of LADOTD's current practices in highway cross-drains. The project also explored the requirement of providing adequate structural support to pavement structures at highway cross-drain locations. This study evaluated seven trench backfill materials in both the laboratory and in the field. Gradation, optimum moisture content, maximum dry density, and other mechanical and performance property tests were conducted in the laboratory for each material.

The field investigation was conducted on existing pavements at various cross-drain locations with and without the pavement surface "dip" problem. Full-scale trench backfill tests were conducted on different backfill materials in a controlled environment at LTRC's pavement research facility (PRF) site to compare and test the workability and performance of these materials. Then, four construction projects were selected to accommodate field trench backfill testing sections to further test and verify the findings obtained previously. The study focused on Louisiana's current construction practices and therefore was limited to concrete pipes (with diameters from 36 inches to 54 inches) available in construction projects although other types of pipes are allowed by LADOTD's specifications. The backfill materials, compaction equipment, and construction methods studied also fell within this scope.

METHODOLOGY

The investigation started with a literature search on the quality of highway trench backfill to acknowledge the state of art on the issue. Various tests were then conducted to investigate the characteristics of highway cross-drains with and without the pavement surface dip problem, the intrinsic characteristics and field performance of different backfill materials, and the complete construction process for highway cross-drain trench backfill.

The study consisted of two major parts: laboratory and field tests. Laboratory tests were conducted to interpret results from the field evaluation tests and to characterize different backfill materials used in the full-scale trench backfill compaction tests and field construction test sections. Several special laboratory testing programs investigated certain issues related to trench backfill in addition to conventional laboratory tests. These programs include the backfill material dry-out, critical dry density of sand, flowable fill, and cement sand mixture.

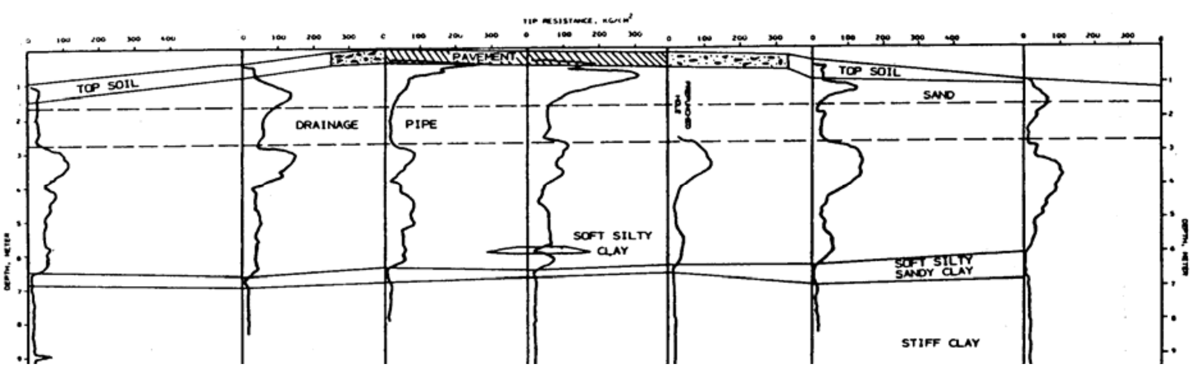
The field test program included three major test programs: evaluation of existing pavements at cross-drain locations, full-scale compaction test at the LTRC's PRF, and field cross-drain construction test sections. The first test program identified the problem and its possible causes. The second sub-test program was conducted in a controlled environment to further verify and validate causes and prepare for up-coming field test program. The third test program investigated the impact of different construction environments on construction quality, tested different backfill materials and procedures, and finalized conclusions and recommendations.

Literature Review

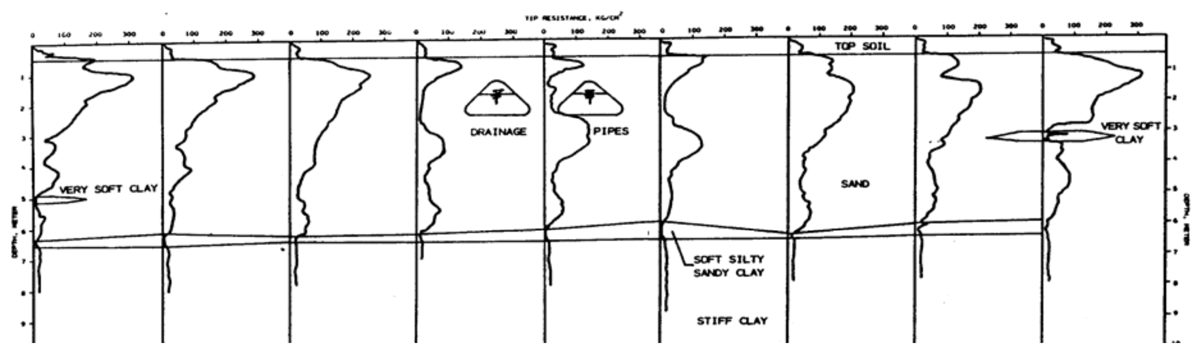
This research focused on highway cross-drains where the backfill should provide a sufficient structural support to pavement, in addition to the integrity and stability of pipe or conduit structures themselves. A large body of research work was available on the trench backfill issue, with numerous publications on different, but related, topics. Although the majority of them focused on guaranteeing the integrity and stability of pipe or conduit structures, only useful information on the performance of different backfill materials and compaction equipments was referred from the past studies to design and conduct this research *[1], [2], [3], [4], [5], [6], [7], [8]*. It was generally referenced that conventional well-graded backfill materials outperform the poorly-graded ones.

The Wyoming Department of Transportation, through the Department of Civil Engineering at the University of Wyoming, conducted research on the mitigation of roadway settlement above buried culverts and pipes *[9]*. The report from that study is the only available document directly related to the pavement surface dip problem discussed here. It concluded that the most likely cause of settlement and roadway damage at a culvert site on a Wyoming highway was a combination of the following factors: use of poor materials as backfill around and above culverts; inadequate compaction; and low soil cover. Therefore, it recommended using high quality granular materials compacted to a high density for backfilling culverts; avoiding highly plastic, compressible fine-grained soils; and using flowable fill for backfilling culverts.

Louisiana has experienced similar pavement surface dip problems over highway cross-drains and accumulated some field testing data on problematic cross-drain sites. Figure 3 shows one example of the tip resistance profiles of a cone penetration test (CPT) conducted along and across a crossing pipe at Interstate 10 (I-10) between Baton Rouge and LaPlace, Louisiana. The profiles were initially presented in a report on a LTRC research project conducted by Yilmaz et al. in 1984 *[10]*. The problem of pavement surface dip was recorded at that location. Figure 3-a indicates that the backfill stiffness along the pipe was not uniform, with weak backfill under the pavement. Most importantly, figure 3-b suggests a noticeable difference in CPT tip resistance between the backfill material and the adjacent embankment subgrade within a depth of 2 to 9 ft. At the locations where pavement surface dips did occur at cross-drain pipes, the backfills were much weaker than the adjacent subgrade soil.



a. Along the pipe



b. Cross the pipe (along the roadway)

Figure 3
Tip resistance profiles of cone penetration test at a crossing-drain (after Yilmaz et al., 1984)

The relative weakness of backfill to adjacent soils outside of trenches was also observed in DOTD's other historically accumulated field test data. "Dips" occurred when the tip resistance of CPT for the trench backfill was 5 tsf (ton per square foot) at Louisiana Highway LA 14; when it was 50 tsf at LA 28; and when it was 250 tsf at I-10. As a comparison, their tip resistances in the adjacent intact soils at these locations were 25 tsf (at LA 14), 110 tsf (at LA 28), and 400 tsf (at I-10) respectively (Zhang, unpublished data).

In the past decades, flowable fill, also called Controlled Low Strength Material, has gained favor in lieu of compacted fill due to its inherent advantages, such as flowing placement, etc. It is a self-compacted, cementitious material and usually consists of some combination of Portland cement, fly ash, high quantities of air, aggregates, water, and/or chemical admixtures. Its good flow characteristics come from the fly ash or air entrainment.

The current evaluation technique for flowable fills resembles that of weak concrete, including the selection of slump range, maximum aggregate size, cement content and fly ash content, estimation of water content, selection of entrained air content, and determination of aggregate content.

Table 1
Example of flowable fill (CLSM) mixture designs

Source State	CO	IA	FL	IL	IN	OK	MI	OH	SC
Cement (lb/yd^3)	50	100	50-100	50	60	50 min	100	50 or 100	50
Fly Ash (lb/yd^3)	-	300	0-600	300 * or 200 **	330	250	2000*	250	600
Fine Aggregate (lb/yd^3)	1700	-	-	-	-	-	-	-	-
Coarse Aggregate (lb/yd^3)	1842	2600	2750	2900	2860	2910	-	2910 or 2850	2500
Water (lb/yd^3)	325	585	500 max	375-450	510	500 max	665	500	460-540

* class F, ** class C

The properties of flowable fill are affected by the constituents of the mix and the proportions of the ingredients in the mix. As the proportion varies to some extent, a wide range of values may exist for the various properties of the material. For example, the upper limit of unconfined compressive strength can be up to 1,200 psi. This allows the material to be used for structural fills. On the other hand, only about 200 psi is expected for the similar material used for trench backfill, which is closer to a well compacted fill. Suitable proportioning can provide adequate flowability, limit segregation, and decrease subsidence, resulting in good in-service properties. Several state highway departments have adopted specifications (1) for mixture proportions of controlled low-strength materials, as shown in table 1, but comprehensive engineering data connected to them are unavailable.

M.C. Webb et al. [11] conducted a flowable fill mix design study; the results are shown in table 2. It was reported [12] that compressive strength increased as the water:cementitious materials ratio decreased, decreased with fly ash:cement ratio, and increased with curing time.

Table 2
Mix component quantities and strength results (after Webb et al., 1998)

a) Mix Constituents (kg)

Material	Mix designation								
	Nom	A	B	C	D	E	F	X	Y
Cement	44	30	59	44	44	44	44	36	44
Fly Ash	296	148	296	222	296	296	296	148	148
Sand	1570	1570	1570	1570	1720	1570	1570	1570	1570
Water	296	296	296	296	296	237	355	296	296
w/c (1)	6.7	9.9	5.0	6.7	6.7	5.4	8.1	8.2	6.7
w/(c+fa) (1)	0.87	1.7	0.83	1.1	0.87	0.70	1.0	1.6	1.5

b) Test Results

7 Day compr. strength, kPa (psi)	1055 (153)	NT (2)	1410 (205)	515 (75)	825 (120)	1435 (209)	515 (75)	205 (30)	NT(2)
28 Day compr. strength, kPa (psi)	1890 (275)	350 (51)	2710 (393)	1645 (239)	1295 (188)	2900 (421)	1115 (162)	540 (79)	295 (43)
Segre-gation	None	Yes	Very little	Little	Little	Very little	Little	Yes	Yes
Spread, mm	380	No spread	250	280	220	No spread	315	–	No spread

Notes:
1. c = cement, w = water, fa = fly ash
2. Specimens A and Y were very fragile at an age of 7 days and broke up during the removal of the plastic molds and/or capping. NT = not tested.
3. ASTM Provisional Standard PS 28-95, Test Method for Flow Consistency of Controlled Low Strength Material
4. 6.89 kPa = 1 psi, 0.45 kg = 1 lbs, 25.4 mm = 1 in.

Laboratory Tests

Laboratory tests were conducted to help interpret results from the field evaluation tests and to characterize different backfill materials used in the full-scale trench backfill compaction tests and field trench backfill construction test sections. For the conventional backfill materials such as sand, gravel, stone, fine-grained soils, etc., the effectiveness of compaction is generally dependent upon their gradation, moisture content, and compaction efforts. Therefore, laboratory tests included the gradation curves (ASTM D 422), Atterberg's limits (ASTM D4318), specific gravities of materials (ASTM D 854), compaction curves with various compaction energy (ASTM D 698), etc. The tested backfill materials included sand, RAP, fine-grained soil, Kentucky and Mexican crushed limestone, etc. In addition to the above basic tests, some other special tests were also conducted as follows.

Material Dry-Out

The material dry-out refers to the process and duration of drying backfill materials when their moisture contents are higher than their optimum (working) moisture contents. The shorter the dry time is, the better the material will be for field backfill construction. Figure 4 shows a laboratory dry-out test setup. It was a simple in-room air-dry process with an ambient temperature of 20°C. Two thousand grams of each dry material to be tested was put in a pan, and saturated with water, as shown in figure 4. Then, the weight changes of the pans due to moisture loss were taken at different times, from which the dry-out rates of materials (moisture loss per hour) were determined.

Figure 4
Dry-out tests of different backfill materials

Triaxial Test of Sand

A triaxial apparatus was used to conduct consolidated drained (CD) test on the sand used as backfill. The purpose of this test was to obtain the correlation between sand's critical dry density and confining stress. The concept of critical dry density is derived from the concept of critical void ratio. Under each level of confining stress, sand with a dry density smaller than its corresponding critical dry density will contract. Likewise, sand will expand if its dry density is larger than the critical value. The critical dry density concept was used in this study to explain the settlement in sand backfill occurring during the trench backfilling, subgrade compacting, and surface paving as the result of sand's volumetric deformation. This volumetric deformation heavily depends on sand's initial void ratio and confining stress (i.e. $p = 1/3(\sigma_1 + \sigma_2 + \sigma_3)$), being either shear dilating or contracting for dense or loose sand, respectively. The confining stress applied to the sand determines its critical void ratio and, in turn, its critical dry density at that stress level.

Specimens for the triaxial test were prepared by using the dry pluviation method with a diameter of 2.8 inches and a height of 6.8 inches. Once a specimen was formed and installed in a loading frame, it was saturated by circulating de-aired water with the aids of backpressure to ensure $B \geq 0.98$. The specimens were then isotropically consolidated under a given level of effective confining pressures (5, 6, 8, and 10 psi). An automatic volume change device monitored the specimens' change in volume during consolidation. The specimens were then loaded at a rate of 0.4 mm/min until they failed. A computer program automatically recorded all the data.

Flowable Fill and Cement-Sand Mixture

A relatively new material for trench backfill, flowable fill is very promising. Moreover, the properties of low-strength flowable fills cross the boundaries between soils and cement concrete, but closer to the ones of compacted soils. Laboratory tests were therefore conducted on several flowable fill recipes to evaluate and document the performance of the material from the perspective of soil mechanics, in addition to the current proportioning technique for general flowable fills.

Material

A single source of cement, fly ash, and fine aggregate was used in this study. The cement was an ASTM C 150 Type I manufactured by Holcim (US) Inc. at the plant in Theodore, LA. A Class C fly ash conformed to ASTM C618 was supplied by Bayou Ash Inc. Fine aggregate was concrete sand (ASTM C33) with specific gravity of 2.53 and loss on ignition (LOI) of 2.0 percent. Chemical admixtures of an air entraining agent and an accelerating agent were also used, which are manufactured by W. R. Grace & Co. Recommended maximum quantities of QPL (Louisiana Qualified Product List) 58BM Grace Daravair 1000 AEA and QPL 58BB Grace Polarset accelerator were 3.0 oz and 60 oz, respectively, per 100 pounds of cementitious materials.

Mixture

Test mixtures based on one cubic yard were prepared according to recommendations suggested by ACI Committee 229 and the AASHTO Guide [13]. Depending on strength and hardening time requirements, cement contents usually range from 50 to 200 lb/yd^3. As the project was focused on the material prepared for trench backfill application, cement contents were maintained below 100 lb/yd^3 in this research. The quantity of fly ash used was determined by flow consistency (flowability) requirements. It is recommended that Class C fly ash is used in quantities of up to 350 lb/yd^3. Selected fly ash contents were a little beyond this upper limit in the study to decrease bleeding and segregation, and to obtain homogeneous specimens. Fine aggregate in the quantities ranging from 2,600 to 3,100 lb/yd^3 are often used in flowable fill mixes. Similar to cement concrete mix design, the amount of fine aggregate in these proportions varies in the quantity that is needed to fill the volume of the flowable fill after considering cement, fly ash, water, and air content.

The recommended maximum quantities of both the air-entraining admixture and the accelerator were added to some specimens. The air-entraining admixture is usually added to improve flowability and decrease the density. As the mixtures are all proportioned with sufficient cementitious materials, adverse segregation due to high air content may be avoided. The accelerator is for early hardening, which is important to some practical applications.

Preliminary trial and error proportioning was performed by adjusting flow consistency with appropriate water content. The recipes selected for further testing and modification produced a flow consistency of about 10 inches in a 3x6 inch-open-ended cylinder flow consistency test, without obvious segregation and bleeding.

Test Procedure

The tests conducted included the standard concrete slump cone (ASTM C 143), 3×6 inch-open-ended cylinder flow consistency test (ASTM D 6103), air content (ASTM C 173), unit weight (ASTM C 138), penetration resistance (ASTM C 403), compressive strength (similar to ASTM D 1633), consolidation (similar to ASTM D 2435), and permeability (ASTM PS 129-01) tests for each mixture.

The LTRC United Compression Model SFM-30E load frame was used for the strength test, and specimens of each mixture with the ages of 1, 7, and 28 days were used in above tests**.** The applied stress sequence in consolidation tests was 1, 2, 4, 8, and 16 tons/ft^2. The duration of each load application was 30 minutes since much of the materials' consolidation would occur within this time interval. The consolidation may come from the compression of small pockets of gas within the pore spaces, the elastic compression of solid grains, and the particle movements and readjustments. Using the falling head technique, specimen cylinders of φ6×4.5 inches were tested for the permeability of flowable fills at the age of 28 days. The specimens were soaked for two days prior to testing.

As a supplement to the flowable fill test program, California Bearing Ratio (CBR) Tests (ASTM D1883) were also conducted on both cement sand mixture and embankment soils allowed by LADOTD's specification to evaluate the suitability of the cement sand mixture as a trench backfill compared to well-compacted subgrade soils. The soils were also tested for their unconfined compressive strength for comparison.

Field Test Program

Evaluation of Existing Pavement at Cross-Drains

Pavements with cross-drains underneath were evaluated to confirm the findings from the literature search and understand why pavement surface dips could occur at certain locations, but not at others. After consulting with local district maintenance engineers and inspectors, researchers selected 20 cross-drain locations in LA DOTD's Districts 03, 08, 61, and 62. Although these locations experienced similar traffic and environmental conditions, some did not have the surface dip problem. The criteria for the selected sites were the severity of pavement "dips" at cross-drains and trench backfill material types. In-situ tests were conducted in both the backfills and the adjacent subgrades for comparison purposes. A typical layout of field tests is depicted in figure 5.

