# NTTA PAVEMENT DESIGN MANUAL

## SUBGRADE INVESTIGATION AND DESIGN MANUAL FOR HIGH PERFORMANCE TOLLWAY PAVEMENTS

NORTH TEXAS TOLLWAY AUTHORITY
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Version 1.1

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## 1.0 Introduction

The purpose of this manual is to present requirements and methods to perform the Geotechnical Subgrade Investigation and Pavement design for North Texas Tollway Authority Projects. The geotechnical and pavement engineers who perform this work are to be independent from all other engineering entities (i.e., section engineers, general engineering consultants, etc.) and are to be selected solely upon qualifications.

The requirements and design methods presented in this manual are based on consensus items presented in the White Paper titled *Design, Construction and Maintenance of High Performance Pavements* (2006). This manual should be made part of any prequalification package to allow proposers to evaluate qualifications, testing requirements, and the scope of work required. This manual is divided into distinct sections by work type and work flow. Upon completion of the work described in this document, the products will be a Geotechnical field and laboratory report, Traffic Analysis, Swell Mitigation Analysis, Grading/Stability Analysis and Pavement Design. This design manual provides guidance only for the subgrade investigation and pavement design. Foundations and structures investigations are typically included in this work and should be performed by a Geotechnical Engineer experienced in the areas of bridges, walls and structures. A corridor wide Geotechnical/Pavement Engineer may be selected to provide all geotechnical and pavement for the entire corridor.

Preliminary Document Review and Site Visit Section 2 is a research exercise designed to accumulate existing data regarding the geologic, geotechnical, and environmental conditions along the alignment. The work includes a site visit with an accounting of anomalous conditions observed along with a geologic, geotechnical, and environmental review of conditions that may affect the alignment and require further investigation during the field investigation.

The field investigation, covered in Section 3, includes methodology to lay out boring locations based upon field, geologic, and coverage considerations. Details are presented to allow the Engineer to provide a logical and detailed field investigation plan that includes boring locations, minimum depths of borings, and a sampling program designed to obtain the number and quality of samples necessary for laboratory testing.

Section 4 covers the minimum laboratory testing program necessary to determine engineering properties and characterization of the soils and bedrock underlying the alignment. The program will provide data for prediction of swell or consolidation of the subgrade and design of mitigation measures. Strength testing will provide total and effective strengths for design of slopes on both the short term and long term basis. Data for the design of the subgrade will include Resilient Modulus values of subgrade and stabilized subgrade, stabilizing agents and concentrations, and compacted properties for fill.

Section 5 presents criteria and calculation methodology necessary to define traffic loading. Specific parametric studies are suggested due to the unknowns associated with traffic projections and future traffic configuration loads, vehicles, and tires.

Swelling or Expansive subgrades are particularly problematic in the Dallas-Fort Worth region. Section 6 presents analysis methods for expansive, settlement prone, and rebound soils including testing, and methods of mitigation.

The grading, drainage, and stability of the project are covered in Section 7, which also specifically discusses stability of cut and fill slopes and drainage requirements. Analysis methods are presented for both short-term and long-term strength of soils, and methods to analyze skin slide stability are included.

The final sections of the manual cover the design of the pavement thickness, definition of the pavement profile (Section 8), and reporting of the investigation and design (Section 9).

As presented and agreed upon in the “Best Practices” document, the contract with the Pavement Engineer and Geotechnical Engineer should contain provisions for testing, observation and consulting services through construction. This is necessary to be sure the design intent is carried out during the construction phase. The testing and observation should include cut observation, fill placement, moisture treatment, stabilization, and paving.

## 2.0 Preliminary Document Review and Site Visit

In order to provide useful and reliable information upon which to base recommendations and considerations, the investigation must be well planned and executed. Even at very early stages of site review/alignment selection, the environmental and geologic considerations should be presented to the planners/engineers. Even minor alignment changes can save significant monies for the agency. Avoiding areas of environmental hazards, such as landfills, dump sites, and areas with contamination, can speed up the acquisition/construction processes and reduce costs. From a geologic/geotechnical viewpoint, avoiding areas of high groundwater, soft soils, hard rock, and expansive soils can be important to the decision-making process. Even if the alignment is fixed, awareness of these concerns can help in the budgeting process.

For most projects, there is a wealth of information available for very little effort. Major urban centers have databases on existing hazardous material sites available through the fire department, city planner, or other municipal agencies. Some areas have historical photos—both aerial and land—that can identify land uses and development history. Sanborn insurance maps, anecdotal evidence from landowners, and area residents can be used to further identify the land uses and locations of possible contamination.

Walking, driving, or even flying the site has obvious benefits and is strongly recommended. The tour should involve the geologist as well as the environmental and geotechnical engineers. Each of these individuals should look for potential problems, anomalies, vegetation clues, and improvements that may affect the cost and time required for construction. These data help to fine-tune the exploration program.

In addition, other sources of information can help define the character of the alignment. The importance of the document-review phase cannot be overlooked and, at a minimum, the following sources of data should be examined:

1) Geologic Mapping from the Texas Bureau of Economic Geology,
2) Topographic Mapping (plan and profile if available),
3) Soil Surveys from the USDA and Extension Service,
4) Environmental Reports for various properties along the alignment,

5) Insurance maps and local sources of environmental information — fire department, city offices, historical aerial photos, library, etc.,

6) Geotechnical Reports for facilities (roadways, buildings, etc.) along or near the alignment, and.

7) City, county and highway departments that may contain historical information on the design, construction and performance of the pavement system in the geographical area.

The Internet can also be a significant source of information and should be included in the document search. As the information on the Internet is not subject to verification, care should be given in assessing its accuracy.

After each document has been examined, the features or facilities of concern should be noted on an alignment map, and the field trip should be arranged. During the field trip, the team should note anomalies, features of interest, potential hazardous waste sites, and other things that need additional investigation during the drilling program.

The results of the document review are presented in a report that contains a bibliography and a reference list of the documents reviewed, results of the review as a list of features or areas of interest, along with details about the nature of the concern, likely materials to be encountered, and minimum boring depth and sampling program to be performed. If it is anticipated that borings will encounter hazardous materials, minimum precautions for drilling personnel and protocols for handling and disposing of the samples must be provided. An alignment map with the suggested boring locations and minimum depths should also be presented.

## 3.0 Geotechnical Field Investigation

The planning for the field investigation will include plotting the boring locations on a plan drawing of the alignment, obtaining entry/drilling permission, locating or adjusting hole locations due to access or owner preferences, clearing utilities, locating and elevating the hole location, and staking and labeling the boring location. Before drilling any borings, the environmental studies for the alignment should be examined and compared with the proposed drilling program. Borings located in areas of hazardous materials should be either relocated or the drilling crew equipped with appropriate safety equipment. In the event hazardous materials are encountered during drilling, the drilling should be stopped, the area isolated, crew decontaminated and the environmental team notified. The area should be mitigated prior to resuming the drilling.

Boring depths should be planned and based upon the proposed construction profile elevations. Preliminary plans that show the plan and profile of the mainlanes and frontage roads are necessary. Boring depths and sampling plans should be based upon the types of information desired. For example, data necessary to analyze subgrade support characteristics include:

1) Subgrade swell-consolidation profiles;
2) Moisture and density;
3) Strength;
4) Classification;
5) Proctor compaction and remolded Resilient Modulus ($M_R$) values.

Sampling will require undisturbed and bulk samples of the subgrade. Detailed requirements are contained in the following sections.

Boring locations can be surveyed by a licensed Professional Land Surveyor, taped from a surveyed center line, or located using the Global Positioning System (GPS). In order to use GPS technology, the survey team must provide longitude and latitude coordinates for each boring location. It should be noted that only survey-quality GPS will provide sufficient accuracy for elevation. Commercial hand-held units can provide locations within about ± 9 feet, which is generally sufficient.

## 3.1 Boring Locations

Drill one (1) boring each 500 feet each direction in the mainlanes (250' alternating lanes) and each 500 feet direction in the frontage roads. Borings shall be staggered to optimize coverage. In addition, borings shall be located at the following:

1. Mapped changes in geology including faults;
2. Geologic or Geotechnical anomalies;
3. Topographic highs and lows;
4. Drainage crossings;
5. High points in cuts over 15 feet;
6. Low points with over 15 feet of fill;
7. Geologic features such as: landslides, springs, seeps, depressions, mounds, stockpiles, etc.;
8. Features noted in the Document Review and not elsewhere covered in the program; and
9. Any designated borrow areas (minimum of one boring per acre).

## 3.2 Boring Depths

Borings shall extend at least 15 feet Below Subgrade Profile Elevation (BSPE). Where fill is to be placed, borings must extend at least 10 feet into competent material (TCP or SPT blow counts greater than 25/12). Boring depths in borrow areas shall extend at least 10 feet below anticipated bottom of cut.