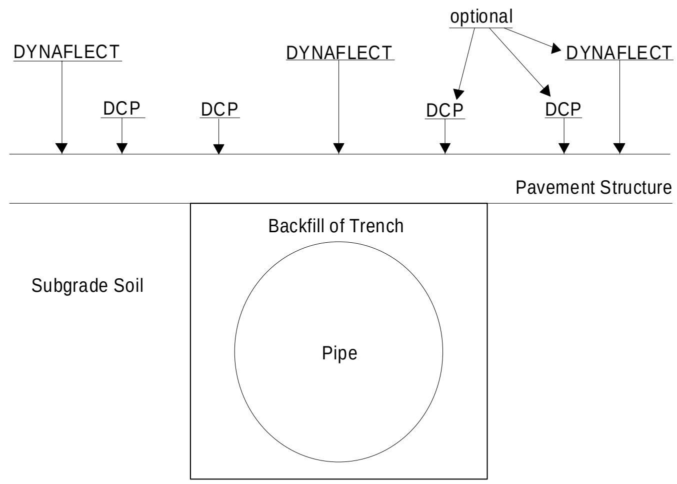


Figure 5
Layout of field test at cross-drain location

Backfill Materials

Sand is one of the major backfill materials in Louisiana due to its availability and low cost. The experience with this material varied widely according to a survey among district construction engineers (communication within LADOTD, unpublished). Some had problems with the material; others did not. Therefore, the field evaluation mainly focused on sand backfill trenches.

Since other materials were also used by LADOTD districts and local cities in their repair and maintenance work on cross-drain pipes, the field evaluation program of existing sites was expanded to include RAP, Mexican limestone, and gravel. The construction procedure used for those materials at the locations tested can best be described as "dump in." No compaction was applied and traffic was allowed to compact the backfills. According to the maintenance crews, the RAP backfill locations were built 3 - 6 months prior to being tested and the gravel and Mexican limestone sections were built approximately one year before. LADOTD has also used flowable fill as a backfill on a trial basis in some districts. Evaluation tests were also conducted at the available locations to obtain some preliminary knowledge about its performance as a trench backfill.

Field Test Techniques

The field-testing techniques included the Dynamic Cone Penetrometer (DCP) and Dynamic Deflection Determination System (DYNAFLECT).

DCP. The DCP is a simple and effective tool for the assessment of in-situ strength of pavement layers and subgrades [9].

<u>Device and Procedure</u>**.** Figure 6 shows the DCP device used in this investigation. It consists of an upper fixed 22.7 inches travel rod with 17.6 pound falling weight hammer, a lower rod containing an anvil, and a replaceable 60° cone of ¾ inches diameter. DCP tests were conducted in the field where existing pavement structures were cored through using drill rig; DCP tests were also conducted at the top of subgrade soils. The test involved lifting and dropping the hammer to strike the anvil, which then penetrated the ¾ inches diameter cylindrical cone from the surface of subgrade soils down to the required depth. It provided continuous measurements of in-situ strength and stiffness of trench backfill and subgrade soils without sampling. During the test, the penetration for each hammer blow was recorded and referred to as the penetration rate (*PR*, in cm/blow).

Figure 6
Dynamic cone penetration (DCP) device

<u>Data Reduction</u>. *PR* represents compliance of soils since larger *PR* values are always associated with weaker soils. To characterize the stiffness of backfill materials and subgrade soils, an index called penetration blow count, N_{DCP}, in blows/4 inches, is used here, which is defined as the average blow count over a 2 inch-thick soil layer, or

$$N_{DCP} = \frac{10}{\frac{\sum_{i=1}^{n} PR_i}{n}} = \frac{10 \cdot n}{\sum_{i=1}^{n} PR_i} \qquad \text{(blows / 4 inches)} \qquad (1)$$

Here, n is the number of *PR* readings within a 2 inch-thick soil layer. If n = 0 in equation (1), N_{DCP} will be

$$N_{DCP} = \frac{10}{PR_{adjacent}} \qquad \text{(blows / 4 inches)} \qquad (2)$$

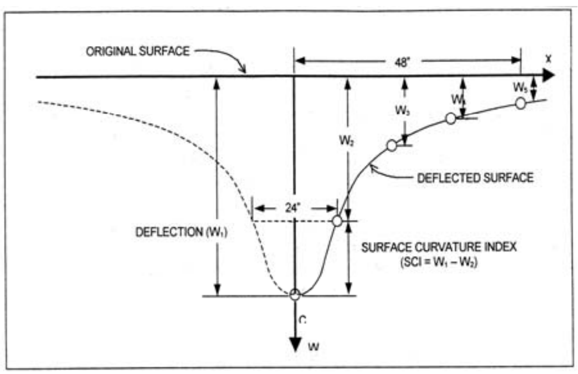
Here, $PR_{adjacent}$ is the penetration rate from the soil just above the 2 inch-thick soil layer considered. Stiffer soils will have higher N_{DCP} values.

The reason for selecting a 2 inch thickness is to cancel out reading errors that occur during DCP tests. The coefficient of 10 is empirically selected with a reference to previous studies *[14], [15]*. Webster et al. (1992) [15] suggested that

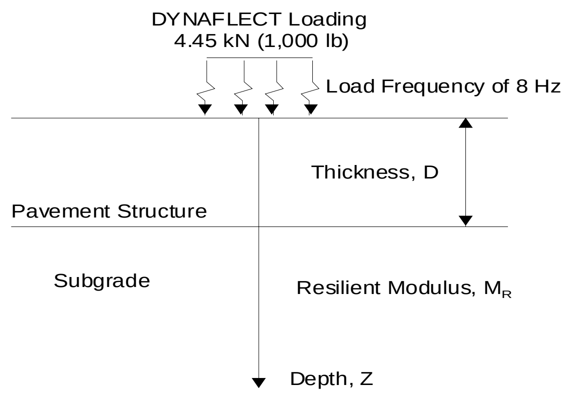
$$CBR = \frac{292}{DCP^{1.12}} = \frac{292}{(10 \cdot PR)^{1.12}} = \frac{292}{10^{1.12} \cdot 10^{1.12}} \cdot \left(\frac{10}{PR}\right)^{1.12} = 1.68 \cdot (N_{DCP})^{1.12} \qquad (3)$$

where *CBR* is the California Bearing Ratio and *DCP* stands for the *DCP* index in mm/blow. Therefore, N_{DCP} will have a simple correlation with other engineering parameters. It also encompasses the normal range of DCP readings for subgrade soils.

DYNAFLECT. DYNAFLECT is a trailer mounted device that induces a dynamic load on the pavement and measures the resulting deflections by using geophone sensors, usually five, spaced under the trailer at approximately 1 foot intervals from the application of the load, as shown in figure 7-a. DYNAFLECT Model Number 1000-8A was used in this study. During the test, a pavement is subjected to 1,000 lbf of dynamic load at a frequency of 8 Hz, which is produced by two counter rotating unbalanced flywheels. The cyclic force is transmitted vertically to the pavement through two steel wheels spaced 20 inches from center-to-center. The dead plus dynamic force during each rotation of the flywheels varies from 1,100 to 2,100 lbf. The resilient modulus, M_R, of subgrade soil is determined according to a normal graph procedure developed by Kinchen and Temple (1980) [16]. This normal graph procedure is based on a double-layer model shown in figure 7-b.



a. Typical DYNAFLECT deflection basin



b. Analytic model of DYNAFLECT for pavement

Figure 7
DYNAFLECT analysis

PRF Full-Scale Compaction Test

The literature review indicated that material types may influence DCP test results [17], [18], [19], [20]. Therefore, full-scale controlled tests were conducted at LTRC's PRF site to establish the correlation between the penetration blow count, N_{DCP},and resilient modulus, M_R,of different materials that were previously tested in the field evaluation. The tests also explored the workability, strength, and stiffness of different materials as backfill at different compaction efforts. The results would serve as a guideline on future field testing sections of trench backfill construction.

Test Trenches and Backfills

The three test trenches as shown in figure 8, each 20 ft. long, 4 ft. wide, and 3 ft. deep, were constructed at the PRF site using three backfill materials: crushed Kentucky limestone, RAP, and sand. Figure 9 shows the photos of sand compaction. These trenches were filled in three 12 inch-thick lifts. Each trench was divided into three equal sections with different compaction efforts: light, medium, and heavy. Light compaction was achieved from one compaction pass by a vibratory plate compactor (Wacker Packer, Model Number WP1550AW, 200 lb); medium compaction was achieved from four compaction passes by the vibratory plate compactor; heavy compaction was achieved from four Wacker Packer compaction (Model BS45Y 53kg, 117lb) passes in addition to four vibratory plate compactor passes. The bottom and sides of the trench were wrapped in geo-fabric to separate the backfill materials from native soils.

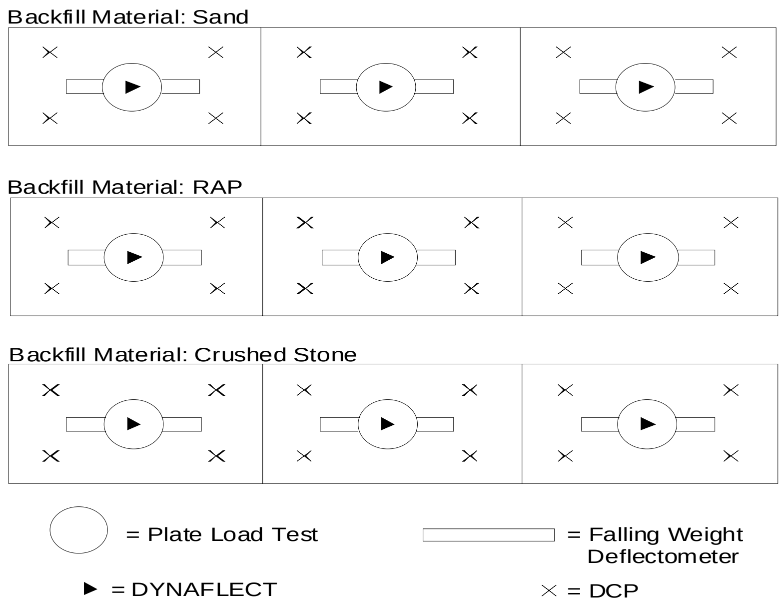


Figure 8
Layout of three full-scale test trenches

a. Compaction of sand at first lift

b. Compaction of sand at third lift

Figure 9
Example of full-scale compaction tests

More Field Test Techniques.

Plate load test (PLT) and Falling Weight Deflectometer (FWD) were also used, in addition to the DCP and DYNAFLECT tests, in this phase of investigation for the mechanical properties of the compacted backfill materials in the full-scale trenches.

PLT. The PLT is a standard statically loading test usually used to determine the modulus of subgrade reaction (k value). A circular plate of 12 inches in diameter was used for applying the load in this study. The PLT was performed according to ASTM D1196. The load was applied in uniform increments at a moderately rapid rate. The magnitude of each load increment was small enough to record a sufficient number of load-deflection points to produce an accurate load-deflection curve. Each increment of the load was maintained until a deflection rate of no more than 0.001 inch/min was observed for three consecutive minutes. The applied load was released in three approximately equal increments once a peak load or the minimum/steady magnitude of load increment to deflection increment was reached. The rebound deformation during unloading was continually recorded until it ceased**.**

Data obtained from PLT tests are usually presented in a load-deflection form as shown in figure 10. The modulus determined in this test is from the second loading cycle. For a rigid plate, the reloading modulus is defined as:

$$E_{PLT} = \frac{2 \cdot \left(1 - \nu^2\right)}{\pi \cdot \delta_2} \cdot \frac{P}{R} \tag{4}$$

Where P is the load applied to the surface of the plate; ν is the Poisson's ratio ($\nu = 0.35$ assumed for the granular materials tested here); R is the radius of the plate; and δ_2 is the deflection under the second loading cycle of the plate.

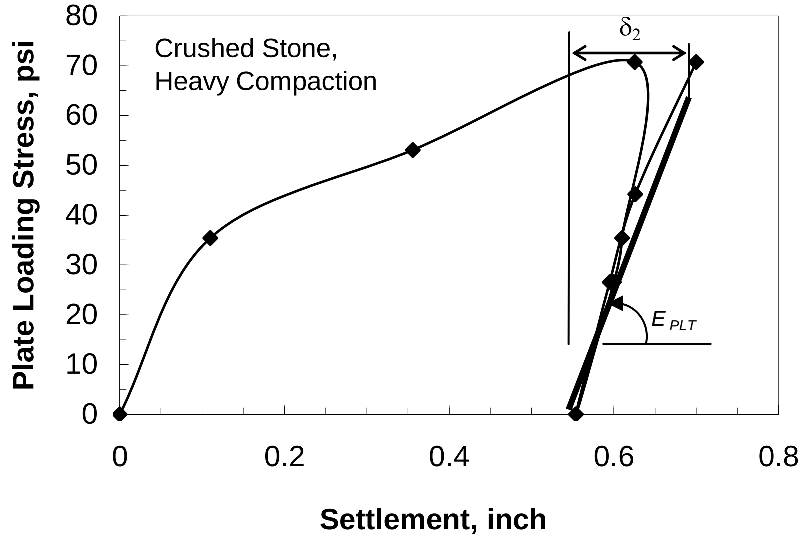


Figure 10
A loading and reloading curve of PLT for crushed stone

FWD. FWD is another trailer-mounted device that delivers an impulse load to the surface of pavement structures under investigation. The equipment automatically lifts a weight to a given height. The weight is dropped onto a 12 inch circular load plate with a thin rubber pad mounted underneath. A load cell measures the force or load applied to the pavement under the plate. Seven sensors measure the deflections caused by the impulse load. The first sensor is always mounted in the center of the load plate while sensors two through seven are spaced at various distances of 12, 18, 24, 36, 48, and 60 inches from the load center. Changing the mass of the falling weight and/or the drop height varies the impulse load. The DYNATEST 8002 FWD was used in this study, and the computer program ELMOD4 was used to back-calculate the resilient modulus.

Field Cross-Drain Construction Test Sections

The main purpose of the field cross-drain test sections was to investigate how and why weak areas formed in trench backfill during cross-drain construction and what measures should be taken to prevent this from occurring in the future.

Construction Projects

Four construction projects were selected to accommodate cross-drain trench backfill test sections with the help of LADOTD District 61. They are LA 964 with 4 trenches using sand as backfill; US 61 with 6 trenches using Kentucky crushed limestone, sand, and gravel sand mixture as backfill; LA 73/74 with 5 trenches using selected soil, sand, and Mexican limestone as backfill; and LA 10 with 4 trenches using RAP as backfill. Table 3 summarizes these field trench backfill test sections.

Table 3
Summary of backfill testing sections

Construction Project	No. of Trenches	Backfill Material
019-30-0015 (LA 964)	4	Sand (Compaction)
019-05-0026 (US 61)	4	Kentucky crushed limestone (Compaction)
	2	Sand (flooding)
	1	Bedding material (30% sand, 70% gravel, compaction)
077-02-0013 (LA 73)	3	Selected soil PI < 10
	1	Mexican crushed limestone
	1	Sand (compaction)
061-05-0044 (LA 10)	4	RAP

The field test sections checked the suitability of different materials as trench backfills and evaluated different means of quality control. It also provided a unique opportunity for the principal investigator to observe the cross-drain construction process and related problems under different construction environments. The DCP was the main tool to evaluate the quality of trench backfill in conjunction with the number of compaction passes and regular nuclear gauge readings (dry density and moisture content) specified by LADOTD's construction specifications.

Backfill Material

The material types tested in the field construction were expanded to include Mexican limestone, Selected soil, and bedding material (sand gravel mixture). All backfill materials were compared from the following aspects: ease to compact, ease to adjust moisture, maximum field dry density, seepage resistance, etc. A good backfill material must perform well not only in a normal construction environment but also under some unfavorable situations. These situations include construction delay, maintaining traffic, inclement weather, high moisture content of backfill for compaction, poor drainage conditions in the trench, etc. A poorly backfilled trench most likely comes out from one of such unfavorable construction environments.

Field Compaction Equipment

The type of compaction machines and the number of passes determine the compaction effort in the field. Figure 11 shows the equipment compared in field compaction under various conditions.

Wacker Packer   Vibratory Roller   Vibratory Plate

Figure 11
Compaction equipment

Flooding Method for Sand Backfill

The flooding method for sand backfill was used on one test section of US 61 to evaluate its feasibility. The method required that the compacted thickness of the first layer of backfill shall be equal to ½ the outside diameter of the conduit, but not exceeding a compacted thickness of 3 feet. The remainder of backfill shall be placed in layers not exceeding 3 feet compacted thickness. During placement, backfill materials shall be thoroughly saturated with water and satisfactory drainage of backfill shall be provided. Each layer of backfill shall be compacted to at least 95 percent of maximum density by approved mechanical compaction equipment prior to placing a subsequent layer.

Traffic Loading Measurement

Traffic loading may cause pavement surface dips. Moving truckload tests were therefore conducted at the test sections of US 61, LA 964, LA 73/74, and LA 10 to understand this factor. Pressure gauges (Geokon Model 3500-1-100) were embedded within the different depths of cover layers, as shown in figures 12 and 13. The 20-ton LTRC cone truck, as shown in figure 14, was used as a slowly moving load (3-5 miles/hour) through its rear dual tandem axles with a total weight of 30 kips.

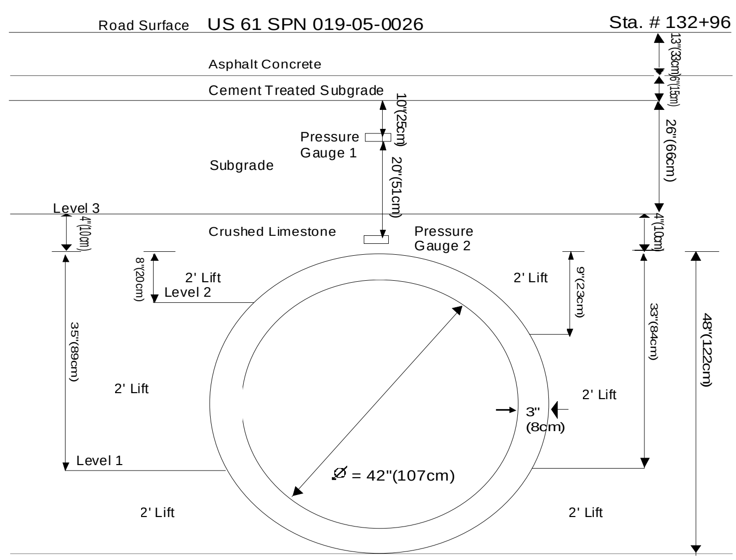


Figure 12
Cross section of trench with pressure gauges at US 61

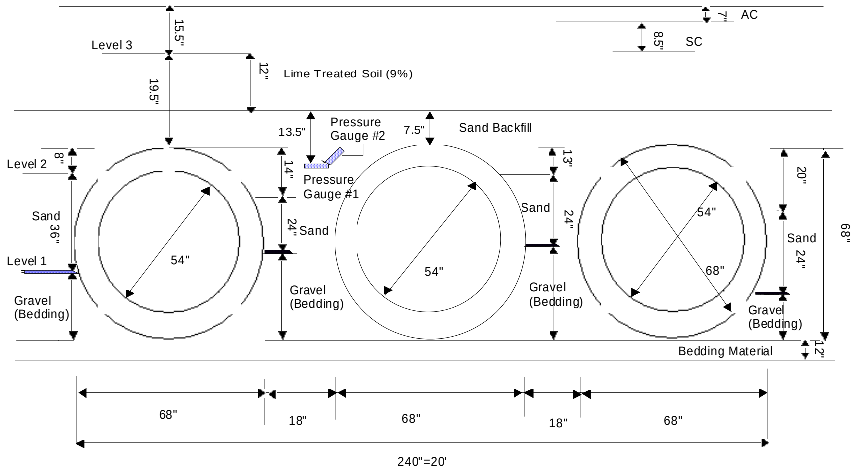


Figure 13
Cross section of trench with pressure guages at LA 964

Figure 14
Twenty-ton LTRC cone truck

DISCUSSION OF RESULTS

Laboratory Evaluation

Basic Characteristics of Backfill Materials Tested

Material types and compaction procedures control the stiffness of trench backfills while materials with different gradations will have different dry densities and stiffness for same compaction effort. Figure 15 shows the gradations of the three materials used in the full-scale compaction test at the PRF site. The shaded area in this figure indicates the current allowance specified by LADOTD for granular material used as trench backfill. The sand shown in the figure has the uniformity coefficient, C_u, of 2.7 and the coefficient of gradation, C_C, of 0.87. Generally, the sand specified by the current specification is difficult to compact and reach the required densities due to its poor gradation.

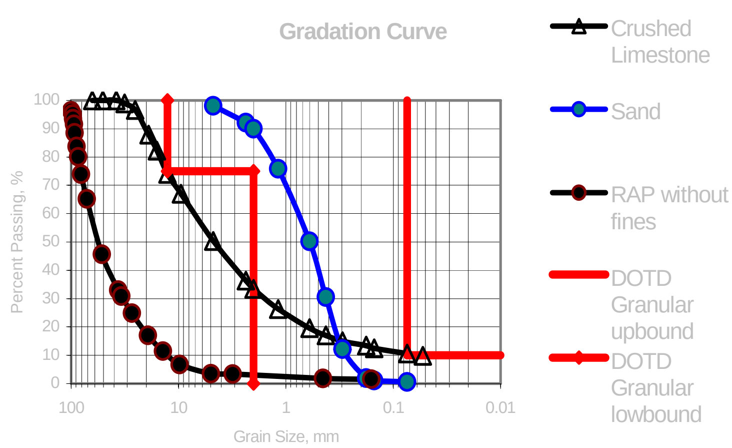


Figure 15
Louisiana DOTD specification on granular material

Figure 16 shows the sample gradation of backfill materials used in the field trench backfill construction test sections. Sand, crushed limestone, RAP, bedding material (sand gravel mixture), and Selected Soil with PI less than 10 were selected to test with the consideration of their availability, costs, past experiences, and expected field performance. According to the Louisiana Standard Specifications for Roads and Bridges (2000 edition), "Selected soils are natural soils with a maximum PI of 20, maximum liquid limit of 35, a maximum organic content of 5 percent, and a maximum silt content of 65 percent."

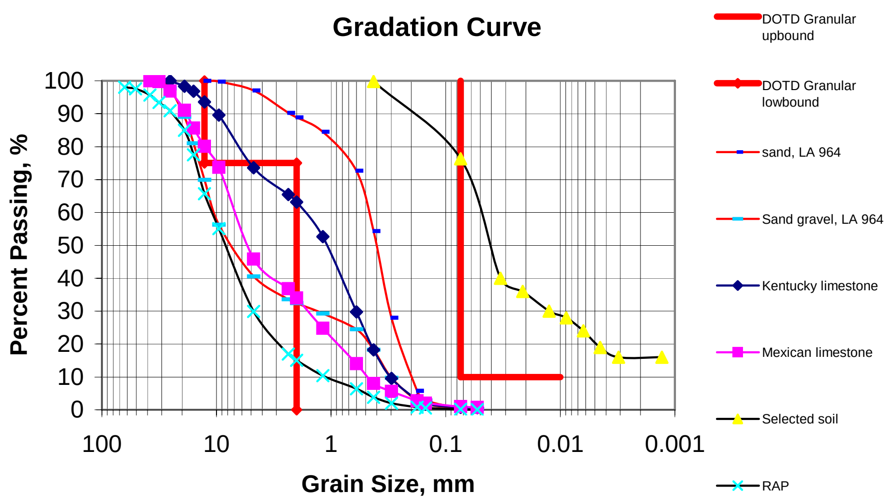


Figure 16
Gradation of backfills in field testing sections

Table 4 shows the ranges of the effective size, D_{10}, D_{30}, D_{60}; Uniformity Coefficient, C_u; and Coefficient of Gradation, C_c of these materials due to the variation of materials used as backfill in the field.

Table 4
Gradation characteristics of backfill materials

Material	Effective size D_{10} (mm)	D_{30} (mm)	D_{60} (mm)	Uniformity Coefficient C_u	Coefficient of Gradation C_c
Sand	0.17-0.3	0.32-0.43	0.46-0.78	1.8-3	0.79-1.36
Kentucky limestone	0.07-0.3	0.6-1.7	1.7-6	5.67-85.7	0.71-6.88
Mexican limestone	0.48	1.7	7.0	14.6	0.86
Bedding (Sand gravel)	0.3-0.33	0.65-2.3	4-12	13.3-40	0.35-2.29
RAP	0.3-1.2	1.6-4.8	6.4-12.5	10.4-21.3	0.76-1.53
Selected soil		0.014	0.048		

Table 5 shows the ranges of maximum dry densities with their working moisture ranges from standard and modified Proctor tests in the laboratory. Here, working moisture ranges, instead of optimum moistures, are used due to the variation of materials' properties and poorly defined compaction curves of granular materials. The last column of the table present field compaction observations and will be further discussed.

Table 5
Compaction characteristics of backfill

Material	Maximum Dry Density pcf		Working Moisture Range, %		Compaction in field
	Standard Proctor	Modified Proctor	Standard Proctor	Modified Proctor	
Sand	105 - 107	107 - 109	4 – 7	4 - 7	Difficult
Kentucky limestone	135 - 139	144 - 146	5 – 7	4 - 6	Very easy
Mexican limestone	116 -121	127-129	8 – 12	8-10	Very easy
Bedding (Sand gravel)	125 - 128	132 - 134	5 – 8	5 - 8	Not available
RAP	102 - 104	110 -112	5 – 9	5 - 9	Easy
Selected soil	106 - 109	112 - 115	15 – 18	13 – 16	Not easy

Material Dry-out

Figure 17 shows the dry-out results from the laboratory test, which established the correlation of moisture content with the time needed to dry the materials to their optimum (working) moisture contents. It indicates that except for the Selected soil, the time needed for the change of unit moisture content is similar for the remaining five materials. The major difference among the six materials is the range of natural moisture contents, as shown in table 6, that possibly exist in the field without extra water. The range is defined as the difference of the maximum moisture content of each material, also shown in table 6, with its optimum (working) moisture content. The maximum moisture content, which represents the maximum natural moisture condition of the materials stockpiled in the field, was determined by allowing fully saturated material to drain to remove all free water in the material. Table 6 indicates that limestone has the least possible moisture range to deal with in the field, which is one advantage needed in field construction. Although the results are from the laboratory conditions, they can be a useful reference to predict field situations.

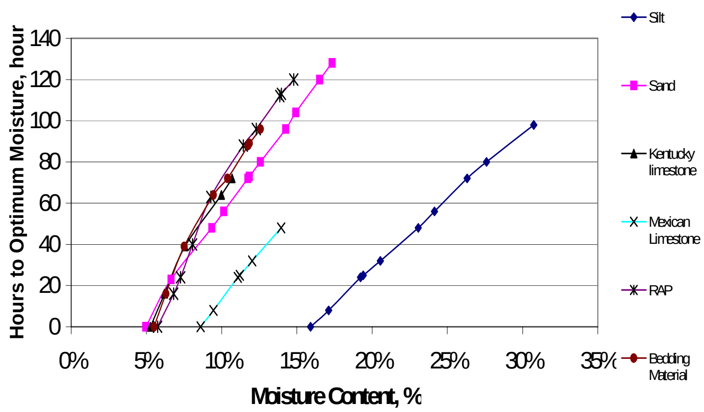


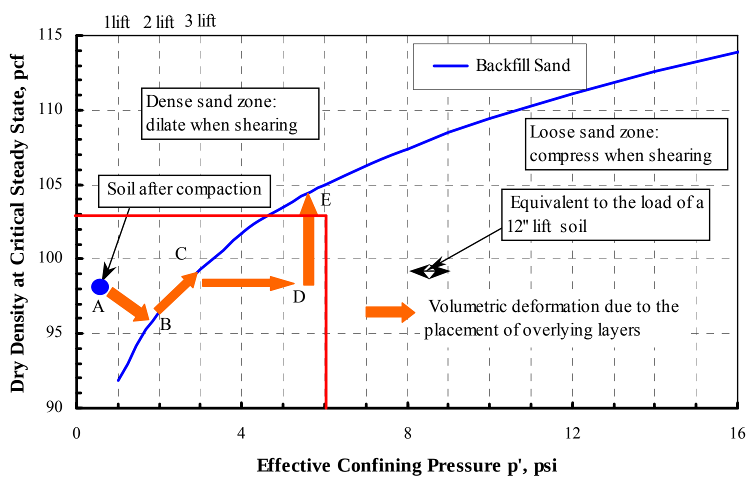
Figure 17
Dry-out results of different backfill materials

Table 6
Characteristics of material dry-out

Material	Optimum Moisture Content, %	Maximum Moisture without water flow-out, %	Possible Field Moisture, %	Moisture Range without Adding water, %
Sand	5	17	< 17	12
Kentucky limestone	5.5	11	< 11	5.5
Mexican limestone	8.5	14	< 14	5.5
Bedding (Sand gravel)	5.5	15	< 15	9.5
RAP	6	15	< 15	9
Selected soil	16	31	< 31	15

Settlement in Sand Backfill

Figure 18 shows the correlation between confining stress and critical dry density of the sand normally used in trench backfill in Louisiana. This correlation is based on the results from the triaxial tests on the sand. The curve divides the chart into two parts – the upper part is a dilating area and the lower part is a contracting area. Sand will expand if its dry density is larger than its critical dry density given by its confining stress; it will contract if its dry density is smaller than the critical dry density.



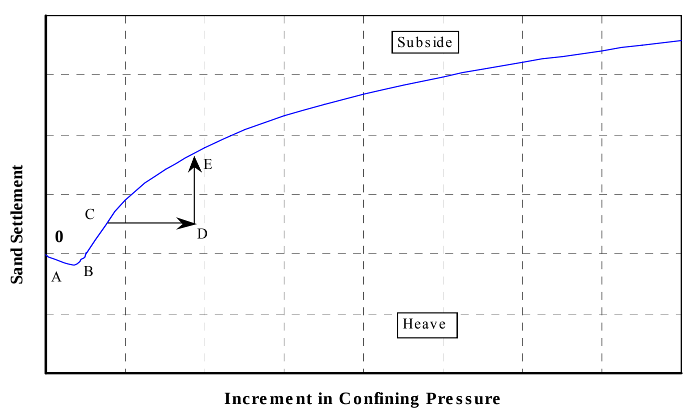


Figure 18
Illustration of sand deformation

Other research has reported that “static pressure is relatively ineffective in reducing the volume of a sand; for example, it is not possible to change a loose sand into a dense sand by static pressure alone. What is shown here is that it is possible to change dense sand into a ‘loose’ sand by static pressure alone” (Wood 1990) [21]. Therefore, the volumetric change of sand backfill during construction can be explained using the relationship shown in figure 18.

The maximum dry density that a sand backfill lift achieves under the compaction, noted as point A in figure 18, is controlled by the compaction energy the sand absorbs during the compaction. As more lifts are added to the sand, both the confining and deviatoric stresses applied to the sand increase due to the additional weight so that the sand will be sheared to dilate to point B. The sand backfill will then be densified during subgrade compaction when the influence of moving (dynamic) compaction can still reach the sand. So the physical status of sand will move from point B along the critical curve to point C. After that, the dry density of the sand will remain constant although its confining stress increases from point C to D, as shown in figure 18, since sand will not be densified under static pressure. Therefore, when the subgrade is finished, the sand is in a loose condition with respect to its confining stress level.

During the paving process of construction, an asphalt paver and vibratory roller will exert a strong vibratory compaction on pavement layers to achieve required densities. If the cover layer of sand backfill is not thick enough to absorb all the vibratory energy of compaction, the backfill will start to contract under the dynamic shearing and the status of sand will move from D to the direction of E, as also shown in figure 18, accompanied by an intolerable subgrade settlement. This has been reported in the cases of paving SUPERPAVE mixes.

Curves similar to the one shown in figure 18 can also be generated for crushed limestone, bedding material, and other aggregates in a Material Testing System (MTS).

Flowable Fill and Cement Sand Mixture

Table 7 shows the compositions of both cement and fly ash used in this study; the gradation of sand used is very similar to that shown in figure 15. Table 8 shows the mixtures designed for trench backfill application. For each mix, bleeding and segregation were all controlled to a low level to obtain homogeneous materials for various tests.

Table 7
Composition of Portland cement and fly ash

Composition (%)	Cement	Fly Ash
SiO_2	21.4	47.5
Al_2O_3	4.6	20.6
Fe_2O_3	2.7	5.2
CaO	64.0	16.2
MgO	2.0	2.5
K_2O	0.2	0.7
Na_2O	0.6	0.3
SO_3	2.5	0.7
C_3S	54.4	-
C_2S	17	-
C_3A	5.7	-
C_4AF	7.3	-
Specific Surface area (m^2/kg)	380	350

Table 8
Test CLSM mixture design (1.0 yd^3)

Specimen No.	#1	#2	#3	#4	#5	#6
Type I cement, lbs	35.1	55.08	75.06	99.9	88.29	75.06
Type C fly ash, lbs	374.49	364.5	354.51	342.09	417.69	354.51
Sand, lbs	2552.31	2502.9	2486.43	2446.74	2924.1	2486.43
Water, lbs	386.37	396.9	403.11	415.26	473.85	403.11
D1000 AEA, oz*	12.15	12.69	12.96	13.23	-	12.96
Accelerator, oz	-	-	-	-	-	33.93
Cement content, %	1.2	1.92	2.63	3.56	2.64	2.6

Plastic Properties

The important plastic properties of flowable fill are flow consistency (flowability), subsidence, and hardening time. Table 9 shows the test results of the recipes listed in table 8. Flow consistency tests were performed with both the 3×6 inch-open-ended cylinder flow consistency test and the standard concrete slump cone test. Figure 19 shows the open-ended cylinder flow consistency test. Both methods indicated a reasonably high flow consistency for the mixtures selected, as shown in table 9.

Table 9
The plastic properties of the mixtures

Sample	Flowability (in.)		Subsidence (%)	Hardening Time (h:m)
	3×6 in open ended cylinder	Concrete slump cone		
#1	9.875	24.313	2.18	12:30
#2	11.750	33.250	2.04	11:15
#3	11.438	27.875	1.91	10:00
#4	12.438	29.875	1.74	9:45
#5	10.000	17.813	0.43	8:15
#6	11.188	28.688	1.47	2:30

Figure 19
3×6 inches-open-ended cylinder flow consistency test

In general, the flow consistency of flowable fill is controlled by the content of fly ash, water, and air-entraining admixture (not necessary in this order). According to the criterion in ASTM 6103, the variation of these components within the six recipes tested did not cause a dramatic change in their flow consistency, and all recipes can serve as trench backfill. Compared to the other 5 mixtures, mixture number 5 had no air-entrained admixture and had more fly ash and water. Table 9 also indicates a sizeable difference in the flow consistency between open-ended cylinder and concrete slum cone tests.

Adding air-entrained admixtures to flowable fills will, in general, improve flow consistency of flowable fill, but will increase the subsidence as shown in Table 9. Subsidence is defined as the reduction in volume of flowable fill as it releases its water and entrapped air through the consolidation of mixtures. No-air-entrained mixture number 5 showed a low subsidence of 0.43 percent, approximately 5 times lower than that of air-entrained mixtures. The addition of accelerator quickened the hardening of flowable fill, as indicated by the mixture number 6. Adding a small amount of accelerator shortened the hardening time of mixture number 6 to 2.5 hours while the mixtures without accelerating agent hardened about 8 to 10 hours.

The cement content also has some effects on subsidence and hardening time. As listed in table 9, as the cement content increased from 35 lb/yd^3 to 100 lb/yd^3, as corresponding to the mixtures numbers 1 to 4, the subsidence decreased from 2.18 percent to 1.74 percent, and the hardening time shortened from 14 hours to about 10 hours.

In-Service Properties

The compressive strength of flowable fill mixtures after 1-, 7- and 28- day curing is shown in table 10. As usual, the strengths increased with curing time or cement content, which is defined as the ratio of weight of cement to the total weight of other solid components. Increasing cement content also decreased permeability. The addition of air-entraining admixture led to a higher air content of mixtures. As compared to those with the air-entraining mixture, mixture number 5 without air entraining admixture obtained a higher strength, with a higher unit weight and lower permeability.

For comparison purposes, two embankment soils allowed by LADOTD specifications were tested for their unconfined compressive strength (UCS) and California Bearing Ratio (CBR). Table 11 shows the basic properties of the soils tested, and figure 20 presents their compaction curves. The UCS values at the optimum moisture and the maximum dry density (standard proctor) were 38 psi for Clay I and 50 psi for Clay II, with a CBR values of 12 for both the clays.