***NOTE:*** *These minimum depths apply only to pavement and grading borings. Structure borings should be as required by the Structural Engineer, NTTA, or TxDOT.*

## 3.3 Sampling Program

The sampling program will consist of obtaining large bulk samples, small disturbed samples, and relatively undisturbed samples. All sampling shall be in accordance with either ASTM or AASHTO standards and shall be sufficient to complete all testing as required by Section 4 of this document. The sampling program is dependent upon the material purpose, i.e., subgrade support, embankment soils, and foundation for embankment support. All samples except bulk samples shall be

protected from moisture change, excessive heating or cooling, and handling disturbance and shall be labeled to clearly identify the Project, Boring No., and sampling depths where the sample was obtained.

### 3.3.1 Sampling of Subgrade Soils (0-15' BSPE)

Sampling of the subgrade soils should consist of bulk, undisturbed and small cup samples.

**3.3.1.1** Sampling is to provide relatively undisturbed samples at a maximum of 2 foot intervals to 15 feet BSPE (Below Profile subgrade elevation) for swell-consolidation and strength testing and is to consist of either Shelby Tube, California Sampler, Pitcher Barrel, Dry Core, NX or PQ Diamond Core or other sampling device approved by NTTA and capable of providing undisturbed samples.

**3.3.1.2** Small disturbed samples allow for classification and moisture content testing are to consist of grab samples of cuttings of each soil type encountered (minimum of 2 per soil type, each boring). Samples are to be placed in heavy-duty, double-lock Ziploc™ bags (or equal) or sealed cups designed to keep moisture constant.

**3.3.1.3** Bulk samples are necessary for Proctor compaction testing, remolded strength testing, stabilization testing, and Resilient Modulus testing. Bulk samples shall contain sufficient material so that when grouped with similar soils, they constitute a sample with an aggregate weight of at least 130 pounds by dry weight.

Bulk sampling should be planned based upon geology and subsurface conditions. Where the conditions are uniform, each bulk sample can be as little as 50 pounds. In variable geologic conditions, samples of 130 pounds each are recommended. Grass, roots, and other organic materials should be excluded during sampling.

### 3.3.2 Sampling of Embankment soils (Borrow Soils)

These are soils in cuts that are to be used as fill in other parts of the alignment and also have subgrade soils at the bottom of the cut. These soils require sampling from the surface to 15 feet Below Profile Subgrade Elevation (15 feet below bottom cut for borrow areas).

**3.3.2.1** Sampling provides undisturbed samples for moisture and density testing, classification, and sonic testing for rock to determine rippability. Sampling should consist of undisturbed sampling at maximum at 2 foot intervals to 15 feet Below Profile Subgrade Elevation. Approved sampling methods include Shelby Tubes, California Sampler, Pitcher Barrel, Dry Core or NX or PQ Diamond Core. Sampling tools shall be appropriate for the soil or rock being sampled. Any other sampling methods must be approved by the NTTA

**3.3.2.2** Sampling for classification and moisture-content testing shall consist of grab Samples of cuttings of each soil type encountered (minimum of 2 per soil type, each boring). Samples are to be placed in heavy-duty, double-lock Ziploc™ bags (or equal) or sealed cups designed to keep moisture constant.

**3.3.2.3** Bulk samples are necessary for Proctor compaction testing, remolded strength testing, stabilization testing, and Resilient Modulus testing. Bulk samples shall contain sufficient material so that when grouped with similar soils, they constitute a sample with an aggregate weight of at least 130 pounds.

The plan for bulk sampling should be based upon geology and subsurface conditions. Where the conditions are uniform, each bulk sample can be as little as 50 pounds. Grass, roots, and other organic materials should be excluded during bulk sampling.

### 3.3.3 Foundations for Embankment

These borings are located in areas that support fill. Examples are low areas, bridge abutments, overpass fills, culverts and drainages, approach fills, etc. Here, the primary concern is the strength and compressibility of the foundation soils; thus, undisturbed samples as well as field-strength testing are required. Testing should take place at alternating 4-foot intervals; thus either undisturbed sampling or in-situ penetration testing is provided every 2 feet.

**3.3.3.1** Sampling is to provide undisturbed samples for moisture and density testing, and classification at 4-foot intervals. Approved sampling methods include Shelby tubes, California Sampler, Pitcher Barrel, Dry Core or NX or PQ Diamond Core. Sampling tools shall be appropriate for the soil or rock being sampled. Any other sampling methods must be approved by the NTTA.

**3.3.3.2** In-situ testing to verify strength in the form of either Texas Cone Penetrometer (TCP) or Standard Penetration Test (SPT) is also required. Testing should alternate with undisturbed sampling at 4-foot intervals.

## 3.4 Boring Logs

As each boring is being drilled, it shall be logged in the field by an Engineer or Geologist qualified and trained in drilling and sampling procedures, soil and bedrock classification, and local geology. Logging by drilling personnel is not allowed. Field boring logs shall contain documentation of the boring number, consultant, and drilling contractor as well as a description of the surface conditions in the area and location (NB mainlane, etc.). Logs shall also contain location information Lat/Long, existing and finished subgrade elevation, and data collection method (Professional Land Surveyor or GPS). In addition, the logs must contain the depths at which soil types are encountered. Logs should also contain:

1. Sampling tools used;
2. Blow count for driven tools along with hammer weight and fall;
3. Soil or rock symbol;

4. Sampling or testing interval;
5. Results of field-testing performed (pocket penetrometer, torvane, etc.);
6. Rock core data, including % recovery and Rock Quality Designation;
7. Groundwater data, including depth of water encountered, time after drilling the measurement was taken, and recommended additional measurements;
8. Coring interval.

Boring logs, similar to those attached as pages 12 and 13, should be used. Logs shall include descriptions of the soil and rock encountered and shall contain the major and minor constituents, stiffness/density, moisture condition, minor components or characteristics, color and Unified Soil Classification System designation. A typical entry might appear as follows:

*Clay, slightly silty, very stiff, very moist, contained calcareous nodules and traces of sulfate crystals, yellow brown to gray brown, (CH)*

### 3.5 Quality Review

Once the logs have been completed, a Quality Review shall be performed by the Engineer or Project Manager. The review shall consist of comparison of logs and samples for proper and accurate descriptions, review of field-sample testing, and sampling procedures for proper application and satisfactory results. Reviews shall be conducted on a daily basis so that changes can be made field procedures in a timely fashion. Each log must be signed off on by the reviewer. Corrections shall be circled, must be in red or other noticeable color, and must be obvious to reviewers. Reviews shall be discussed with the original logger.

### 3.6 Reporting

Logs of the exploratory borings shall be reported in the final "Subgrade Investigation and Pavement Design" report on the plan and profile sheets, as well as, fence logs plotted to elevation with the finished subgrade shown on the same

drawing. Single-page logs may, at the discretion of the Engineer, also be presented in the Appendix (See Section 9.0).

NORTH TEXAS TOLLWAY AUTHORITY

PAGE 1 OF ____

# LOG OF BORING NO. _____

JOB NAME: _____

CONSULTANT: _____ LOCATION: _____

DRILLING CONTRACTOR: _____ RIG TYPE: _____

SURFACE CONDITIONS: _____ LOGGED BY: _____

LAT/LONG: _____ METHOD: _____ DATE DRILLED: _____

GROUND ELEVATION: DRILLER: JOB NO.:

DRILLING METHOD(S):

GROUNDWATER INFORMATION - DEPTH AND TIME AFTER DRILLING:

| SAMPLE TYPE | | | |
|---|---|---|---|
| ST Shelby Tube | RC Rock Core | SS Split Spoon | DC NX Core |
| CT Cuttings | TC THD Cone | CS California Spoon | PB Pitcher Barrel |

| FIELD DATA | | | | | | | | SOIL & ROCK SYMBOL | GEOTECHNICAL DESCRIPTION |
|---|---|---|---|---|---|---|---|---|---|
| DEPTH, FT | SAMPLE/TEST INTERVAL, FT | SAMPLE TYPE | N: BLOWS/FT CS or BLOWS/6" SS | T: BLOWS/FT | P: TONS/SQ FT | R:PERCENT | RQD: PERCENT | | |
| | | | | | | | | | |
| 5 | | | | | | | | | |
| 10 | | | | | | | | | |
| 15 | | | | | | | | | |
| 20 | | | | | | | | | |
| 25 | | | | | | | | | |
| 30 | | | | | | | | | |

FIELD TEST

N - STANDARD PENETRATION TEST RESISTANCE
T - THD CONE PENETRATION RESISTANCE
P - POCKET PENETRATION RESISTANCE
R - PERCENTAGE OF ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:

PAGE 2 OF ____

# LOG OF BORING NO. _____

| FIELD DATA | | | | | | | | SOIL & ROCK SYMBOL | GROUNDWATER INFORMATION - DEPTH AND TIME AFTER DRILLING: |
|---|---|---|---|---|---|---|---|---|---|
| DEPTH, FT | DEPTH INTERVAL, FT | SAMPLE TYPE | N: BLOWS/FT CS or BLOWS/6" SS | T: BLOWS/FT | P: TONS/SQ FT | R:PERCENT | RQD: PERCENT | | GEOTECHNICAL DESCRIPTION |
| | | | | | | | | | |
| 35 | | | | | | | | | |
| 40 | | | | | | | | | |
| 45 | | | | | | | | | |
| 50 | | | | | | | | | |
| 55 | | | | | | | | | |
| 60 | | | | | | | | | |
| 65 | | | | | | | | | |

| SAMPLE TYPE | | | |
|---|---|---|---|
| ST Shelby Tube | RC Rock Core | SS Split Spoon | DC NX Core |
| CT Cuttings | TC THD Cone | CS California Spoon | PB Pitcher Barrel |

FIELD TEST
N - STANDARD PENETRATION TEST RESISTANCE
T - THD CONE PENETRATION RESISTANCE
P - POCKET PENETRATION RESISTANCE
R - PERCENTAGE OF ROCK CORE RECOVERY
RQD - ROCK QUALITY DESIGNATION

REMARKS:

## 4.0 Laboratory Testing Program

The program shall be developed to obtain the data necessary to support design and analyses of grading, slope stability, embankment, selective grading, moisture treatment, stabilization, and subgrade support. All soils and bedrock shall have basic classification and engineering properties determined; gradation, Atterberg limits, as well as strength and swell-consolidation properties. In addition, critical state properties, moisture content, and density must be determined to provide beginning points for the analysis. When the project has significant grading, other factors, including rebound, settlement, and fill crowning, must be determined. This data provides the basis for the analyses used to predict movement potential of both embankments and cuts.

Slope stability considerations are very significant in the highly expansive clays typically encountered in North Central Texas, where slopes as flat as 3:1 horizontal to vertical commonly fail each spring. Both total and effective stress strengths are required to analyze slope stability.

Design of the subgrade is the most critical part of the program, especially where expansive soils are present. In order to provide adequate design data, it is necessary to create swell profiles typical of both wet spring and dry summer conditions. These data will be used to predict maximum movement potential and to develop engineering methods to mitigate damages. Strength and dynamic properties associated with the subgrade soils, both treated and untreated, are necessary to design the pavement section and engineer slopes.

The following sections will present minimum laboratory tests and frequencies. It should be noted that the frequencies are minimums and that additional testing is normally required to adequately characterize the support conditions, movement potential, and other engineering properties and classifications.

Since the products of this work include designs and design parameters for highway pavements, the testing standards used in this manual will be AASHTO, TxDOT, and where appropriate, ASTM (in this order of preference). Where there is a material difference, only one reference will be cited.

Laboratory protocols can have a significant affect on the validity of the rest results. To help standardize testing and provide reliable results, laboratory protocols for many of the standard tests have been provided in Appendix A.

## 4.1 Classification Tests

Classification tests can be performed on either disturbed or undisturbed soil samples. These properties are related to the soil particles and their constituent parts rather than to the behavior of the mass. Drilling and sampling programs usually assume that the disturbed cup, baggie, or grab samples will be used for moisture content and/or classification tests. Undisturbed samples should be reserved for testing to determine engineering properties of the mass such as density, strength, permeability or swell.

Classification testing is used to provide the engineer with either the AASHTO or Unified Soil Classification System classification. These classifications relate soil type and predict certain behaviors under load or as a result of changes in moisture and density. For example, a classification of CH or A-7-6 denotes high plasticity clay with a significant potential to swell when wetted, generally poor subgrade support, and a low wet strength. Similarly, a soil classified as GW or A-1-a would connote well-graded gravel having excellent support characteristics, load transfer capability, stability under moisture changes, and high strength.

### 4.1.1 Testing Methodology

Classification testing consists of gradation analyses and Atterberg limits and is governed by AASHTO M-145 or TEX-142-E standards. The Unified Soils Classification System is generated by ASTM D 2487.

1. Gradation Analysis AASHTO T-27 or TEX-110-E (applicable for soils having 50% or more retained on the No. 200 sieve).
2. Percent Passing the No. 200 Sieve AASHTO T-11 or TEX-111-E (applicable for all soils and soil-like bedrock).
3. Atterberg Limits

   Liquid Limit AASHTO T-89 or TEX-104-E

   Plastic Limit AASHTO T-90 or TEX-105-E

### 4.1.2 Testing Frequency

The classification tests shall be completed for each soil type encountered, with a minimum of one test per boring. The classification tests shall be conducted on samples obtained at or just below finished subgrade. Borrow and soils from cut areas shall also be tested.

## 4.2 Sulfate Testing

Sulfates present a special problem in the design of stabilized subgrade. Sulfate, calcium, water, and most clay soils react to form a highly expansive mineral call Ettringite. Some clays react quickly and forcefully, and others show virtually no reaction. Recent studies have shown the reaction is based on the amount of free silica in the system, a factor not easily measured. Interestingly, these two opposite reactions can be found in the same formation with an identical chemical make up. The only way to determine the severity of the reaction is to test the soils during the design process. This testing is necessary for both natural subgrade soils as well as materials to be used as fill.

### 4.2.1 Test Methodology

Testing protocol follows TxDOT-145-E. The process is performed at room temperature, and no additives are used to aid the sulfate extraction. The result is a more accurate representation of the water-soluble sulfates available for reaction in the field, which are extractable by water at ambient temperatures.

### 4.2.2 Testing Frequency and Location

Unlike other types of testing, sulfate sampling and testing locations are critical. The upper 2 feet of the subgrade and subgrade adjacent to concrete, such as drilled pier retaining wall, are important locations for testing. Most of the structure testing will be performed by the Geotechnical Engineer for the structures.

The primary sampling and testing responsibility for the Subgrade and Pavement Engineer are for the subgrade that comes into contact with the pavement system, including to portions which may be stabilized. Care should be taken to determine the sulfate concentration in the entire profile of all significant cuts (>10 feet). Test frequency shall be a minimum of one per boring.

## 4.3 Other Testing

Depending upon the corrosivity and nature of construction, other tests that may be performed to determine classification properties include:

1. Shrinkage Limit;
2. Specific Gravity;
3. pH;
4. Chloride Concentration; and
5. Resistivity.

## 4.4 Engineering Properties Tests

Engineering properties relate to the behavior of the soil or bedrock mass and include such properties as moisture content/density, strength, permeability, consolidation potential, and swell potential. Knowing the properties of both the natural soils and remolded (fill) soils is necessary for predicting the behavior of the subgrade.

### 4.4.1 Moisture/Density Profile

Moisture and density profiles are typically performed in conjunction with other testing, such as strength, swell-consolidation, or permeability tests. It is necessary to also provide moisture/density profiles for cut areas along the alignment at 2-foot intervals to at least 10 feet below finished subgrade. These data are necessary for calculating moisture requirement and shrinkage or bulking factors used in grading analyses.

#### 4.4.1.1 Testing Methodology

Testing for the moisture/density profiles shall consist of measuring the moisture content in accordance with AASHTO T 265 or TEX-103-E. For undisturbed samples, density shall be determined by AASHTO T 204.

#### 4.4.1.2 Testing Frequency

When data are available from other testing, such as swell-consolidation or strength testing, supplemental testing is not required. Where the alignment is in cut over 10 feet in depth, a boring shall be drilled at the high point and a moisture/ density profile provided. Other moisture/density profiles should be considered necessary and helpful in determining the condition of the soils and subgrade.

As a minimum, there shall be at least one moisture/density profile each 1,500 lineal feet of alignment.

### 4.4.2 Swell-Consolidation Testing

Swell-Consolidation testing provides engineering data on the movement potential of the subgrade, either upward or downward. Swell-Consolidation profiles shall be provided at 2-foot intervals to at least 10 feet below finished subgrade elevation. Properly conducted, these tests can be used to estimate the amount of movement and, for settlement, the time required for the settlement to occur. Movement potential can come from subgrade soil or sedimentary bedrock and, in most cases, is attributable to either of the following:

- Settlement associated with soft soils, debris, or uncontrolled fill, collapsing soils or wet/saturated soils subject to additional loading;
- Rebound associated with deep cuts; or
- Expansion as a result of wetting expansive clay minerals.

#### 4.4.2.1 Settlement Prone Soils

Testing should be as appropriate for the geotechnical conditions; i.e., collapsing soils can be tested using the simplified ASTM D 4546. The procedure should follow the predicted conditions in the field: loaded to overburden, wetted, then loaded to the final stress or loaded to final stress, then wetted. Where conditions include soft soils that will be loaded with embankment, timed consolidation tests are appropriate (ASTM D 2435). The data will provide the basis for predicting the amount and time rate of consolidation. The testing frequency should be dependent upon the field conditions and planned construction. At least one test is necessary where any significant embankment (>15 feet) is planned and the soils are softer than 25 blows for 12 inches of penetration by Standard Penetration Test.