Table 10
The in-service properties of the mixtures

Mixture	Strength (psi) 1d	Strength (psi) 7d	Strength (psi) 28 d	Air content (%)	Unit weight (lbs/ft3)	Permeability coefficient at 28 d
#1	11.7	29.5	64.1	10.0	117.2	3.32×10^{-4} cm/sec
#2	15.2	75.4	113.7	10.0	123.6	6.62×10^{-5} cm/sec
#3	16.5	128.0	151.9	10.3	122.0	2.71×10^{-5} cm/sec
#4	16.3	160.0	210.3	9.0	125.6	4.67×10^{-6} cm/sec
#5	21.5	193.4	291.7	0.8	133.6	1.86×10^{-6} cm/sec
#6	34.3	80.6	101.3	8.0	125.2	7.43×10^{-6} cm/sec
Clay I	38				126.4	
Clay II	50				133.9	

Table 11
Physical indices of soils tested

Soil No.	Percent of Silt, %	Percent of Clay, %	LL, %	PI, %	Optimum Moisture Content, %	Group Index, GI	Soil Classification USCS/AASHTO
Clay I	64.5	27.5	34	12	17.5	11	CL/A-6
Clay II	30.6	27.9	37	22	13.5	10	CL/A-7

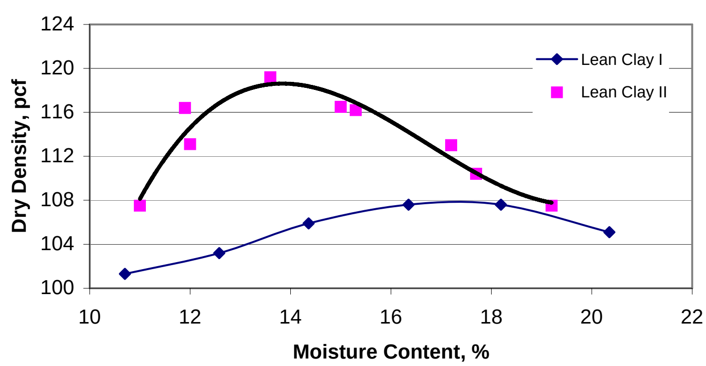


Figure 20
Compaction curves of soils tested

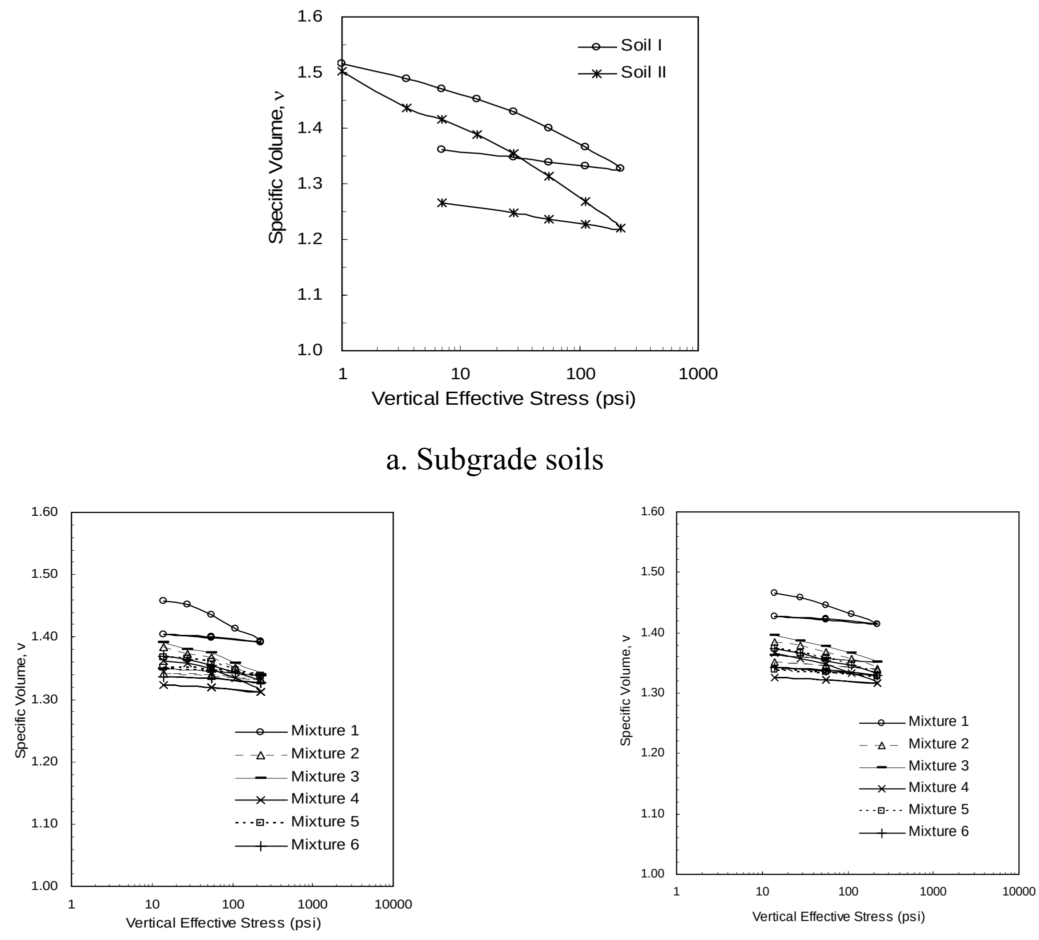
A comparison of the UCS of the flowable fill mixtures in table 10 with the one of the two soils indicate that mixture number 1 is not suitable as a normal backfill due to its low early strength. It took 28 days for its strength to be compatible with that of subgrade soils. Mixtures number 2 to 5 had a lower first day strengths but higher 7- and 28-day strengths. The 28-day strengths of mixtures number 3 to 5 are 3 to 4 times that of the well-compacted subgrade soils. Only the strength of mixture number 6 was closest to that of subgrade soils due to the addition of the strength accelerator. The addition of the accelerator increased the early strength, but the final strength (28 days) was lower. This is in agreement with the results of cement concrete.

The LADOTD flowable fill mix design shown in table 12 is close to the mixtures number 4 and 5. It can be predicted that the first day strength of this mix is lower than the one of a well compacted embankment or subgrade. Therefore, the use of heavy machinery should be avoided within the first day unless a strength accelerator is added.

Table 12
LA DOTD flowable fill mix design

Material	Quantity Per Cubic Yard (Cu m)
Portland Cement	100 lb (60 kg)
Fly Ash	250 lb (150 kg)
Sand	2800 lb (1660 kg)
Water	60 gal. (300 L) (max)
Cement content	3.28%

Figure 21 shows the consolidation test results from both the specification-allowed embankment soils and the six flowable mixtures after 1- and 28-day curing. The specific volume, v, defined as $1 + e$, is used in the chart. Here, e is the void ratio of the materials. Table 13 shows the initial conditions of the six mixtures and the embankment soils at the consolidation tests. Although the freshly made flowable mixtures with an air-entraining admixture had much higher air content than the mixture without air-entraining admixture did, as shown in table 10, the void ratios of hardened flowable fills were much lower, compared to embankment soils and well-compacted sand. This means that most extra air bubbles in the fresh mixtures evaporated during the placing and hardening process. Therefore, the air-entraining admixture had no detrimental effect on the stability of flowable fill, such as a collapsive settlement caused by extra air voids. The low void ratio of hardened flowable fills resulted from to the filling function of cement and fly ash, as well as their hydrated products. The lowest void ratio among the 6 mixtures was seen in mixture number 4, which had the highest cement content ratio of 3.56 percent, as shown in table 8. The cement content is defined as the ratio of cement weight to the total weight of other solid components.



b. Flowable fill, 1 day

c. Flowable fill, 28 days

Figure 21
Consolidation of subgrade soils and flowable fills

Table 13
Initial condition of flowable fill mixtures tested

Mixture number	1	2	3	4	5	6	Soil I	Soil II	Well-compacted sand only
Dry density, pcf	121.0	119.1	118.5	133.1	120.4	120.4			105
Initial void ratio, *e*	0.368	0.390	0.398	0.268	0.376	0.375			0.57

Table 14 shows the compressibility coefficients of flowable fill mixtures and embankment soils used in the specific volume and ln p' domain. It indicates that the compressibility of flowable fills decreased as the curing time increased and was 2 or 3 times less than that of embankment soils in general.

Table 14
Compressibility coefficients of flowable fill and embankment soils

Mixtures	1-Day Curing		28-Day Curing	
	λ^{*}	κ^{**}	λ	κ
1	-0.02664	-0.00414	-0.01775	-0.00480
2	-0.02565	-0.00407	-0.01590	-0.00468
3	-0.02578	-0.00410	-0.01564	-0.00389
4	-0.01992	-0.00418	-0.01631	-0.00305
5	-0.01533	-0.00436	-0.01553	-0.00377
6	-0.01824	-0.00390	-0.01599	-0.00502
Soil I	-0.04179	-0.01034		
Soil II	-0.05660	-0.01313		

* : slope of normal compression line in v:ln$p$' plane;
**: slope of unloading-reloading line in v:ln$p$' plane.

Cement Sand Mixture

In addition to flowable fills, cement sand mixtures with different cement contents were tested for their CBR values in conjunction with the two embankment soils to explore the lower boundary of low-strength cement sand mixture. The tests used the same sand used for flowable fills. The cement sand mixtures were prepared at a moisture content of 15 percent, which was near saturated but had no free water seeping out.

Figure 22 shows the CBR results of both the subgrade soils at their maximum dry densities determined by the standard proctor and the cement sand mixture with 0, 2, 4, and 6 percent cement (type I). The CBR values for both soils were about 12, while the CBR value for clean sand (0 percent cement) was about 2. The CBR values of the cement sand mixtures with 2 to 6 percent cement increased with time. The cement sand mixtures with at least 4 percent cement will be as good as well-compacted subgrade soils after a 24-hour curing. No additives were added to the cement sand mixtures tested.

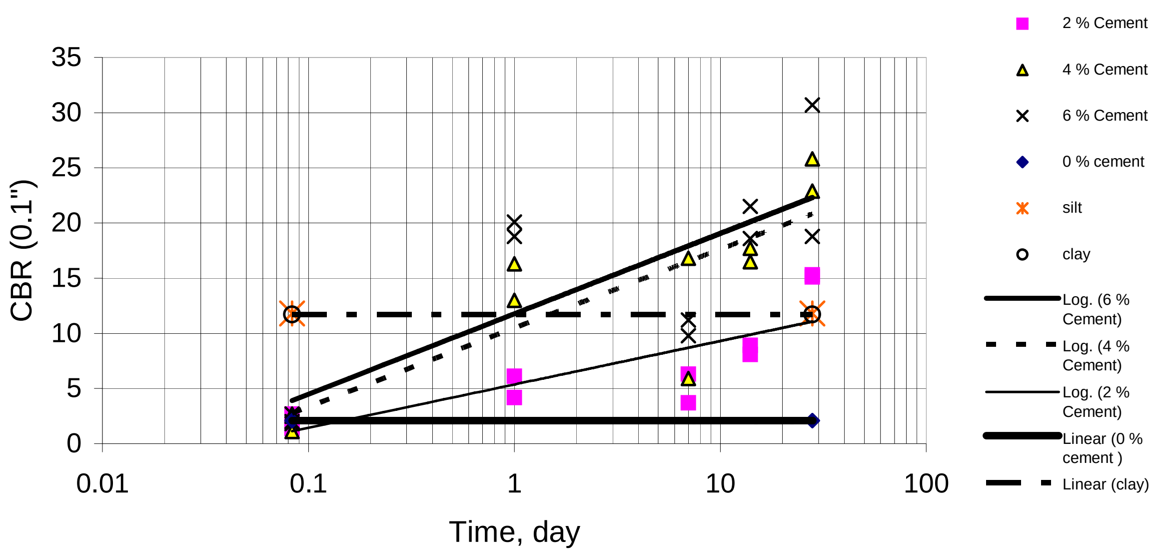


Figure 22
Illustration of sand deformation

Field Evaluation

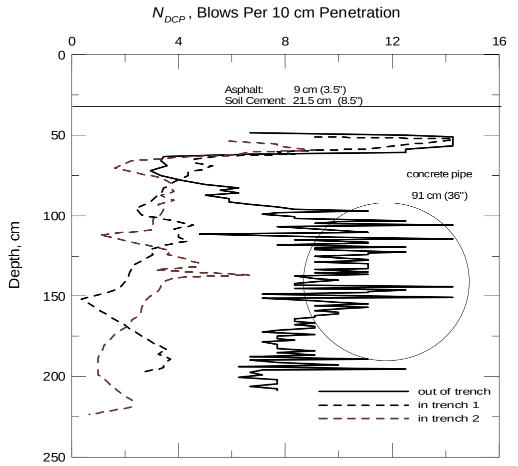
Evaluation of Existing Pavement at Cross-Drains

Twenty cross-drains with and without the surface dip problem under similar traffic and environmental conditions were tested in LADOTD's Districts 03, 08, 61, and 62.

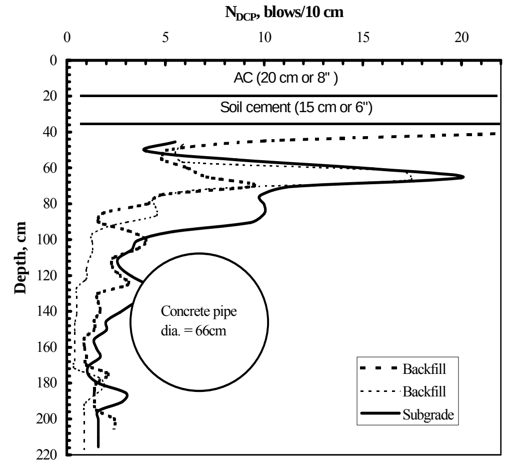
Settlement versus Non-Settlement

The first group of cross-drains used the sand as a trench backfill, and subsequent pavement "dips" developed. The DCP results from those locations indicated that when a pavement surface dip occurred at a cross-drain-pipe location, its trench backfill was usually much weaker than the adjacent subgrade soil. DCP profiles in figure 23-a illustrate this phenomenon. At a typical problematic location, the penetration blow count, N_{DCP}, was 5 blows per 4 inch penetration (20 mm/blow) in the backfill, and it was 9 blows per 4 inch (11 mm/blow) outside the trench. DYNAFLECT tests indicate a resilient modulus of 5,600 psi within the sand backfill and 6,900 psi in the subgrade soil out of the trench.

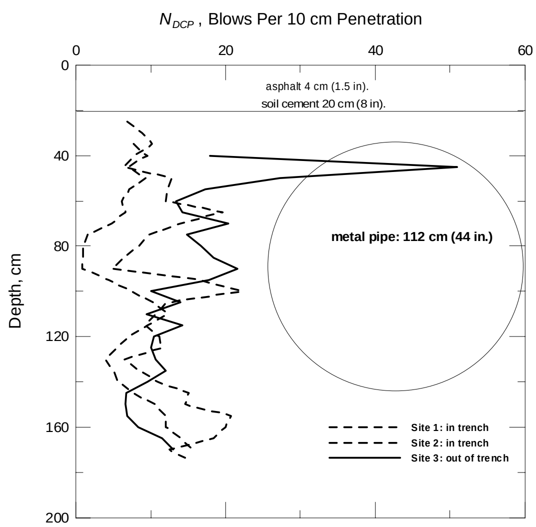
The second group of test locations also used the sand as a backfill material, yet no pavement "dips" developed. When a trench backfill was weaker than the adjacent subgrade soil, the occurrence of pavement surface dips depended on the stiffness of pavement structures and truck traffic loading on them. Figures 23-b and 23-c show two cases where the trench backfill was weaker than the adjacent subgrade soils, but no pavement surface dip occurred. For instance, in figure 23-c, the N_{DCP} value of the sand backfill at one side of the pipe was about 6 with a resilient modulus of 4,800 psi, while the N_{DCP} value at the other side was about 11 with a resilient modulus of 8,100 psi. The N_{DCP} value outside of the trench was about 17 with a resilient modulus of 11,000 psi. Even though the N_{DCP} values inside the trench were less than those outside the trench, pavement "dips" do not always occur. As shown in figure 23-b, this was attributed to the strong pavement structure that consisted of 8 inches asphalt and 6 inches soil cement. In figure 23-c, it was attributed to the observed light traffic.



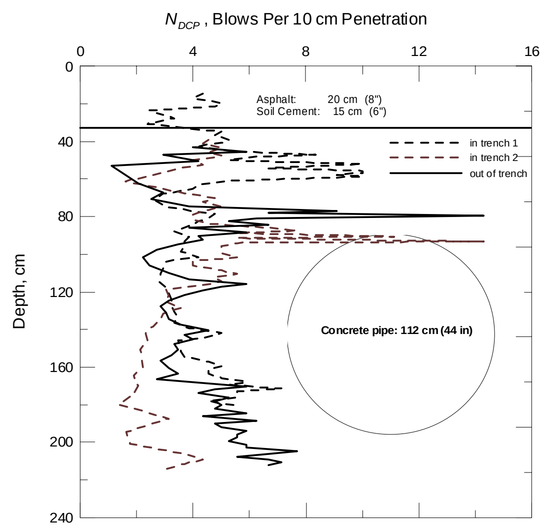
a. The Reduced DCP Profile with Pavement Settlement



b. A N_{DCP} Profile without Pavement Dip under Truck Traffic



c. A N_{DCP} Profile without Pavement Dip under No Truck Traffic



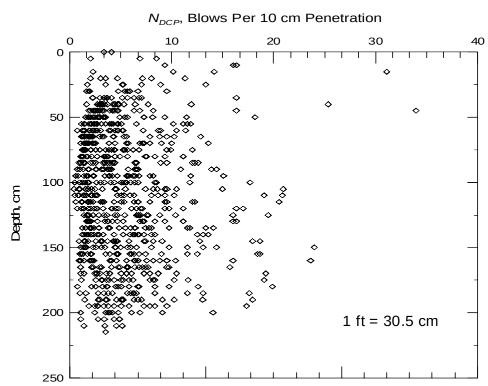
d. A N_{DCP} Profile without Pavement Dip under Truck Traffic

Figure 23
Typical field DCP profiles

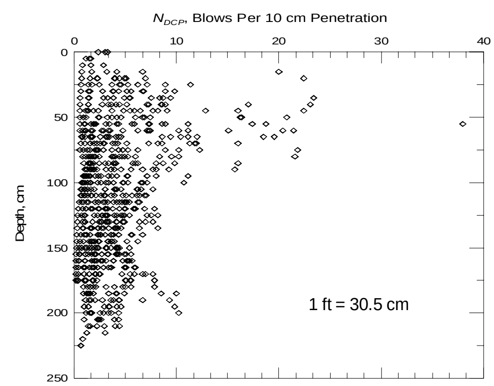
Figure 23-d shows the case where the DCP values within and outside of trenches were very close to each other and no differential settlement occurred in the trench backfill under heavy truck traffic. DYNAFLECT tests indicate a resilient modulus of 7,400 psi within the sand backfill and 7,000 psi in the subgrade soil out of the trench.

Subgrade Soils versus Sand Backfill

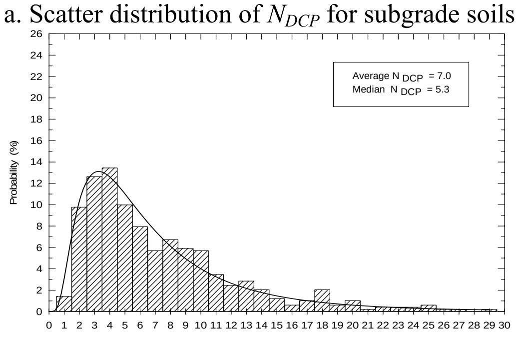
Figures 24-a and 24-b are the scatter distributions of N_{DCP} values with depth for native subgrade/embankment soils and the sand backfill, respectively. These two figures include all the data collected from native subgrade/embankment soils and the sand backfill material, regardless of pavement "dip" presence. They indicate that both subgrade/embankment soils and the sand backfill had a wide spectrum of variation with respect to N_{DCP} values. This variation is normal for native subgrade/embankment soils because of natural processes, but it is unexpected for the sand backfill since it was constructed under specifications designed to produce consistency. Statistically, the data in these two figures indicate that the sand backfill is generally weaker than the native subgrade/embankment soils, as shown in figures 24-c and 24-d.



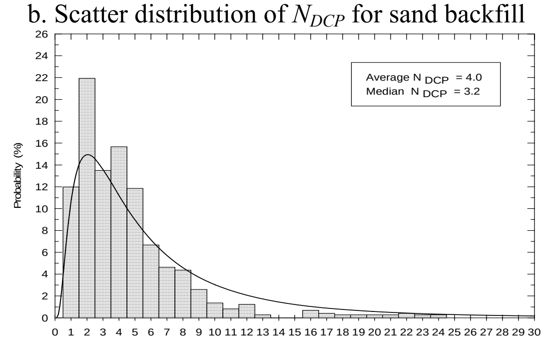
a. Scatter distribution of N_{DCP} for subgrade soils



b. Scatter distribution of N_{DCP} for sand backfill



c. Histogram of N_{DCP} for subgrade soils



d. Histogram of N_{DCP} for sand

Figure 24
Summary of field DCP data

Other Backfill Materials

The experience obtained with the sand backfill led to other backfill materials not specified by current LA DOTD specifications. These materials include RAP, Mexican Limestone, and washed gravel. Figure 25 shows the DCP test results from those materials.

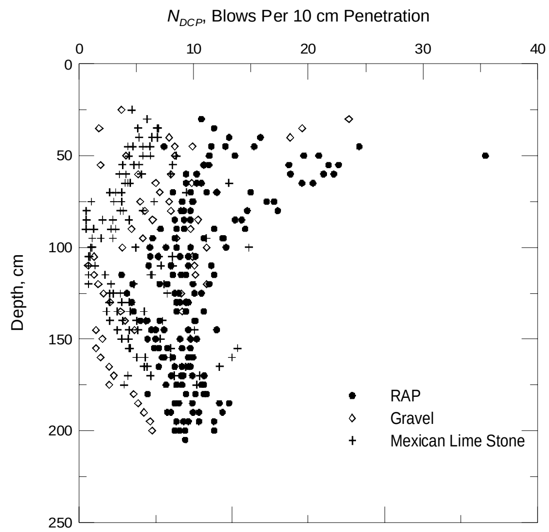


Figure 25
Scatter distributions of N_{DCP} for different materials

As to the flowable fill, DCP tests were conducted and no penetration was possible on this material.

Summary of Field Evaluation

Reason for Pavement Surface Dips. The field investigation of existing pavements indicated that pavement dips occurred at some cross-drain locations but not at others under the same traffic and environmental conditions. The DCP results from those locations indicated that when a pavement surface dip occurred at a cross-drain location, its trench backfill was much weaker than adjacent subgrade soil. On the other hand, when trench backfill was weaker than adjacent subgrade soil, the occurrence of pavement surface dips depended on the stiffness of pavement structures and truck traffic loading on them.

Criterion of Backfill Quality. In general, the quality control of highway cross-drain trench backfill serves two purposes: (1) guaranteeing a sufficient surrounding support to pipes for their stability and integrity; and (2) providing an adequate support to pavement structures over the cross-drain pipes. Since the first aspect is beyond the scope of this study, the quality control here mainly refers to the second aspect. A further discussion is conducted later in the field construction section to address why this second aspect of quality control has sometimes been missed so that pavement surface dips occurred at those cross-drain locations. The ultimate criterion for the quality control of cross-drain backfill is the stiffness compatibility of trench backfill materials to their adjacent subgrade soils so that no differential settlement occurs between them. This criterion is used throughout this study for the quality control, evaluation, and recommendation of trench backfill processes.

Hypothetically, if trench backfills could reach a stiffness corresponding to N_{DCP} of 10 blows per 10 centimeters (1 blow per centimeter) or larger, most pavement "dips" caused by backfill settlement could be prevented at those locations. This is because most native subgrade soils in Louisiana have N_{DCP} values less than 10 blows per 10 centimeters, as shown in figures 25-a and 25-c. This is consistent with the DCP data for subgrade soils obtained by the Minnesota Department of Transportation [17]. Pavement structures over trench backfills also have a function of "bridging" traffic loading over weaker areas. Though this "bridging" function is not fully understood at this time, pavement structures distribute traffic loading over a larger area, reducing the settlement in trench backfill due to lower loading stresses.

PRF Full-Scale Compaction Test

Table 15 summarizes the basic information for the three testing trenches. The average moisture content obtained from the nuclear gauge reading during the field compaction tests was 3.7 percent with a dry density of 99.8 to 106.6 pcf for the sand, 8.4 percent with a dry density of 98.0 to 111.6 pcf for the RAP, and 5.1 percent with a dry density of 117.2 to 130.8 pcf for the crushed stone.

Table 15
Summary of full-scale trench test information

Trench Number	Material	Section Number	Compaction Effort	Standard Proctor γ_d pcf (kN/m^3)	Field Moisture Content (%)	Field Dry Density γ_d pcf (KN/m^3)
1	Sand	1	Light*	104.1 (16.8)	3.7	99.8 (16.1)
		2	Medium**			106 (17.1)
		3	Heavy***			106.6 (17.2)
2	RAP	1	Light	114.1 (18.4)	8.4	98.0 (15.8)
		2	Medium			104.8 (16.9)
		3	Heavy			111.6 (18.0)
3	Crushed Limestone	1	Light	132.7 (21.4)	5.1	117.2 (18.9)
		2	Medium			118.4 (19.1)
		3	Heavy			130.8 (21.1)

* : One pass of vibratory plate compactor

** : Four passes of vibratory plate compactor

*** : Four passes of vibratory plate compactor + four passes of Wacker Packer compaction

DCP Test Result

DCP tests were conducted after the compaction of each lift during the backfill process. Figure 26 is an example of the DCP data after each lift of backfill. This figure shows how penetration blow count, N_{DCP}, increased with the additional lifts. Figure 27 shows all data presented in percent increase of average N_{DCP} values in each lift. The left part of the figure shows the data produced when the first lift had one overburden lift above it, where

$$\text{Percent Increase of } N_{DCP} = \frac{\text{average } N_{DCP} \text{ with one overburden lift}}{\text{average } N_{DCP} \text{ without overburden lift}} \tag{6}$$

The right part of the chart shows the data produced when the first lift had two overburden lifts above it, where

$$\text{Percent Increase of } N_{DCP} = \frac{\text{average } N_{DCP} \text{ with two overburden lifts}}{\text{average } N_{DCP} \text{ without overburden lift}} \tag{7}$$

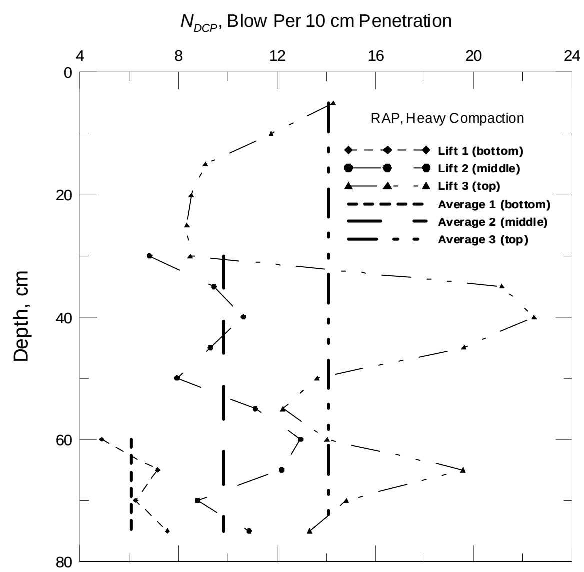


Figure 26
N_{DCP} profiles after each lift for RAP material

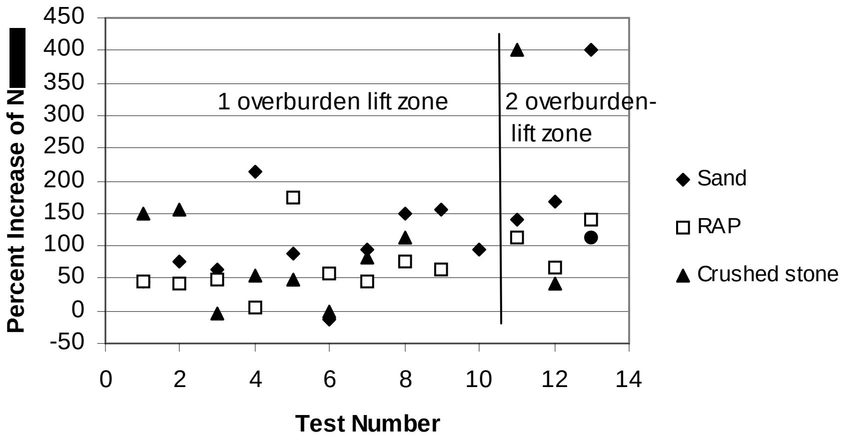


Figure 27
Percent increase of average N_{DCP} values in each lift

In general, the first lift's average N_{DCP} was affected more by the placement and compaction of the second lift (one overburden layer) than by the third lift (two overburden layers).

Figure 28 shows how the average N_{DCP} values correlate with backfill thicknesses for different materials. This figure indicates that the average N_{DCP} values increased with the increase of backfill thickness, mainly due to the overburden effect discussed earlier. This figure also shows that the sand was the least sensitive to compaction effort, next to RAP. The increase of average N_{DCP} values resulting from different compaction efforts for the sand was very limited due to its very poor gradation. On the other hand, increasing compaction effort can dramatically improve the stiffness of the RAP and the limestone, as shown in figure 28.

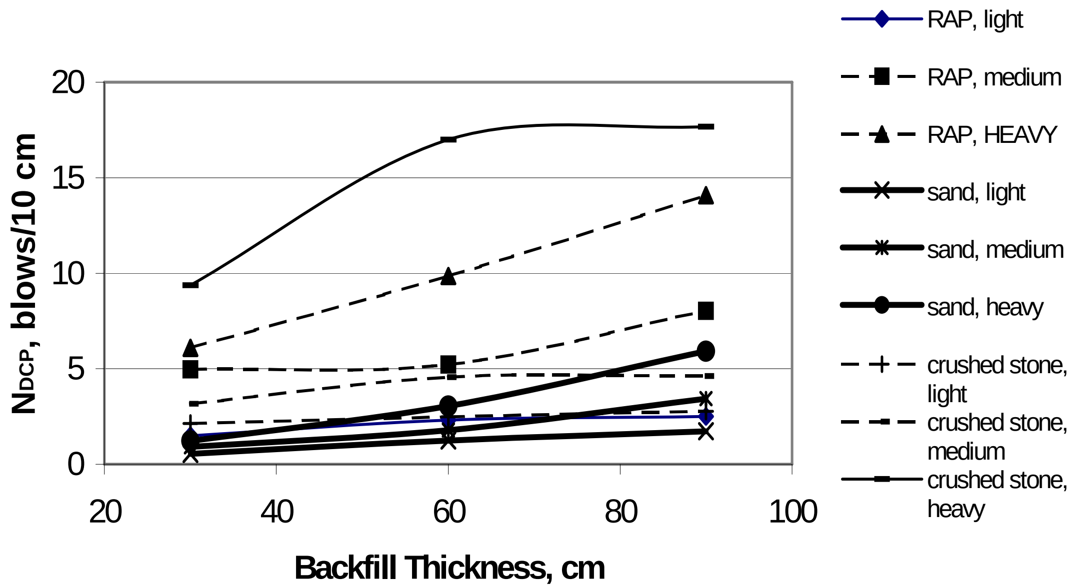


Figure 28
Correlation of average N_{DCP} values in a backfill with its thickness

Field Cross-Drain Construction Test Sections

The stiffness of trench backfills relative to their adjacent subgrade soils is a combined result of many factors involved during a construction process. These factors include in-situ construction conditions, quality inspection, contractors' workmanship, backfill materials and compaction, traffic loading during construction, configuration of pipes and trenches, etc. The observation in field construction indicates that poor backfill, in most cases, was related to one or more of the factors. If these factors are all favorable to trench backfill construction in ideal conditions, current LADOTD specifications [23] for trench backfill can provide satisfactory performance. This explains the fact that the majority of highway cross-drains function quite well in Louisiana. Unfortunately, this is not always the case in reality and many such trench backfill constructions were and will be conducted under unfavorable conditions, such as construction delay, or under traffic, poor weather, wrong backfill moisture content, or poor drainage condition of trench, etc. Different field construction conditions require different backfill material, equipment, and compaction procedures. Since this study aims to prevent all unexpected pavement surface dips at highway cross-drains, the solutions from this study should provide options to allow proper backfill of highway crossing-drains under different conditions, including those unfavorable and severe ones.

Influence Factors on Trench Backfill Quality

A trench backfill having a stiffness compatible (same or close to) with adjacent subgrade soils will reduce or eliminate the potential of differential settlement, and can support a pavement structure as seamlessly as adjacent subgrade soil does. The following factors help to meet this requirement.

Workmanship. Contractor workmanship is one of the most important factors in the quality control of cross-drain construction since it directly affects trench backfill quality. It reflects the contractors', or their employees', willingness to cooperate with project inspectors from LADOTD and to follow the department's construction specification on trench backfill. It is also affected by the construction environment and conditions. Most contractors hired by LADOTD are experienced with highway cross-drain construction. However, the contractor-LADOTD relationship may become tense when contractors are under pressure and challenge project inspectors' expertise and instructions on trench backfill procedure. This is especially true when construction is under traffic, weather is not cooperative, backfill material does not have the right moisture content, or the drainage condition of the trench is not good, etc. In such difficult situations, contractor cooperation is very important and valuable. Therefore, their perspective on the quality control of trench backfill should be taken into consideration. They should be one of the team players and also share the consequence of the quality of their construction, whether good or bad.

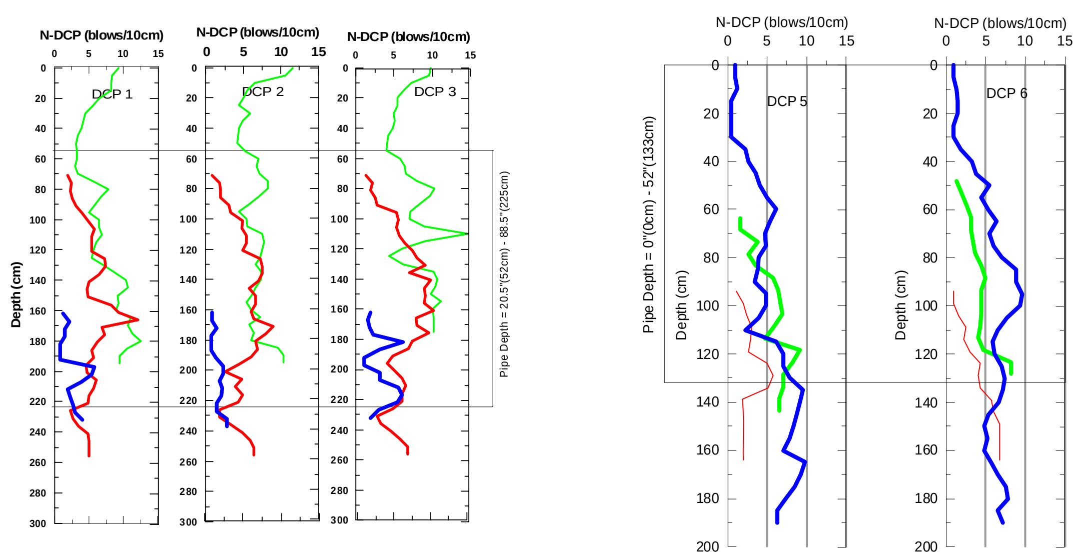
Backfill Material. Different backfill materials were evaluated through the aspects of compaction, moisture adjustment, and seepage stability.

<u>Compaction.</u> Compaction is the simplest way to increase the stability and load-bearing capacity of backfill materials in trenches. The effectiveness of compaction is

generally dependent upon the compaction effort, backfill material's gradation, and moisture content.

Equipment. The type of compaction machines and the number of passes determine the compaction efforts in the field. Among the equipment used in field compaction, the Wacker Packer is the most effective, followed by the vibratory roller, and the vibratory plate, as already shown in figure 11. This conclusion resulted from field DPC test results averaging over a 3-foot thickness. In the field compaction, none of the machines could achieve the maximum dry density larger than that determined by the standard Proctor testing in the laboratory for lifts equal to or larger than 12 inches, no matter how many passes were applied to the materials tested in this study.