Significantly more testing should be conducted where data suggests the soils are soft or subject to hydro-collapse, as may be typical with silts and some silty sands. For these conditions, a testing program similar to the one for expansive soils is appropriate.

#### 4.4.2.2 Rebound Considerations

Rebound is a consideration for deep cuts (>25 feet) where unloading the soil results in an upward movement similar to a spring being unloaded. Testing must be customized for the individual conditions, much like timed consolidation testing. The load should be sufficient to exceed the overburden pressure by a significant amount in order to predict the behavior during unloading and reloading. Timed loading increments are useful for estimating the time rate of the rebound and reloading as appropriate per ASTM D 2435. The testing frequency shall be as appropriate for the proposed construction, but should be at least one per cut exceeding 25 feet.

#### 4.4.2.3 Expansive Soil Conditions

Testing should be assigned to characterize the soil, meaning that the effect of moisture content and overburden pressure must be evaluated. The testing is provided in swell profiles taken at no more than 1,500-foot intervals. In addition, samples shall be tested on a routine basis in the upper four feet of the subgrade. The proof stress used should be approximately equal to the overburden stress after grading. Testing shall be in accordance with ASTM D 4246.

To replicate the summer, worst-case condition, the samples taken in the upper six to eight feet of the subgrade shall be air-dried for a minimum of 24 hours prior to testing (dependent upon engineering judgment and natural moisture content of the individual sample).

In cut areas, potential fill materials must also be evaluated for movement potential in the “as compacted” state. Usually, it is most efficient to sample Proctor compaction samples at the design moisture content to determine movement potential. Otherwise, samples will have to be manufactured.

The Eagle Ford Shale represents a special case in subgrade preparation. The shale is so dense and impermeable that it takes several weeks to achieve saturation. These soils cannot be moisture treated within a reasonable length of time and are generally removed and replaced when they are within ten feet of the finished subgrade.

### 4.4.3 Strength Testing

The primary reason for strength testing is to calculate appropriate slope ratios for cuts and fills. Thus, both total stress strength (unconsolidated-undrained) and effective stress (consolidated-drained) values are necessary. Strengths for natural soils in cuts and below fills and remolded soils for fills will be required.

Generally, either unconfined compressive strength test ADTM D 2166, direct shear ASTM D 3080, or Q triaxial tests ASTM D 2850 can be used to determine the total stress shear strength. Effective stress values must be determined by either consolidated-drained (S) test or consolidated-undrained with pore pressure measurements (R bar) test ASTM D 4767.

Testing frequency is dependent upon the plan and profile of the alignment. Where cuts exceeding 15 feet are planned, strength testing shall be performed. Where the slopes are critical, a suite of testing, including undrained and effective strength testing, is necessary to evaluate the stability; at least three tests of each soil type in the cut shall be performed.

The embankments are built using soils derived from the cuts or borrow areas along the alignment and shall be tested as remolded soils. Moisture contents shall be on the order of 3% above standard Proctor optimum moisture content for soils considered expansive and ±2% for granular or non-expansive soils. Both total and effective stress strength parameters are required. At least three samples for each major soil type are to be tested.

### 4.4.4 Resilient Modulus

Subgrade strength and support values for pavement design are determined using Resilient Modulus ($M_R$) testing. Testing shall be performed in accordance with AASHTO T 307 on remolded samples prepared using the Static Load Method, which are at least 3% to 5% above standard Proctor optimum moisture content. No other methodology for determining $M_R$ is permitted.

Testing must also be performed on each type of stabilized subgrade, base, subbase, and other components within the pavement system. As a minimum, the testing

frequency for subgrade testing shall be one test for each 2,000 feet of pavement. For uniform subgrade conditions, a minimum of three replicates is required. Where the subgrade is variable, testing shall be performed on each subgrade type encountered. Grouping or selective[1] testing may be used.

### 4.4.5 Embankment/Cut/Borrow Soils

This subgrade represents a unique case in the pavement design, since there is no way of knowing where the material is likely to be placed. If the material is of either good quality or poor quality the engineer may elect selective placement. Regardless, it is the geotechnical engineer's duty to classify and determine the engineering properties of the material before rational decisions can be made.

To provide rational recommendations, the classification and engineering properties of the materials must be measured in the laboratory under conditions similar to the field, i.e., the same density and moisture content as recommended for compaction. Samples from the Proctor compaction test ASTM D 698 (minimum of three per soil type) can be used to determine strength and swell-consolidation characteristics. Both total stress and effective stress strengths are needed for slope stability analyses. In determining swell-consolidation properties for expansive soils, it is necessary to allow drying (24 hours) to simulate drought conditions.

Where the fill and/or subgrade consist of expansive soils, a series of samples should be manufactured to be between standard Proctor optimum and 5% above optimum. These samples shall be subjected to swell-consolidation tests with loadback to at least 10 ksf and unconfined compressive strength or Q triaxial tests to provide engineering properties of moisture treated materials.

### 4.4.6 Stabilization of Subgrade Soils

Pavements will have a stabilized subgrade a minimum of 8 inches in thickness. This is a requirement where the subgrade has been moisture treated (MTS). Moisture-treated subgrades are soft, yielding, insufficient for supporting additional loads from pavement or

[1] Selective testing is a process during which samples with similar properties and the same AASHTO classification are grouped and tested as a single unit. Typically, three tests are done on the combined sample to provide a basis for comparison.

traffic, and susceptible to pumping. Other types of subgrade also benefit substantially from stabilization. Benefits include improved support, credit as a structural layer, decreased pumping, improved stability and improved constructability. In order to be considered "Stabilized Subgrade," the material must be engineered and must have the following properties:

- Unconfined Compressive Strength > 160 psi
- Swell <1%
- pH = 12.3 after mellowing (applies to lime stabilization only)

The design must consist of samples prepared at various percentages of additive (lime or Portland cement) and cured to replicate field conditions and testing in accordance with the protocols listed in Appendix A. Field testing must be used to confirm the design and construction.

## 5.0 Traffic Analysis

The purpose of this section of the Design Manual is to present methods for analyzing the traffic data to be used for pavement design. Guidelines for evaluating current and projected traffic numbers based on the different roadway design conditions, traffic distributions, design lives, and other factors to develop design ESALs (Equivalent 18 kip Single-Axle Loads) are presented.

Traffic data shall be analyzed using the Adjusted Demographic Analysis, not T & R (Traffic and Revenue) forecasts. These data are available from the NTTA traffic analysis consultant.

The traffic data shall be converted into Equivalent 18-kip Single-Axle Loads (ESALs) according to the pavement type and estimated thickness. Steps required to calculate the ESALs are as follows:

1. Select Average Daily Traffic (ADT), one direction, from the adjusted demographic analysis;
2. Select the analysis period based upon pavement type;
3. Determine the annual growth rate based on the projected ADT using the adjusted demographic criteria;
4. Determine traffic distribution consisting of percentage of traffic for each vehicle type;
5. Determine the Load Equivalency Factor (LEF):
    i. Empirical based value for Portland Cement Concrete Pavement (PCCP) or Asphalt Cement Concrete Pavement (ACCP) according to AASHTO Guide (1993).
    ii. Mechanistic based value for high performance Continuously Reinforced Concrete Pavement (CRCP) according to Dr. Zollinger's spreadsheet;
6. Estimate directional and lane distribution factor;
7. Bracket for low, moderate and high percentages of trucks;
8. Calculate ESALs for pavement design and analyses; and
9. Develop traffic matrix which consists of low, moderate and high risk based on the different percentage of truck traffic with 80 kips and 97 kips Gross Vehicle Weight (GVW).

## 5.1 Traffic Data Review

Input for traffic data shall be based on traffic study estimates provided by the transportation planners/engineers. Components to the traffic data required for traffic analysis are presented below. Engineering judgment should be used in selecting the appropriate traffic distribution and growth factor to be used in the traffic analysis.

### 5.1.1. Traffic Projections

The traffic study provides traffic projections in terms of estimated Average Daily Traffic (ADT). The traffic projection is based on "Traffic and Revenue" (T&R) projections and "Adjusted Demographic" projections. These are two separate and distinct projections. The T&R projections are conservative on the low side for financial purposes to predict cash flow, bond rating, and financial qualifications for the system. A traffic projection study based on the Adjusted Demographic is more realistic and is appropriate for design purposes. Projected adjusted demographic traffic number on the high side should be used in the traffic analysis. The projected traffic numbers are typically presented in a Lane Line Diagram. The diagram represents estimated ADT in the mainlane, ramps, frontage roads, and cross roads. Usually, the ADT is split by direction so that extrapolation is not necessary.