Sand. The field maximum dry density for the sand was around 105 pcf (the nuclear gauge reading), with an average DCP value of 5 blows per 4 inches penetration for the well compacted work. Figure 29 shows the example of DCP profiles with good and poor compaction obtained in the field-testing sections. The thickness of a lift was 12 inches. As expected, the sand was not easy to compact to the required dry density.



(i). Good Compaction (ii). Poor Compaction

Figure 29
DCP profiles for the sand backfill

Flooding of Sand. The difficulty in the flooding method lies in setting up an effective drainage system in the trench. Theoretically, a designed filter layer is required around a perforated drain pipe embedded at the bottom of a trench, but this is impossible in trench backfill cases. The issue of effective drainage was handled according to contractor's personal experience and judgment. For example, at a cross-drain trench at station 298+50 of US Highway 61, a perforated pipe wrapped with geosynthetic fabric was embedded at the bottom of the trench to drain the flooding water. However, no flooding water from the sand went out through the pipe, and the flooded trench was not accessible for about 8 to 10 hours. Figure 30 shows the DCP Profiles from the flooding method of sand. The solid DCP profile curves in the figure were obtained 8 hours after flooding while the dashed line profiles were the results of DCP tests 6 days later when two 12 inch compacted subgrade lifts were placed on the backfill.

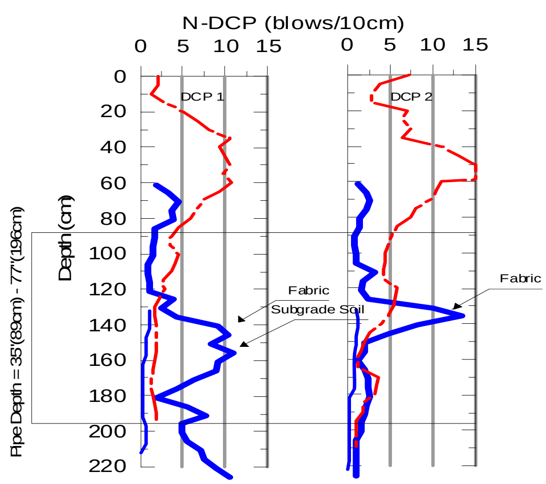
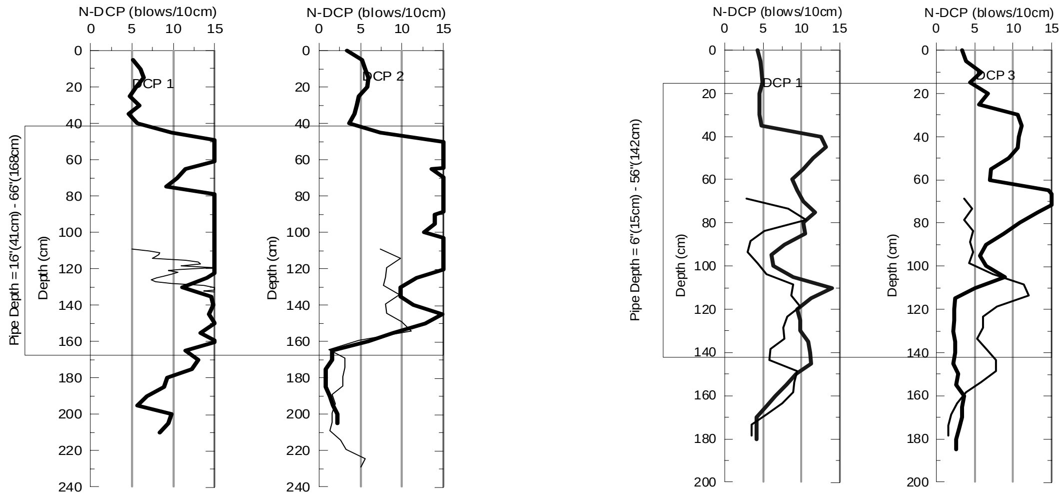


Figure 30
DCP profile of flooding sand

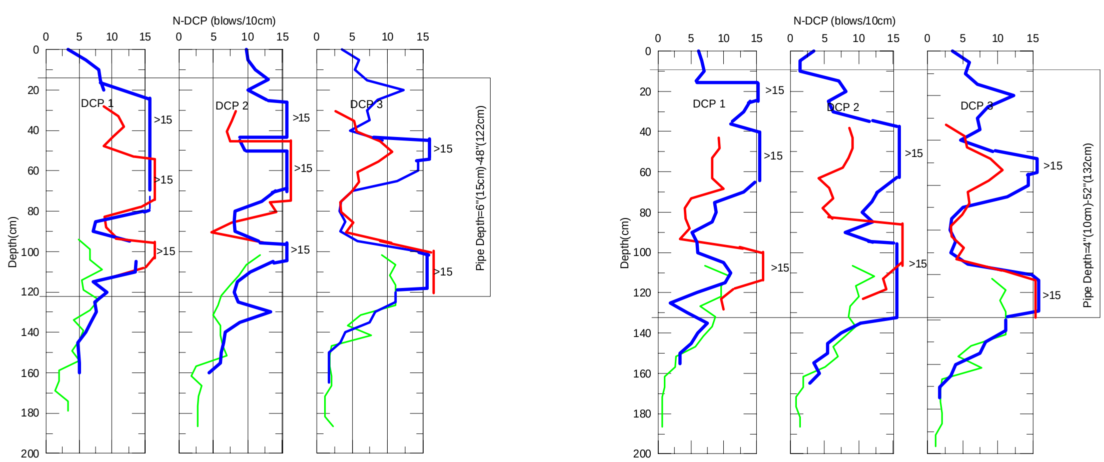
*RAP***.** RAP is a low- or no-cost material for LADOTD and its field compaction is much better than sand's. Five to eight passes of the Wacker Packer compaction will generally result in an average DCP value of 9 – 10 blows for 4 inches penetration. Figure 31 shows the example of DCP profiles with 12 inch- and 18 inch-lifts obtained from the field-testing sections.



(i). 12-inch Lift (ii). 18-inch Lift

Figure 31
DCP profiles for RAP backfill

Crushed Limestone. Crushed limestone is easy to compact to its field maximum dry density of 134 pcf. For the Kentucky limestone used in the field, three to five passes of the Wacker Packer compaction would achieve the desired results. Its average DCP blows for a 4 inch penetration was generally larger than 15. Figure 32 shows the example of DCP profiles with 1foot- and 2 foot-lifts obtained from the field-testing sections.



(i). 1-ft Lift (ii). 2-ft Lift

Figure 32
DCP profiles for Kentucky limestone

Selected Soils. The original test plan required the Selected soil with a PI between 8 and 12 to be used for the LA 73/74 project. However, the project contractor tried to locate the Selected soil for several months without success. Therefore, the soil's PI was allowed to be less than 12. The actual Selected soil used in the project had a PI of 0 to 7. It was very sensitive to moisture content in the field compaction. The field compaction did not reach the required dry density determined by the standard Proctor testing due to the soil's high moisture content since there was no drying time. Therefore, the average DCP value was about 2 – 3 blows for a 4 inch penetration. Figure 33 shows the example of DCP profiles obtained from the field-testing sections.

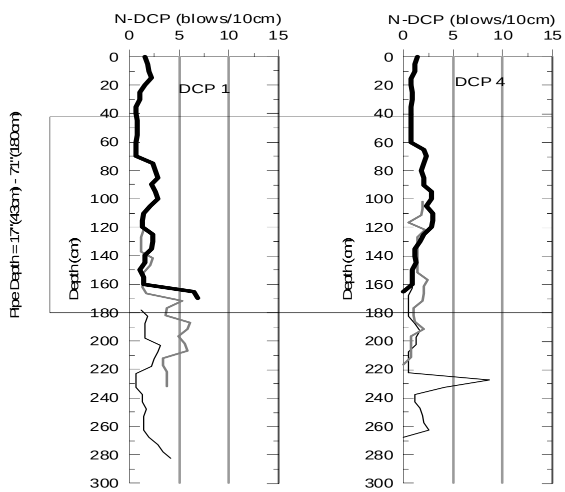


Figure 33
DCP profiles for selected soils

Bedding Material. Bedding material required by LADOTD specification normally consists of 30 percent to 35 percent sand and 65 percent to 70 percent gravel. Table 5 indicates that it can reach at least a maximum dry density of 125 pcf in the laboratory. According to its laboratory results, the bedding material should perform similarly to RAP and much better than the sand in field compaction. The field test on the material was not successful due to poor quality control on the lift thickness (> 2 ft) of the backfill. Figure 34 shows its field DCP profiles.

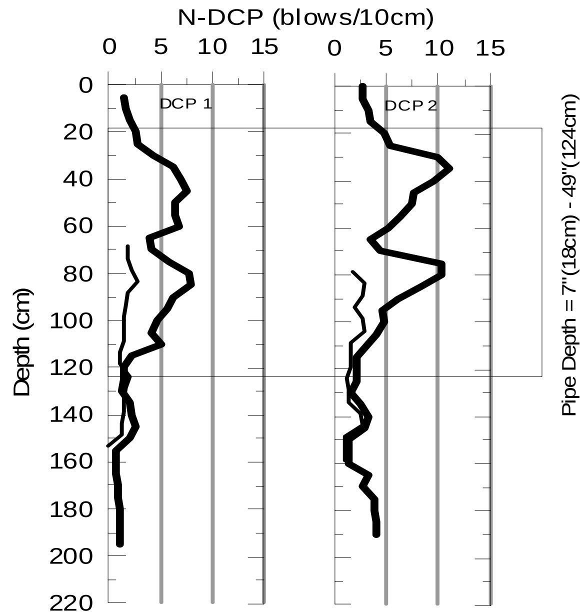


Figure 34
DCP profile of sand-gravel mixture

Flowable Fill. Flowable fill has many advantages compared to conventional backfill materials. Past experience with this material in Louisiana indicates that it is very strong (stiff), impenetrable by the DCP after setup, easy to be used in backfill construction, and needs no compaction, etc. It also takes time to set up unless some additives are used, as already shown in the laboratory test. As for construction, flowable fill requires no compaction, but extra anchorage is required to keep the alignment and grade of flexible pipes in place and maintain the integrity of joints when flowable fill is spread in haunch zones and sides of pipes. This minimum waiting period is also important for scheduling construction sequences if a flowable fill backfill has to be conducted in stages. A flowable fill will provide good support for pipes and seal any leakage at pipe joints. Field tests conducted in this study have already shown that no displacement could possibly occur after flowable fill is set. Compared with other backfill materials, flowable fill will also cost more due to a higher material cost (similar to lean cement concrete).

Material Moisture Adjustment. The material moisture adjustment was discussed previously in the laboratory evaluation. No field test was conducted on this issue.

Seepage Stability. Seepage stability occurs when ground surface water goes through the roadway not by cross-drain pipes but by the backfill of pipes, or when water leaks to the backfill from the joints of cross-drain pipes. The erosion caused by such seepage in backfill will be controlled by factors like hydraulic gradients and the resistance to seepage force caused by these gradients.

Under a certain hydraulic gradient, the resistance to erosion will depend upon dry density, cohesion, and particle interlock of backfill materials. Therefore, sand has the least seepage stability due to relatively low dry density, no cohesion, and a weak interlock among its particles. The seepage stability of the Selected soil with a low PI will be a little better than the sand due to its cohesion. RAP also has low dry density and a poor interlock among its particles, but it has high cohesion among particles due to the residual asphalt. Mexican limestone and bedding materials have higher dry densities. Kentucky limestone has the highest dry density and strongest interlock among particles. Kentucky limestone has the best seepage stability, followed by Mexican limestone, RAP, and the bedding material, then followed by the Selected soil and the sand.

The previous discussions reveal that crushed limestone is the best choice for a backfill material because it is easy to be compacted to a high density with a narrow range of field moisture content and good seepage stability.

Pipe Cover Layer. Pipe cover layer refers to the subgrade soil over pipes and measures from the top of a pipe to the bottom of a pavement base course. It is important because it is a buffer zone between a pavement structure and embedded pipes for construction and traffic loading. A thick and well-compacted cover layer will spread traffic loading well out of trench area. Figures 35 to 37 show the examples of well-compacted cover layers with 3-foot, 4.4-foot, and 7-foot thickness. The DCP profiles within and out-of trenches are very similar within those layers, indicating a consistency along the roadway over the cross-drain(s), so that no differential settlement occurs. In such cases, the cover layer's thickness controls the trench backfill's influence on the stability of pavement structures.

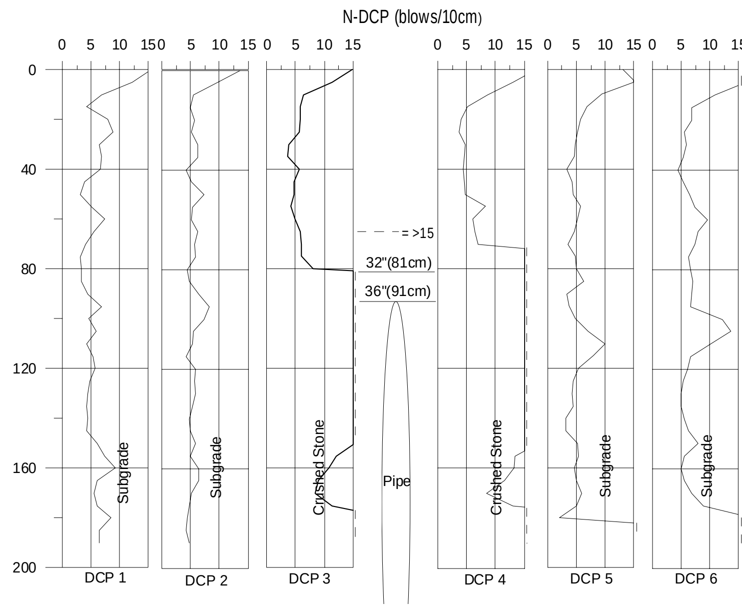


Figure 35
Three-fold cover layer

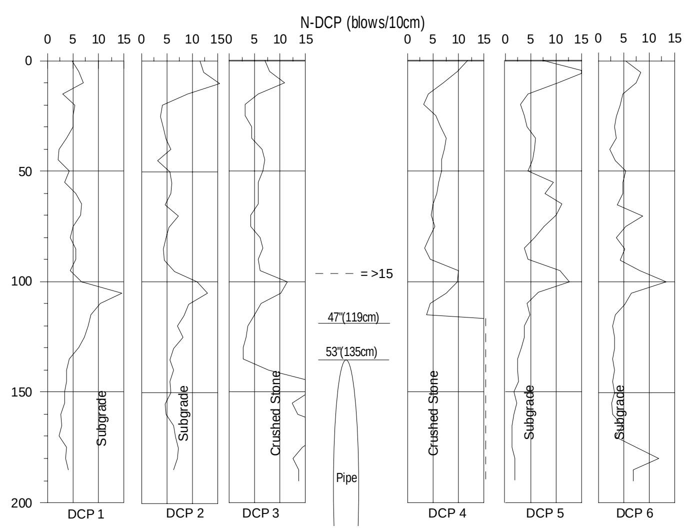


Figure 36
4.4-foot cover layer

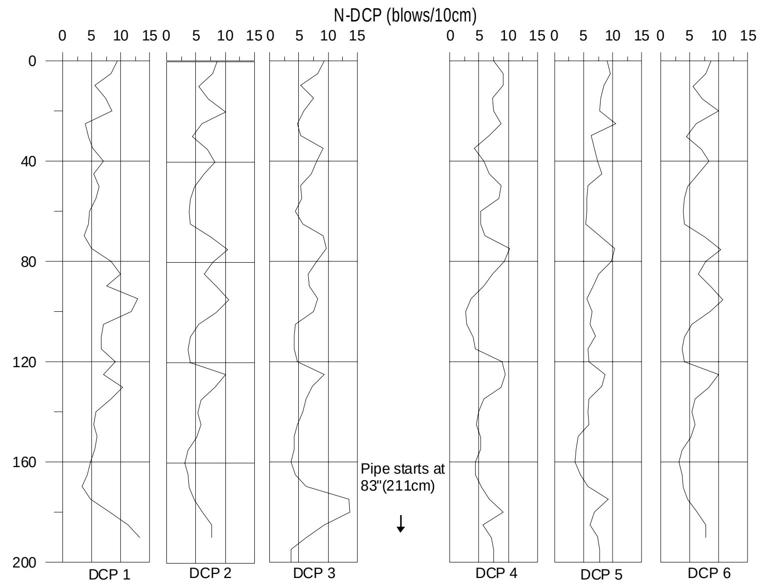


Figure 37
Seven-foot cover layer

Stress Condition**.** Construction traffic improves the stiffness of cover layers. When construction is under traffic (no detour road), the public traffic during the construction also densifies the trench backfill, as backfilled trenches with cover layers act as a temporary roadway during construction. Moving truckload tests were conducted in this study and figure 38 shows the applied load-time and measured load-time curve with an enlarged scale.

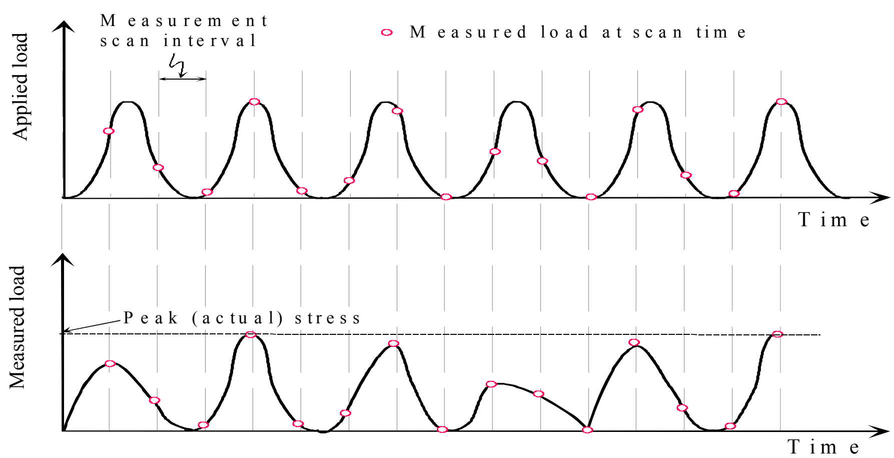


Figure 38
Illustration of applied load- and measured load-time curves (enlarged scale).

Figures 39 and 40 show the examples of maximum working vertical stresses responding to the moving load in the cover layers of trenches at US 61 and LA 964 testing sites. Traffic loading, pavement structure, and the thickness of the cover layer determine the stresses as shown in those figures. These two examples indicate that for the same traffic loading, a thicker subgrade cover layer resulted in less vertical stress (36 inches versus 25.5 inches; 4.2 psi versus 5.6 psi) as expected. As more pavement structures are added to the subgrade, the corresponding vertical stress becomes less and less. As a rule of thumb, a trench backfill takes about 10 percent construction traffic loading in magnitude when the thickness of the cover layer is 3 ft. Empirically, the compaction of pavement structures (subgrade and surface layers) will have a very small impact on the trench backfill during construction when the cover layer is thicker than 4 ft.

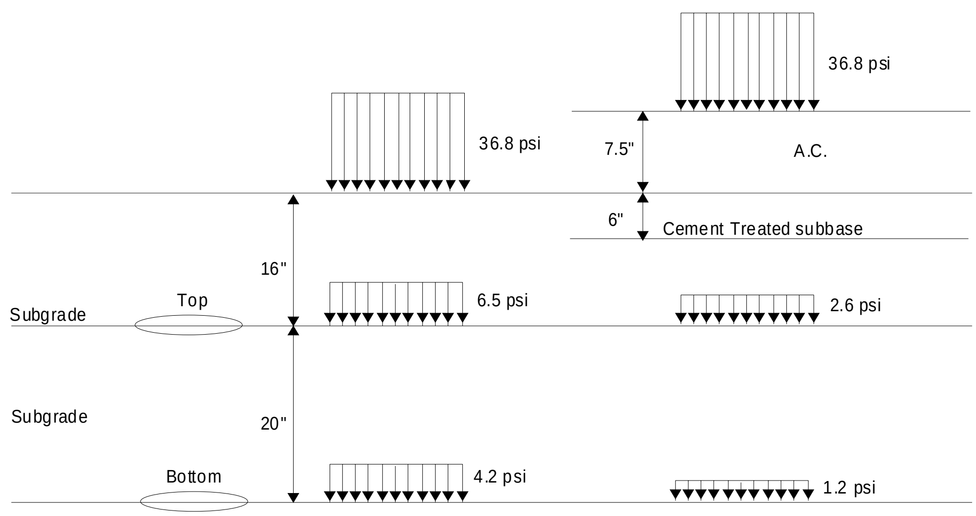


Figure 39
Variation of vertical stress in different construction stages at US 61

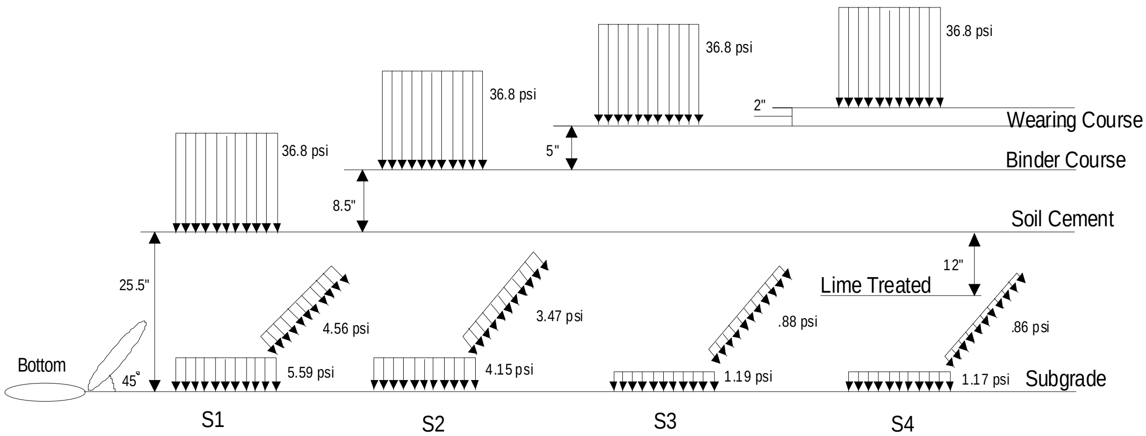


Figure 40
Variation of vertical stress in different construction stages at LA 964

Construction Traffic. Figure 41 shows the field setup for three pressure gauges to measure the loading stresses in the cover layer at a LA 73 test section. The first cell was 4 in. above the pipe. The second and third cells were about 26 in. above the first cell with one inclined and another horizontally oriented.

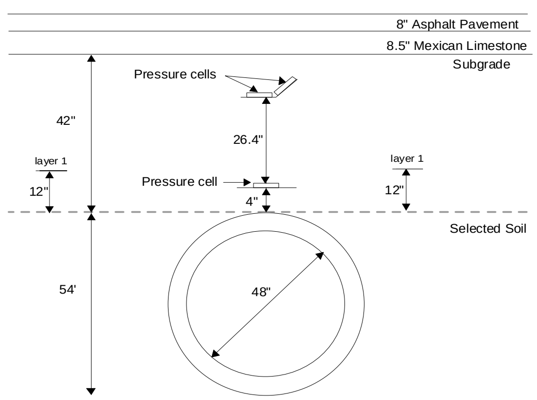


Figure 41
Pressure gauge setup at LA 73

Figure 42 shows a vibratory compaction roller working on a 4 in. asphalt binder course and figure 43 shows the recorded construction loading stresses caused by this roller when it was above those gauges. The stress was about 18 psi at the depth of 24 in. below the compaction surface, and was less than 4.5 psi at the depth of 50 in. below the surface. So as the thickness doubled, the working stress was reduced four times in the cover layer.

Figure 42
Construction traffic loading

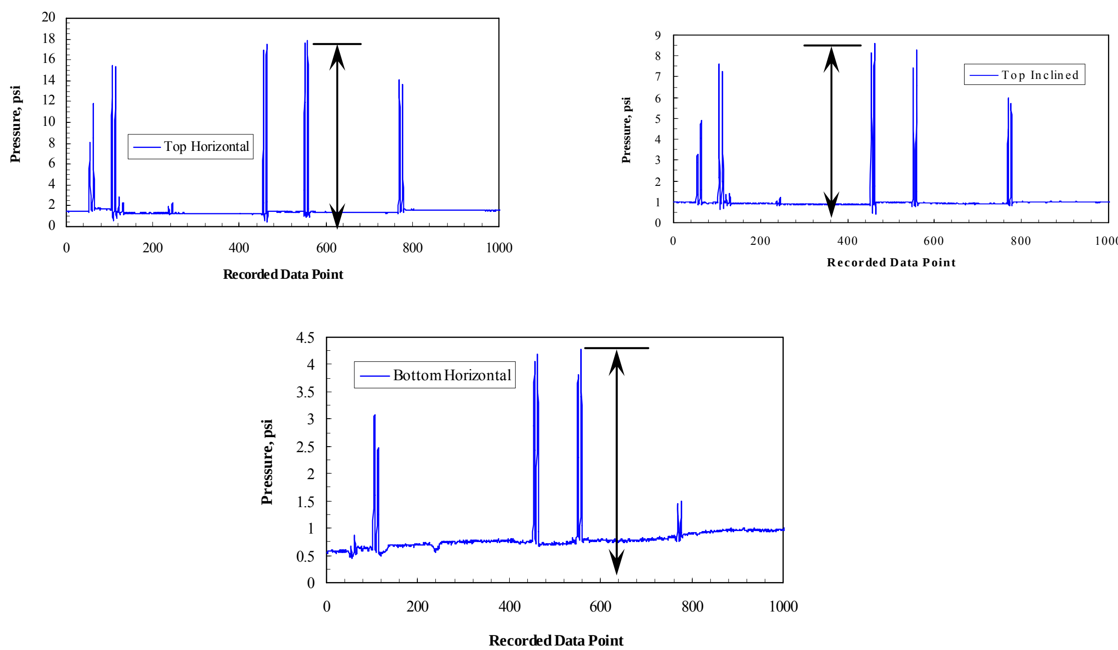


Figure 43
Construction stress caused by the compaction roller

Generally, construction machinery places a much higher stress level on a highway subgrade than does normal traffic. The subgrade should be strong enough to take the construction traffic loading. Otherwise, premature damage can occur undetected, which will weaken the pavement structure and shorten the pavement life. This is especially important for a subgrade with high moisture content and even for a lime- or cement- treated subgrade.

Configuration of Cross-Drain Trench. The configuration of a cross-drain trench can affect the stiffness compatibility of the trench backfill with its adjacent subgrade soils. This point can be explained through a diagram shown in figure 44. LADOTD Specifications and Standard Plans defines the trench backfill area shown in figure 44. However, the compaction quality in the two subgrade backfill areas on each side of the trench backfill area is also important. If properly compacted, it can provide a smooth transition between the backfill and subgrade soils in stiffness, especially in cases where the existing subgrade is very stiff. Otherwise, it can be a weak zone if poorly backfilled.

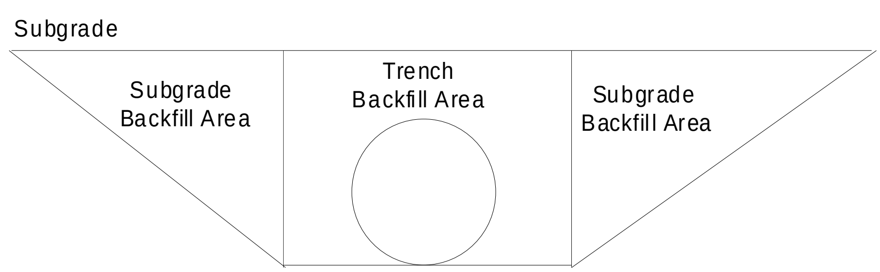


Figure 44
Diagram of cross-drain trench with transitional areas

An example of such a case was the cross-drain trench at station 2+316 on LA 964. At this cross-drain location, transitional subgrade backfill areas were formed in an effort to remove mud from the trench bottom by a backhoe machine after a rainfall. Figure 45 shows a group of photos taken during the cleaning and backfill process.

Figure 45
Example of backfill with transitional area

Cost Comparison

The material costs for some major trench backfills available in Louisiana are shown in table 16, though their field performances are quite different. Labor and time associated with the construction process for each of the materials might be different and are in addition to the material costs. The actual cost for individual projects will vary depending on project locations and material availability.

Table 16
Unit cost of different backfill materials

Material	Material Unit Cost $/cubic yd
Sand	5 – 6
Kentucky limestone	29
Mexican limestone	25
Bedding (Sand gravel)	8 – 15
RAP	undetermined
Selected soil	3 – 8
Flowable Fill	70 – 100

CONCLUSIONS

Pavement surface dips at highway cross-drains on Louisiana highways involve many inter-related complex factors. Field investigations indicated that pavement dips occurred at some cross-drain pipe locations but not at others that were under the same traffic and environmental conditions. The field probing tests conducted at the problematic sites revealed that pavement dip occurrence depended largely on the relative stiffness of trench backfill materials with respect to their adjacent natural soils. When a pavement surface dip occurred, the trench backfill underneath was weaker than adjacent subgrade soils. However, when the trench backfill was weaker than the adjacent subgrade soil, the occurrence and magnitude of pavement surface dips depended on the stiffness of pavement structures and truck traffic loading. Therefore, the probability of a pavement dip at an existing cross-drain trench can be predicted using the field testing techniques discussed in this report.

An effective way to prevent a pavement dip from occurring at a cross drain site is to establish the stiffness compatibility between trench backfills and their adjacent subgrade soils. Specifically for DCP tests, the N_{DCP} values of a trench backfill material should be at least equal to or larger than those of their adjacent subgrade soils.

The stiffness satisfaction of trench backfills with respect to their adjacent subgrade soils is the combined result of many factors involved during backfill construction. These factors include in-situ construction conditions, quality inspection, contractor workmanship, backfill materials and compaction, traffic loading during construction, configuration of pipes and trenches, etc. Observation during construction indicates that poor backfill, in most cases, was related to one or more of the factors. If these factors are all favorable to trench backfill construction, current LADOTD specifications on trench backfill can provide satisfactory performance, which explains why the majority of highway cross-drains function quite well in Louisiana.

However, trench backfill construction is often conducted under unfavorable conditions, such as construction delays, construction under traffic or in a poor weather, wrong backfill moisture content, or poor drainage condition of trench, etc. Construction under these conditions creates special requirements for backfill material, equipment, and compaction procedures. Since this study aims to prevent all unexpected pavement surface dips at highway cross-drains, the recommendation from this study should provide options to allow proper backfill of highway crossing-drains under different construction conditions, including those unfavorable and severe ones. The readers of the report should pay attention to the scope of this study for its limitation.

As such, more specific conclusions of this study are summarized as follows.

- With respect to supporting pavement structures, sand used in Louisiana is not a good backfill for highway cross-drains due to its very poor gradation and difficulty in compaction. Due to its poor gradation, compacted sand in trenches has the potential to further settle when subjected to heavy paving loads. Sand flooding is not an effective way to backfill highway trenches.
- Alternatives such as crushed limestone and flowable fill should be considered for highway cross-drains because of their good performance after placement. Kentucky limestone can be easily compacted to reach its required field density of 135 pcf. When its cost is justified, flowable fill is a good option because it has a very low settlement potential, even if an air-entraining admixture is used.
- Crushed limestone needs the least drying time in the field due to a narrow range of field moisture content, followed by RAP, sand gravel mixture, sand, and selected soil.
- Kentucky limestone has the best seepage stability, followed by Mexican limestone, RAP, and bedding material (not necessarily in this order), then followed by Selected soil and sand.
- Current LADOTD flowable fill mix design is a good specification, but it has a limitation of low first-day strength. Adding a strength accelerator to a flowable fill improves its

mechanical performance, resulting in a higher initial strength but a not-too-high long-term strength.

- Construction environment, contractor workmanship, backfill materials, and compaction are the major factors controlling the quality of trench backfill compaction.
- The occurrence of pavement surface dips at highway cross-drains is also affected by traffic loading, pavement structure, pipe cover layer, and the configuration of cross-drain trenches, etc.
- The DCP device can be used to evaluate the quality of trench backfills. Well compacted embankment/subgrade soils in Louisiana normally have a range of DCP values, N_{DCP}, between 5 – 7 hammer blows per 4 in. penetration. Therefore, the DCP values of trench backfills at 10 hammer blows per 4 in. penetration will prevent pavement surface dips from occurring at highway cross-drains.
- Of the compaction equipment used in the field, the Wacker Packer compactor is the most effective, followed by the vibratory roller and the vibratory plate compactors. This conclusion resulted from the DCP testing over a 3 ft. backfill.
- Construction machinery places a much higher stress level on highway subgrade than does the normal traffic. Subgrade should be strong enough to take the construction traffic loading. Otherwise, premature damage can occur undetected, which will weaken the pavement structure and shorten the pavement life. This is especially important for a subgrade with high moisture content and even true for a lime- or cement- treated subgrade.
- The cost given in this report is for reference purposes only.

RECOMMENDATIONS

The following recommendations are made based on the findings of this study:

Material

- Crushed Limestone, RAP, and sand gravel mixtures (bedding material) are recommended as cross-drain trench backfill if their gradations meet LA Specification 1003.03(d).
- Flowable fill can be used when its cost is justified.
- Sand and Selected soil that meet the current LADOTD specification can be used if proper compaction can be reached. Sand should not be used as backfill for cross-drains when the thickness of the cover layer is less than 4 ft.
- More field monitoring on using RAP and bedding material (sand gravel) as backfill should be continued.

Quality Control

DCP tests can be a measure for the final acceptance of completed cross-drain work as follows.

- DCP tests can be conducted directly in a backfill with a 1-ft. overlay. The DCP value N_{DCP}, over the depth of 4 ft. within backfill shall at least be 10 blows per 4 in. penetration (smaller than 10 mm/blow); or
- DCP tests can be conducted both within and out of trench areas on adjacent subgrade for comparison. Make sure the DCP value N_{DCP} within the backfill is larger than that of outside trench areas.

For the simplicity of use in the field, LADOTD project engineers can develop a correlation between the number of Wacker Packer passes and the DCP penetration blow count in the field as demonstrated in this study.

ACRONYMS, ABBREVIATIONS, & SYMBOLS

The following symbols are used in this report:

E_{PLT}	=	Elastic Modulus determined from Plate Load Test;
M_{FWD}	=	Resilient Modulus determined from Falling Weight Deflectometer;
M_R	=	Resilient Modulus back-calculated from DYNAFLECT;
N_{DCP}	=	Average penetration blow counts over a 5 cm thick soil layer;
n	=	Soil layers;
P	=	Applied force in a Plate Load Test;
PR	=	Penetration per each blow of Dynamic Cone Penetrometer;
R	=	Diameter of the loading plate used in a Plate Load Test;
R^2	=	Coefficient of Determination, and
δ_2	=	Reloading-induced deformation in a Plate Load Test

REFERENCES

1. ASTM D1196-93 (1997). “Standard Test Method for Nonrepetitive Static Plate Load Tests of Soils and Flexible Pavement Components, for Use in Evaluation and Design of Airport and Highway Pavements.” *Annual book of ASTM standards,* Vol. 04. 03.

2. ASTM D2321-00 (2002) “Standard Practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity-Flow Applications.” *Annual book of ASTM standards,* Vol. 08. 04.

3. ASTM D698 (2001). “Standard test methods for laboratory compaction characteristics of soils using standard effort (12,400 ft-lbf/ft^3(600kN-m/m^3)).” *Annual book of ASTM standards*, Vol. 04. 08, pp. 78-88.

4. Ahmad, I., and Putcha, S. (2000). “Evaluation of Compaction Methods for Pipe Trench Backfill in Areas of High Water Table”, *Transportation Research Record*, No. 1736, pp. 55-61.

5. Ghataora, G., and Alobaidi, I. (2000). “Assessment of the performance of Trial Trenches Backfilled with Cementitious Materials”, *International Journal of Pavement Engineering*, Vol. 1, No. 4, pp. 297-316.

6. Ghataora, G.S., Alobaidi, I.M., and Billam, J. (2000). “Use of Pulverized Fuel Ash in Trench Backfill”, *Journal of Materials in Civil Engineering*, Vol. 12, No. 3, pp. 228-237.

7. Anon (1988). “Trench Reinstatement with Foamed Concrete”, Civil Engineering (London), July-August, p. 66.

8. Ford, G., and Eliason, B. (1993). “Comparison of Compaction Methods in Narrow Subsurface Drainage Trenches”, *Transportation Research Record, No. 1425, pp. 18-27.*

9. Bester, M.D. and Hallat, L. (1977). “*Dynamic Cone Penetrometer (DCP)*”, University of Pretoria, Pretoria.

10. Yilmaz, R., Lakeman, B., and Melancon, J. (1984). “The Use of Cone Penetrometer Testing to Investigate Sand Fill Subsidence at Cross-Drain Locations”, *FHWA/LA-84/170*, Louisiana Transportation Research Center, Baton Rouge.