**Estimated Average Daily Traffic (ADT)**

The analysis periods are based on optimistic traffic forecasts of at least 40 years for high performance Continuously Reinforced Portland Cement Concrete Pavement (CRCP) and Portland Cement Concrete Pavement (PCCP), and 20 years for Asphalt Cement Concrete Pavement (ACCP).

The ADT numbers should be evaluated for the mainlanes, ramps, frontage roads, and cross roads. The Lane Line Diagram from the traffic study divides the roadways into sections based on the number of entering and exiting vehicles through a particular section of the roadway. Projected traffic data are provided for each section of the mainlane, ramp, and frontage roadway. ADT design numbers can be either analyzed individually or grouped into like sections having a range of similar values. Experience has shown that numbers varying less than 20 percent are typically satisfactory for design purposes. Design ADT should reflect numbers that will

encompass the expected traffic to be experienced by the majority of mainlanes, ramps, and frontage roads.

### 5.1.3. Traffic distribution

Vehicle traffic distribution is typically obtained from historical traffic data taken from similar roadways, highways, and tollways. It can vary greatly depending upon land use, alternate routes available, tolling charges, and geopolitical considerations beyond the Engineer's control. While many design methods utilize up to 10 traffic categories, only 4 are being used in this design methodology. Vehicles are categorized to include passenger cars, light trucks, and 80-kip and 97-kip heavy trucks.

The damaging effect of truck traffic on pavement outweighs the effect of passenger cars on pavement. Due to the uncertainty of the projected future traffic configuration, it is recommended that the truck traffic projections to be used in the analysis are on the high side. Underestimating the truck traffic could significantly reduce the serviceable life of the pavement. Engineering judgment should be used when selecting truck percentages for traffic analysis.

### 5.1.4. Growth factors for design periods

The projected traffic data could range from 20 to 40 years after construction. The annual growth factor can be calculated based on the current and projected ADT from the adjusted demographic traffic study. Depending on the project type, engineering judgment can be used to select a range of growth factors for developing a traffic matrix to be used in the pavement thickness analysis. Typical values for growth factors are between 3 and 7 percent with a typical design value of 4 percent. Growth should be limited by number of lanes including possible widening.

### 5.1.5. Truck Percentages

The single most significant unknown in the Traffic Analysis is the percentage of truck traffic. In the past, it was common to use truck percentages varying from less than 2 to 6 percent trucks (light and heavy trucks). Today some highways have truck densities approaching 50 percent. As the NTTA system expands and connects to

major highways within the DFW area, a significant increase in truck traffic should be anticipated, especially in systems linking the east and west corridors.

For purposes of the analyses, the selection of the truck percentage can be based either on studies of highways paralleling the proposed construction or on estimates based on historical data or perception which are typically included within the Traffic & Revenue (T&R) studies. At a minimum, 4 percent trucks should be used unless the traffic consultant has data to the contrary. The minimum value should also be increased by 2 or 3-percent increments depending on the accuracy of the estimate. The larger the uncertainty; the higher the bracket value that should be assigned. A minimum of 2 brackets should be used so that a table of values can be determined as follows:

**Table 5A Truck Percentage vs. Risk**

| Risk | Trucks |
|---|---|
| High | T&R -2% |
| Moderate | T&R |
| Low | T&R +2% |

## 5.2 Traffic Calculations

This section presents detailed traffic analyses for each of three pavement conditions; i.e. Asphaltic Cement Concrete Pavement (ACCP), Jointed Plain Portland Cement Concrete Pavement (JPPCCP) and Continuously Reinforced Concrete Pavement (CRCP). Each pavement condition has its own specific requirements, Load Equivalency Factors (LEF), and procedures. Note the traffic volume must be tempered to not exceed the maximum lane capacity.

### 5.2.1 Portland Cement Concrete Pavement (PCCP)

Plain Portland Cement Concrete Pavement sometimes noted as Jointed Plain Portland Cement Concrete Pavement is permitted only on frontage roads and cross roads with specific NTTA permission. Design is based upon the AASHTO (1993) methodology with the following requirements:

1. Pavement design life should be selected according to the AASHTO, FHwA, or TxDOT requirements. Pavement design life is 40 years for Portland Cement Concrete Pavement (PCCP).

2. Total ADT (2-way) used for design purposes should reflect numbers that will be experienced by the majority of mainlanes, ramps, frontage roads, and cross roads. The traffic number should be based on conservative projections of the traffic study using the adjusted demographic.

3. Truck percentages for heavy truck traffic can be determined based on data from the adjusted demographic traffic study. For analysis purposes, the percentage of heavy trucks from the high end of the range should be used. The heavy truck traffic should be classified further into single axle and tandem axle. Percentage of truck traffic should be increased by 2 to 3 % to take into account projected moderate and high-traffic numbers.

4. Directional Distributional factor of 0.5 is typically used for traffic analysis unless there is significant difference in the number of traffic experienced in one roadway direction compared to the other roadway direction. $D_D$ can range from 0.5 to 0.7.

5. Lane Distribution Factor ($D_L$); according to AASHTO:

Table 5B Lane Distribution

| Ultimate Number of Lanes in Each Direction | Percent of 18-kip ESAL in Design Lane |
|---|---|
| 1 | 100 |
| 2 | 80–100 |
| 3 | 60–80 |
| 4 | 50–75 |

6. Annual Growth Rate (g) is determined using the current and projected ADT data provided by the adjusted demographic T&R study. Annual growth rate can be calculated using the following equation:

$$g = [(ADT_i / ADT_0)^{1/n}] - 1$$

Where $ADT_0$ = current year ADT

$ADT_i$ = projected year ADT

n = analysis period (years)

7. Growth Factor (GF) is based on the annual growth rate (g); traffic growth factor can be calculated as:

$$GF = \frac{(1 + g)^n - 1}{g}$$

where g = annual growth rate

n = analysis period (years) = 40 years

8. Load Equivalency Factor for each vehicle type can then be determined based on the axle configuration (single, tandem or tridem), $p_t$, axle load (kips), and slab thickness (inch) in accordance with the 1993 AASHTO "Guide for Design of Pavement Structures."

#### 5.2.1.1 ESAL Traffic Calculations

Traffic input consists of Average Daily Traffic (ADT), the Traffic Distribution ($D_D$ and $D_L$, Growth Factors (GF) and Load Equivalency Factors (LEF); the design, ESALs for each vehicle type can be calculated as:

ESALs = $D_L$ x [(Total ADT x $D_D$) x % of ADT x 365 days x Design Period-years x LEF)] x GF

Total ESALs = Σ ESALs per vehicle type

### 5.2.2 Asphalt Cement Concrete Pavement ACCP (AASHTO)

Asphaltic concrete is allowed only on frontage roads and cross roads. Design value requirements for ACCP are similar to PCCP. Differences are listed as follows:

1. Pavement design life should be 20 years for Asphalt Cement Concrete Pavement (ACCP).

2. Load Equivalency Factor (LEF) for each vehicle type can be determined based on the axle configuration (single, tandem or tridem), $p_t$, axle load (kips), and Structural Number (SN). For estimation purposes, SN = 5.0 and D= 9 inches may be used as a starting point for the analysis. LEF values are listed in the 1993 AASHTO "Guide for the Design of Pavement Structures."

#### 5.2.2.2 ESAL Traffic Calculations

The design ESAL can be calculated using the equations provided in Section 5.2.1.1 can be used to calculate the ESAL.

### 5.2.3 Traffic Analysis for High-Performance CRCP

Trucks cause much more damage than passenger cars; therefore it is critical to use engineering judgment when analyzing percentage of truck traffic based on the adjusted demographic traffic study. The following sections discuss the recommended approach of traffic analysis.

#### 5.2.3.1. Truck Distribution

Truck traffic typically consists of single and tandem axle. A conservative estimate of truck traffic distribution of 60% single axle and 40% tandem axle trucks is the recommended distribution. Tridem axle trucks are seldom encountered, and their effect on pavement was shown to be less than that of tandem axle trucks and is therefore not considered in the traffic analysis.

Traffic data shall be analyzed for tandem axle trucks with Gross Vehicle Weight (GVW) of 80 kips and 97 kips. Given that the NTTA has elected to install Weigh-In-Motion (WIM) devices and enforce legal load limits, overloaded trucks with GVW greater than 97 kips should not be a problem and are not considered in this analysis.

#### 5.2.3.2. Percentages of Truck Traffic

A range of truck traffic percentages can be obtained from the adjusted demographic study or from analyses of comparable highway systems paralleling the proposed route. Due to the uncertainty of the projected traffic volume throughout the design life of the pavement, a traffic matrix consisting of low, moderate, and high risk should be developed and used in the design of mainlane pavement.

The traffic matrix should be developed based on project conditions and engineering judgment. For the high risk situations, use the highest truck percentage value from the adjusted demographic study. For moderate risk situations, an increase in truck percentage of 2 to 3 percent is acceptable. Subsequently, for low risk situations, an additional 2 to 3 percent above the truck percentage for moderate traffic can be used. The additional increase in truck percentages for moderate and low traffic number can be increased if justified.

#### 5.2.3.3. Traffic Analysis

The traffic assumptions are critical to the long-term performance of the pavement system. A parametric analysis using the traffic volume assumed for the design, along with probable variants, is needed since any traffic projection is little more than an educated guess. The variation of traffic assumptions should be based on the realistic traffic volume projections and variations in the amount and type of truck traffic. Traffic analysis should be performed as low, moderate, and high total traffic categories. These traffic categories are based on the low, moderate, and high percentage of trucks. Truck ESALs can be calculated by using the appropriate LEF for trucks with 80 kips GVW and 97 kips GVW. Following the guidelines for ESAL calculations for each roadway and pavement type presented in above, the following traffic matrix should be developed:

Table 5B Risk vs. ESALs

| Risk | ESALs | |
|---|---|---|
| | 80k GVW | 97 k GVW |
| Low | 8% | 8% |
| Moderate | 6% | 6% |
| High | 4% | 4% |

This traffic matrix will provide different scenarios on which pavement thickness calculations are based and will be discussed in the Pavement Design Section.

## 6.0 Swell Mitigation

The technique is to pre-wet the expansive clay; thus satisfy the soil's desire for water. The wetting effectively pre-swells the soil and provides a strain-absorbing layer that reduces the slope of the surface projection of the heave feature. This pre-wetting is accomplished by excavating, moisture treating and replacing the soils in the subgrade during the mass grading process. The thickness of the moisture-treated zone is a function of the slope of the surface feature and traffic speed. Data and experience have shown that the faster the traffic, the more susceptible the vehicle is to differential movement caused by the change in slope. Curves were developed based on finite element analysis and will be used to determine depth of moisture treatment based on average swell of soil subgrade below pavement (Figure 6.1).

Guidelines for evaluating the soil boring logs and laboratory test results to be used for calculating required depth of moisture treatment will be provided below in Section 6.2. Field sampling and laboratory testing procedures required for subgrade treatment analysis were discussed in the previous Sections 3 & 4. Moisture treatment or moisture injection is not required for non-expansive sand, sandstone, or limestone.

### 6.1 Laboratory Testing Assignment

Swelling characteristics of the soil subgrade can be evaluated based on the laboratory test results that include moisture content (ASTM D 2216), one-dimensional swell/consolidation (ASTM D 4546-03), and soil properties (ASTM D 4318-05 and D 1140). Laboratory testing procedures were discussed in the Geotechnical Investigation section. The following laboratory testing tests are needed for the analysis:

1. Moisture content (ASTM D 2216)—Moisture content profile should be assigned in each soil boring log. Samples should be tested for moisture content at 2-foot intervals.
2. One-dimensional swell/consolidation (ASTM D 4546-03)—Swell/consolidation profile should be assigned in each soil boring log. Samples should be tested for swell at 2-foot intervals. The confining stress shall be equal to the overburden stress after grading and should relate to finished subgrade, not existing grade. The moisture content should reflect dry periods of the year and may require at least 24 hours for drying prior to testing.

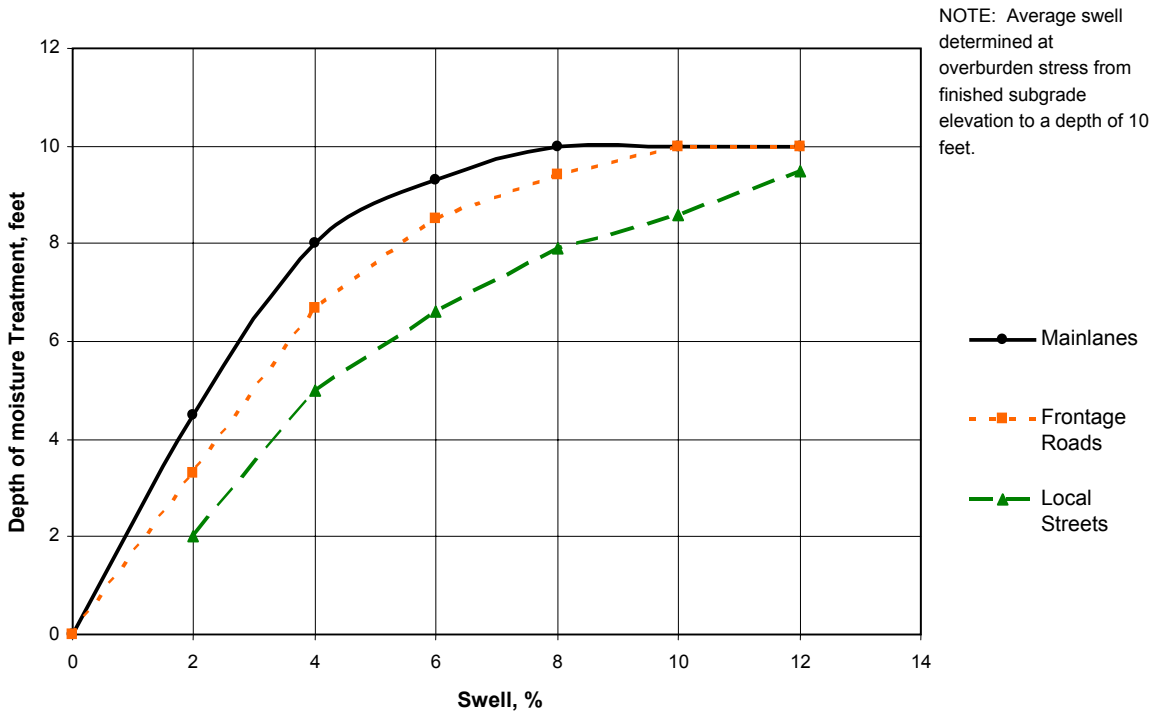
3. Soil properties (ASTM D 4318-05 and D 1140)— Atterberg Limits and passing No. 200 sieve tests should be assigned in each soil type within each soil boring log to determine strata change within the soil boring.

## 6.2 Moisture Treatment Analysis

Depth of moisture treatment should be calculated on an individual soil boring log basis using laboratory test results that include soil properties, moisture content, and one-dimensional swell/consolidation. The following procedure lists the analyses and calculations recommended for depth of moisture treatment:

1. The soil profile in the boring can be determined based on soil properties test results. The soil in each soil boring should be categorized according to USCS (Unified Soil Classification System) using the liquid limit, plastic limit, plasticity index, and percent material passing No. 200 sieve test results.
2. Within each strata, moisture content profile test results can be used to further evaluate the in-situ condition and assigned swell value.
3. Once the swell values have been assigned to each stratum in the soil boring, calculate the mean swell of the soil boring to use as the design swell percentage.
4. Calculate the median and standard deviation using the swell values in the soil boring.
5. For sampling and testing conducted during the months of July, August, and September use the mean swell percentage. For all other time periods, use the mean plus one standard deviation to determine the design swell percentage.
6. Once the design swell percentage has been determined, use Figure 6.1 to determine required depth of moisture treatment or the amount of replacement non-swelling fill.

## 6.1 Swell vs. Moisture




Once the recommended depth of moisture treatment has been determined for each of the soil boring, evaluate the recommended depths of moisture treatment for the borings located within the area. Minimize changes in recommended depths of moisture treatment through a roadway section to reduce design and constructability issues.

## 7.0 Grading

Grading operations involve excavation in planned cut areas and fill in embankment areas using slopes engineered to remain stable for the life of the project. Prior to grading, both cut and fill areas must be cleared of trees and stripped of organics. Preparation of the embankment and subgrade are part of the grading process and critical to the pavement performance.

During the excavation, the soil and rock are disturbed from their in-situ state and transported for use as fill in embankments. In determining quantities for borrow and waste, careful planning should address material bulking as the soils are excavated and shrinkage as they are placed.

Subgrade can be excavated, processed (moisture treated), and replaced using a rolling-box cut construction method very similar to open pit strip mining. The process involves excavating an initial cut and stockpiling the soil. Subsequent excavation is processed as it is placed into the previous excavation. The top foot of moisture treated subgrade shall be lime stabilized. The bulking of the lime stabilized subgrade can be 15 to 20 percent and must be taken into consideration during grading operations.

Proper drainage rapidly collects and conducts water away from the roadway and structures. Drainage can be achieved through surface and subsurface drainage. Drainage is also required in many cut slopes and existing drainages to intercept groundwater, which could impact pavement performance.

### 7.1 Clearing and Grubbing

The clearing and grubbing process is performed prior to grading or other roadway construction operations. The process consists of clearing wooded areas along the right of way of trees and shrubs followed by grubbing, which is the removal of deleterious matter that is embedded in the underlying soil (tree roots and stumps). Stripping removes the remaining roots, grass, sod, crop, and any remaining vegetation until only natural soil is exposed and all organics are removed. Organics are undesirable because they decay, forming voids and creating noxious, explosive gases. In addition, organics inhibit lime and cement reaction with the soils.