11. Webb, M. C., McGrath, T. J., and Selig, E. T., (1998). "Field Test of Buried Pipe with CLSM Backfill," The Design and Application of Controlled Low-Strength Materials (Flowable Fill), *ASTM STP 1331,* A. K. Howard and J. L. Hitch, eds., American Society of Testing Materials, pp. 237-254.

12. Du, L., Folliard, K., and Trejo, D., (2002). "Effects of Constituent Materials and Quantities on Water Demand and Compressive Strength of Controlled Low-Strength Material", *Journal of Materials in Civil Engineering*, ASCE, Vol. 14, No. 6, pp. 485-495.

13. ACI 229R-94, (1999). "Controlled Low Strength Materials (CLSM)", *American Concrete Institute Manual of Concrete Practice, Part I Materials and General Properties of Concrete,* American Concrete Institute, Farmington Hills, MI.

14. Kleyn, E.G. (1975), "The Use of Dynamic Cone Penetrometer (DCP)", *Report 2/74*, Transvaal Roads Department, Pretoria, South Africa.

15. Webster, S.L., Grau, R.H., and Williams, T.P. (1992). "Description and Application of Dual Mass Dynamic Cone Penetrometer", Geotechnical Laboratory, US Army Corps of Engineers, *Instruction Report GL-92-3*.

16. Kinchen, R.W., and Temple, W.H. (1980), "Asphaltic Concrete Overlays of Rigid and Flexible Pavements", Report No. FHWA/LA –80/147, Louisiana Department of Transportation and Development.

17. Burnham, T.R. (1997), "Application of the Dynamic Cone Penetrometer to Minnesota Department of Transportation Pavement Assessment Procedures", Report No. MN/RC – 97/19, Office of Minnesota Road Research, Minnesota Department of Transportation.

18. Chen, J., H. Mustaque, and T. M. LaTorella, (1999), "Use of Falling Weight Deflectometer and Dynamic Cone Penetrometer in Pavement Evaluation", Paper Presented in the Transportation Research Board, Washington, D.C.

19. Harison, J.A., (1989), "Correlation Between California Bearing Ratio and Dynamic Cone Penetrometer Strength Measurements of Soils", Proceedings of In-situ Civil Engineers, Part 2.

20. Jayawickrama, P.W., Amarasiri, A.L., and Regino, P.E. (2000), "Use of Dynamic Cone Penetrometer to Control Compaction of Granular Fill", *Transportation Research Record 1736*, Paper No. 00-1416, pp. 71-80.

21. Wood, D. M. (1990). "*Soil Behavior and Critical State Soil Mechanics*", Cambridge University Press.

22. Konard, J.M. and Lachance, D. (2000), "Mechanical Properties of Unbound Aggregates from DCP and Plate Load Tests", *Proceedings of the Fifth International Conference on Unbound Aggregate in Roads*, editor, A. Dawson, Nottingham, United

Kingdom, pp. 225-230.

23. LA S1003 (2000). *Louisiana Standard Specifications for Roads and Bridges: Part X, Materials*, *Section No. 1003*. State of Louisiana, Department of Transportation and Development, Baton Rouge.

APPENDIX

CORRELATIONS OF DIFFERENT IN-SITU TESTING RESULTS

The results in this Appendix are only for the documentation purposes only.

Correlation between NDCP and MR

Figure A-1 shows a general correlation between the resilient modulus, M_R, determined by DYNAFLECT and DCP data, including all the material types tested. Average N_{DCP} values over 2 ft. and 3 ft. were calculated at each location and plotted against the resilient moduli at the same locations. The difference between the two linear regressions is marginal for a practical purpose. With a correlation coefficient $R^2 = 0.62$, the relationship is given as:

$$M_R = 0.504 \cdot N_{DCP} + 4.097 \qquad (3 < N_{DCP} < 30) \qquad \text{(a-1)}$$

Here, M_R is in ksi and N_{DCP} is in blows per 10 cm, which is an average value of N_{DCP} over 3 ft. depth. Equation a-1 is quite different from the correlations suggested by other studies [17], [18], [19]. This variation can be attributed to the different methods used to determine the resilient modulus, M_R.

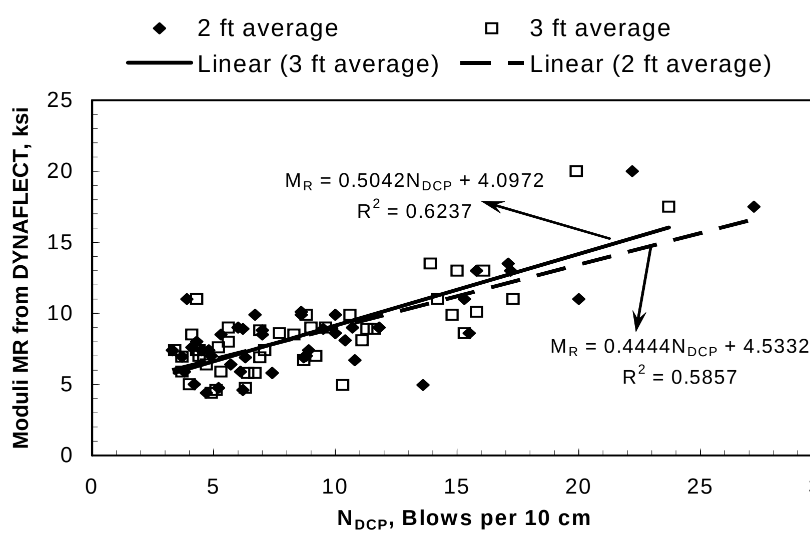


Figure A- 1
Correlation of N_{DCP} with the DYNAFLECT resilient modulus, M_R

DCP versus PLT

Figure A-2 is the correlation of average N_{DCP} values over a 3 ft. layer with the reloading elastic modulus obtained from PLT for the three backfill materials. This figure indicates that the various backfill materials studied generally follow the same trend with regard to their stiffness, and that a higher N_{DCP} value means a higher reloading modulus, E_{PLT}, of soils. With a regression correlation coefficient of $R^2 = 0.84$,

$$E_{PLT} = -0.0493 \cdot (N_{DCP})^2 + 2.0264 \cdot N_{DCP} - 1.9828 \qquad (2 < N_{DcP} < 15) \qquad \text{(a-2)}$$

Here, N_{DCP} is in blows per 10 cm and E_{FWD} is in ksi. Figure A-2 also shows the correlation suggested by Konard et al. 2000 [22] that is

$$\log(E_{PLT}) = 0.884 \cdot \log(N_{DCP}) + 1.977 \qquad \text{(a-3)}$$

Where, PR is in cm/blow and N_{DCP} is in blows/10-cm, and E_{PLT} is in ksi. The correlations described by equations a-2 and a-3 are quite close to each other.

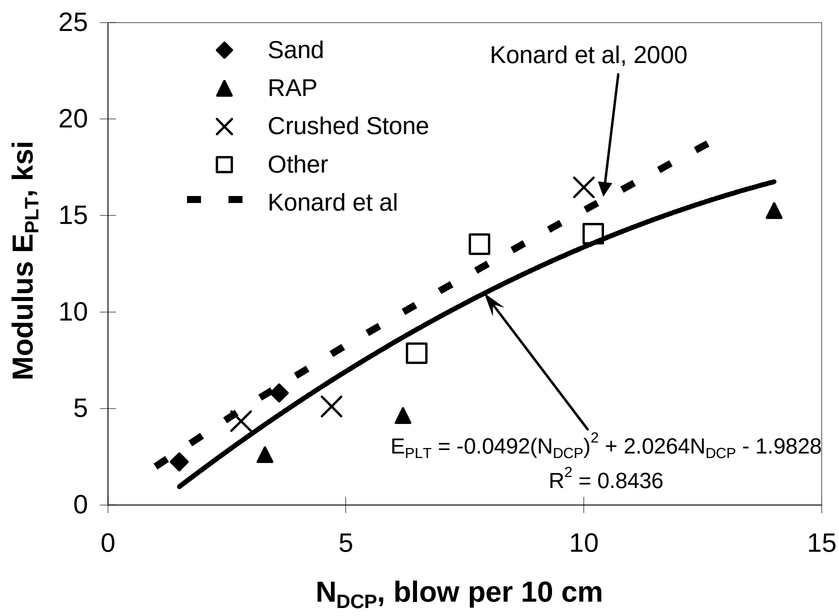


Figure A- 2
Correlation of average N_{DCP} values of a 3 ft layer with E_{PLT}

DCP versus FWD

The average values of N_{DCP} over 3 ft. were also correlated with the resilient modulus, M_{FWD}, determined by the FWD, as shown in figure A-3. With a regression correlation coefficient of $R^2 = 0.852$,

$$M_{FWD} = -0.0205 \cdot (N_{DCP})^2 + 1.5389 \cdot N_{DCP} + 2.4821 \qquad (2 < N_{DCP} < 15) \quad \text{(a-4)}$$

The units in equation a-4 are the same as in equation a-3. Figure A-3 also shows the correlation suggested by Chen et al. 1999 [18] that is

$$M_{FWD} = 8.13(N_{DCP})^{0.39} \quad \text{(a-5)}$$

Where, *PR* is in cm/blow and N_{DCP} is in blows/10-cm, and M_{FWD} is in ksi. The correlations described by equations a-4 and a-5 are quite different. The resilient modulus, M_{FWD}, determined by equation a-4 is 4.4 ksi lower than the values determined by equation a-5.

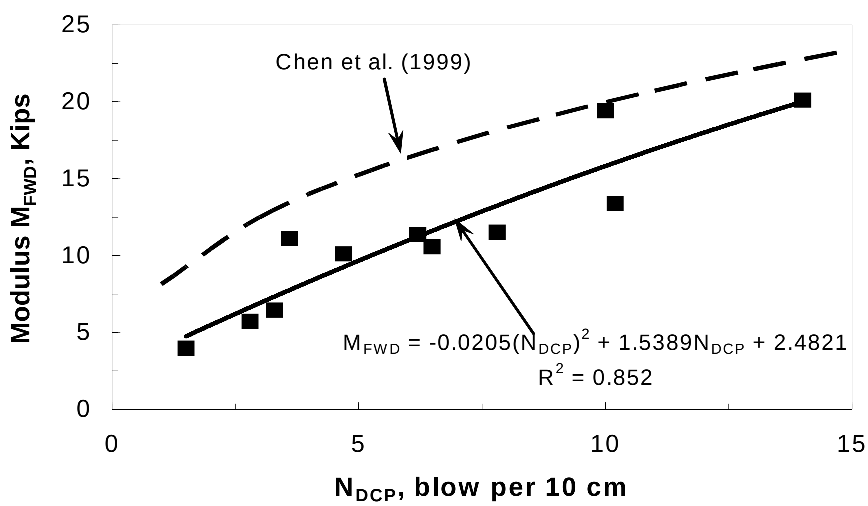


Figure A- 3
Correlation of average N_{DCP} values over a 3 ft layer with M_{FWD}

DYNAFLECT, PLT, and FWD

Figure A-4 compares the moduli determined by DYNAFLECT, PLT, and FWD by re-plotting equations a-1, a-2, and a-4. It indicates that the moduli from FWD are much higher than the values from DYNAFLECT and the values from PLT are between them.

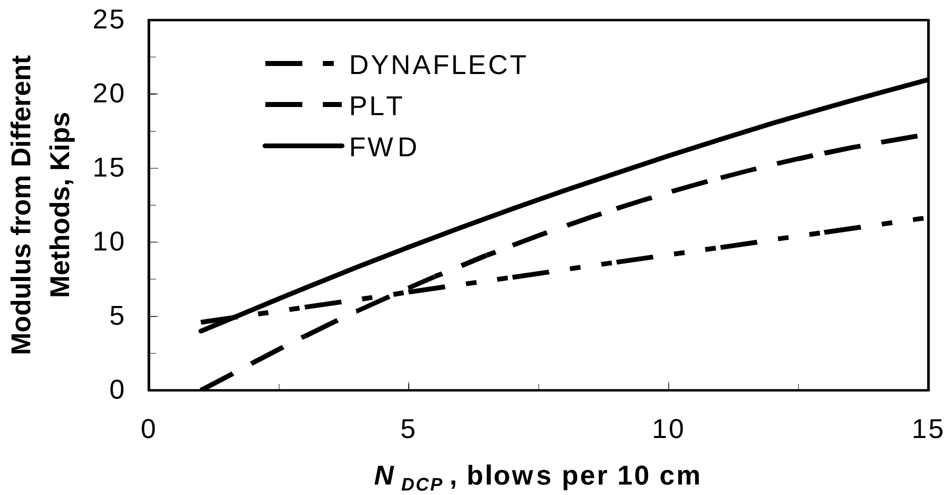


Figure A- 4
Relationship among moduli by different methods

With the available data from PLT, FWD, and DYNAFLECT obtained from the trench test site, correlations among PLT, FWD, and DYNAFLECT can be established as shown in figures A-5, A-6, and A-7. Figure A- 5 shows an empirical correlation between FWD and PLT. With a regression correlation coefficient of $R^2 = 0.785$,

$$M_{FWD} = 14.14 \cdot (E_{PLT})^{0.612} \qquad (1.45 \text{ ksi} < E_{PlT} < 17.4 \text{ ksi}) \qquad \text{(a-6)}$$

where both M_{FWD} and E_{PLT} are in ksi.

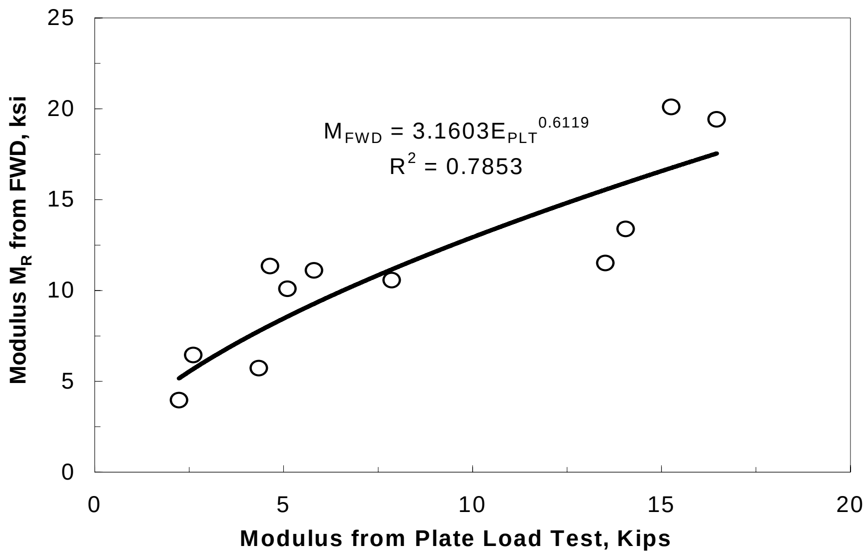


Figure A- 5
Direct correlation of E_{PLT} with M_{FWD}

Figure A-6 describes the direct empirical correlations between DYNAFLECT and PLT, and between DYNAFLECT and FWD. The abscissa in figure A-6 is $\alpha = W_1/SPD$, where W_1 is the maximum deflection read by first sensor as shown in figure A-6. *SPD* stands for percent of spread that is equal to the average of all sensor readings divided by the reading of first sensor. Therefore,

$$\alpha = \frac{W_1}{SPD} = \frac{(W_1)^2}{\frac{\sum_{i=1}^{7} W_i}{7}} = \frac{7 \cdot (W_1)^2}{\sum_{i=1}^{7} W_i} \qquad \text{(a-7)}$$

Here, W_i and α are in centimeters.

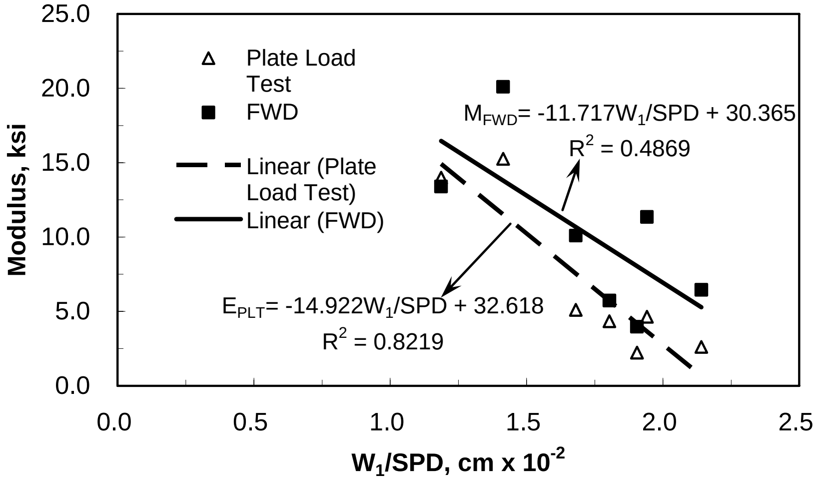


Figure A- 6
Correlation between DYNAFLECT reading and moduli

With a regression correlation coefficient of $R^2 = 0.49$,

$$E_{LPT} = -11.72 \cdot \alpha + 30.37 \qquad (1 < \alpha < 2) \qquad \text{(a-8)}$$

and with $R^2 = 0.82$

$$M_{FWD} = -14.92 \cdot \alpha + 32.62 \qquad (1 < \alpha < 2) \qquad \text{(a-9)}$$

Here M_{FWD} and E_{PLT} are in ksi. Figure A-7 shows the analytic model of DYNAFLECT for testing directly on subgrade soils where equations a-8 and a-9 are valid.

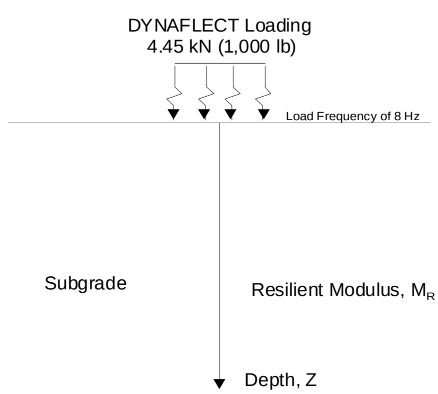


Figure A- 7
Analytic model of DYNAFELCT for subgrade test