### 7.1.1 Removal of Organics

The process consists of removal of most organics and vegetation, such as grass and shrubs. Depth of organic removal depends on the type of vegetation and land use. The process consists of stripping the grass and shrubs to a depth of 3 to 12 inches in fallow land and 12 inches to 2 feet in farm land. Soil survey maps, site observation, and boring logs can provide information on the topsoil and organic matter thickness of the area. Organics should be removed to where no vegetation or root is greater than 1 inch in diameter in embankment areas or 3 inches in diameter in non-structural areas. Voids left due to removal of tree roots and stumps should be backfilled with similar material and compacted to the required fill moisture and density.

Since the presence of organics in the soil may interfere with the reaction of stabilizing agents (lime and cement) with soil, all organic materials within 2 feet of finished subgrade should be completely removed. These organic materials can be stockpiled for reuse as landscaping material. Where the finished subgrade is at or near the natural elevation, extra care is required to remove the entire root zone and to fill the excavation with soils similar to the native soils but with no organics.

### 7.1.2 Tree Removal

During field site observation, locations and size of wooded areas, density, and types of trees should be noted. Depth of removal is dependent on the type of tree and its root system. Taproot trees, such as pecan, and shallow root trees, such as cedar, do not require extensive effort for removal. Deep root trees, such as oak, will require considerable effort for stump removal and backfilling the resulting hole. Trees should be removed according to the following future-use guidelines:

1. **Structural Embankment:**

   Remove organics greater than 1 inch in diameter.

   Replace similar soil into the root cavity with compactive effort.

   Observe and test moisture and density of the placed fill.

2. **Non-structural Embankment:**

   Remove organics greater than 3 inch in diameter.

   Replace similar soil into the root cavity with compactive effort.

3. **At Subgrade:**

All organics should be excavated, and areas to be stabilized should be tested and verified for organics.

Visually visible organics should be removed in areas supporting structural elements and pavement.

### 7.1.3 Disposal of Vegetation/Organics

Soils with organics which are removed during clearing and grubbing can be stockpiled for use as topsoil or wasted in non-structural areas. Vegetation/organics greater than 1 inch in diameter should be chipped before being mixed with topsoil. The organics can also be wasted at embankment out-slope in non-structural areas, disposed off-site or burned.

## 7.2 Compaction

Grading operations should follow the moisture and density compaction requirements set forth in this design manual and project specifications. Subgrade compaction requirements include subgrade preparation, fill placement, processing with moisture and compaction. At a minimum, subgrade preparation includes scarification and compaction of areas to receive fill. Additional subgrade preparation including moisture treatment in cut areas, transition areas and locations near the surface of fill sections, as well as removal and replacement or re-compaction of soft subgrade and collapsing soils may be required.

### 7.2.1 Compaction Requirements

Compaction requirements should be as follows:

#### 7.2.1.1 Moisture-Treated Fill

Embankment within 10 feet of pavement in expansive soils shall be placed as moisture-treated fill. Subgrade shall be processed and placed in lifts no more than 8 inches in thickness. Moisture treated fill shall be placed at 3 percent or more above its optimum moisture content at 94% compaction, according to standard Proctor ASTM D698 or Tex-113-E. The design percentage of moisture shall be determined from the laboratory test results.

#### 7.2.1.2 Subexcavation for Moisture Treatment

Subgrade should be processed and placed in no more than 8 inch lifts. Moisture treated fill should be placed at 3% or more (depending upon test results) above its optimum moisture content at 94% compaction, according to standard Proctor ASTM D698 or Tex-113-E.

#### 7.2.1.3 Ordinary Embankment

Subgrade should be processed and placed in no more than 8 inch lifts. Non-expansive fill should be placed at -1 to 4% above its optimum moisture content at 95% compaction, according to standard Proctor ASTM D698 or Tex-113-E.

#### 7.2.1.4 High/Deep Embankment

Where fills exceed 20 feet in height, a zoned fill compaction is recommended. Use of modified compaction standards in lower parts of the fill will increase the strength of fill sections and moisture treatment (if necessary) will reduce swell in the upper 10 feet. Subgrade should be processed and placed in no more than 8 inch lifts. Lower part non-expansive fill should be placed at -1 to 4% above its optimum moisture content at 95% compaction according to modified Proctor ASTM D1557.

Where large fills (>20 feet) are required, the design should also provide recommendations for overbuild (to allow for settlement) and crown (differential settlement of sloped fill).

### 7.2.2 Special Conditions

Special subgrade conditions should be evaluated on a case-specific basis to determine the appropriate recommendations for alleviating potential problems during construction. These special conditions can be identified from topographic and geologic features. Findings are then confirmed through field observations, boring logs and laboratory test results. The following conditions deserve special attention:

#### 7.2.2.1 Soft Soils

The thickness of the soft zone should be determined during the drilling program. If the soft zone occurs near the top of the subgrade, the soil can be removed to firm subgrade, processed, and replaced with compactive effort. Stabilizer may be added

to increase the strength. As an alternative, geogrids products with flexible base may be used to bridge the soft soil areas.

#### 7.2.2.2 Collapsing Soils

Areas of collapsing soils should be identified, and their effect on pavement structure should be evaluated. If possible, excavate the collapsing soils to firm subgrade and replace with compaction control. If the location of collapsing soils zone prohibits removal and replacement, then an alternative method to reduce settlement should be employed (i.e. dynamic or vibratory compaction).

#### 7.2.2.3 Saturated Conditions

Subsurface drainage must be designed and installed where saturated conditions are expected within the pavement subgrade. Subsurface drainage alternatives are discussed in Section 7.4.

#### 7.2.2.4 Artesian Conditions

Additional drainage, both surface and subsurface drains and other precautions should be employed to reduce potential subgrade failures. Installation of interceptor or mat drains can be considered. The source of artesian flows should be determined and, if possible, the water source should be cut off or redirected.

#### 7.2.2.5 Abutments

Use of flow fill or sleeper slab should be considered in areas where pavement will adjoin the bridge structures to reduce the 'bump' at the bridge effect.

#### 7.2.2.6 High/deep embankments

Settlement of high/deep embankments will affect the performance of pavement and related structures. To account for settlement, the embankment can be overbuilt and adjusted to final grade after most of the settlement has occurred. Provide crown on the roadway to reduce differential settlement and to allow for surface drainage. Also, provide for a settlement monitoring program.

Large fills can require a significant amount of time for consolidation to occur; for clean sands, periods of weeks and for high plasticity clays, periods of years. The

Engineer should provide estimate of the time required for consolidation and methods to accelerate the process. Wick drains, drainage layers, pre loading and other methods to reduce the time should be presented.

### 7.3 Bulking and Shrinking of Fill Material

Several factors influence the change in volume of material excavated from cut areas and used as fill.

As the soil and bedrock are excavated, the volume will increase by about 100%. This swelling is due to excavation and dumping followed by transporting and loose placement. Loose or fluffed material undergoes volume shrinkage when compacted. Comparing natural density with compacted density; the difference will indicate bulking (volume increase) or shrinkage (volume decrease). Bulking and shrinkage factors are the ratios of in-situ density/compacted density. These values can be used to determine amount of import fill required for the embankment or to determine the amount of excess fill that will be generated from a cut section.

In-situ density of soils and rock is a function of compaction or overconsolidation, density and moisture content. The stiffer, denser, and harder the material, the greater the decrease in density will be due to excavation. Any moisture increase/decrease will also affect the shrinkage or bulking of the material. Usually, rock will have a tendency to bulk while most soils will shrink. The effect of moisture content can be upwards of a 4% difference in the ratios and, in a large project, can be tens of thousands of cubic yards. Likewise, overcompaction, or compaction above the minimum specified, while allowable and even admirable, could result in a 5 or more percent change in the ratio.

If we look at a typical example of a 110 pcf dry density natural soil and 110 pcf dry density compacted soil, the net bulking/shrinkage factor = 0. If these soils are overcompacted by 5%, then the dry density becomes 115.5 pcf and the net shrinkage 5%.

If we examine the same situation using total weight and thus consider the moisture content, the problem becomes even more complex. If the natural soils exist at 119 pcf total density, then they contain 9% moisture. Using a 95% compaction with a 12% optimum moisture content yields a total density of 123.2 pcf. These values would indicate a net shrinkage (119 pcf/123.2 pcf x 100 = 96.4%) of 3.6% for the as specified condition. At 3% over optimum

and 5% over minimum compaction the values become (133 pcf/123.2 pcf x 100 = 108%) or 8% bulking. Therefore, it is important to examine the total weight or net weight condition. Based on change in density, the percentage of shrinkage bulking can be calculated as follows:

$$\text{Shrink/Bulk, \%} = [\ 1 - (\gamma_c / \gamma_i)\ ] \times 100$$

$\gamma_c$ = Total or Wet Density of compacted fill (pcf)
$\gamma_i$ = Total or Wet Density of in-situ material (pcf)

Bulking and shrinkage can assist in determining the design volume required for the proposed fill and embankment areas. By averaging the density for each material, an estimate of shrinkage/bulking factor can be determined for the soil, weathered bedrock, and bedrock. This calculation should be done using a statistically significant number of measurements and must include allowances for the season in the upper 7 to 9 feet. Density of the in-situ material can be determined from soil borings and laboratory test results. Designers should be warned that variation in slope ratios, moisture content, and over or under compaction will significantly affect the validity of these results. The values listed below should be used only for initial estimation and should be replaced by actual calculated values adjusted by operating values as soon as the data is available. Typical changes in volume for different materials are as follows:

| Material | Net Volume Changes (factor) |
|---|---|
| Loose or Soft Soil | -15 |
| Dense or Very Stiff Soil | -5 to +5 |
| Weathered Shale/Marl | +5 |
| Shale | -10 |
| Weathered Limestone | +15 |
| Limestone | +20 |

### 7.4 Surface and Subsurface Drainage

Proper drainage is essential for rapidly collecting and conducting water away from the roadway. Drainage can be achieved through surface and subsurface drainage. Improper or insufficient drainage can lead to pavement and slope failures and significant increase in the cost of maintenance.

### 7.4.1 Surface Drainage

Surface drainage can be obtained by having adequate roadway cross slope to ensure rapid water removal from pavement sections. Installation of properly sloped roadside drainage features will allow water to be rapidly removed. Typical roadside drainage consists of flat-bottom and v-section ditches with longitudinal slopes of 1 to 3%. The following are the recommendations for surface drainage:

#### 7.4.1.1 Drainage Features

Roadway ditches should be designed to remove the water and to not allow the water to saturate the subgrade. At a minimum, ditches should be installed at 6:1 side slopes. Drainage features should be sloped longitudinally at a 2% minimum where possible. Steeper slopes up to 5% are desirable.

#### 7.4.1.2 Pavement Crown/Cross Slopes

Movement potential should be considered when determining pavement crown. A cross slope of 1 to 2% should be considered for the roadway and shoulder. Additional cross slope should be designed considering potential for differential subgrade movement, heave, or settlement.

#### 7.4.1.3 Run Off Protection

Water from surface runoff should be diverted away from the pavement subgrade and collected in the swale, median, or another site-specific detention area away from structures, slopes, or pavements. Where subsurface drains (storm sewers) are used and the subgrade is moisture sensitive (swelling or collapsing), pipes should be leak proof and tested before acceptance.

#### 7.4.1.4 Slopes

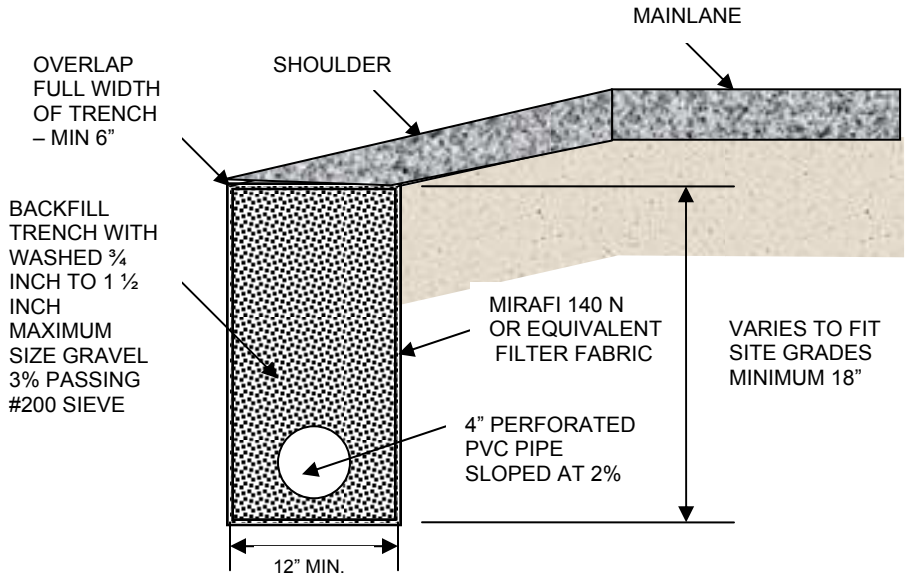
The face of the slope should be protected from erosion. Protection can be provided through surface diversion drainage, vegetation, erosion control, structures and stabilization. Surface slope protection can be achieved through the installation of rip rap, concrete, structures (i.e. drop structures and energy dissipators), by planting appropriate vegetation and cement or lime stabilization of drainage slopes.

### 7.4.2 Subsurface Drainage

If ground water is encountered near the pavement subgrade during the investigation or is anticipated due to geologic, drainage, or other conditions, a subsurface drainage system should be installed to control the flows and reduce impact on pavement and structures. Cut slopes may intercept the water table and can be stabilized only by subsurface drains. Seasonal fluctuations should be considered when determining areas of high groundwater.

Recommended subsurface drainage tools include the pavement drain, the interceptor drain, and the sausage or burrito drain. Pavement and interceptor drains are longitudinal drains designed to intercept groundwater and convey the groundwater to surface flow. The design must be performed using flow nets to determine the amount of drawdown and spacing to lower the groundwater level to the minimum necessary to keep the subgrade and structures dry. Pavement drains consist of collector pipes placed in trenches below the pavement. The pipes are placed on both sides, sometimes in the center of the pavement system, and are parallel to the alignment. The typical pavement drain is presented on Figure 7.1.

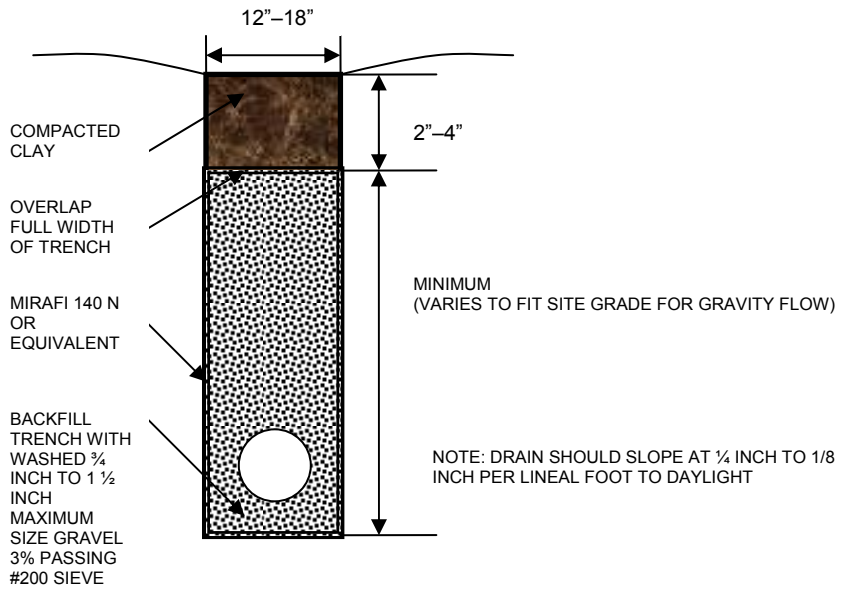
Figure 7.1 – Pavement Drain




Interceptor drains are typically used to draw down high groundwater conditions. Designs must include lateral daylight benches and may contain clean outs if the drains

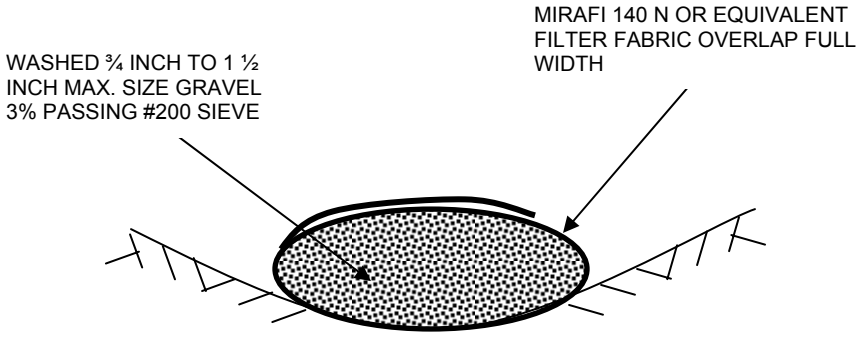
are piped and are critical to the long-term drainage plan. Figure 7.2 presents an interceptor drain. Drains without a pipe are is typical for low volume installations

Figure 7.2 – Interceptor Drain




Sausage or burrito drains are normally placed below fill in natural drainages. This precaution prevents water from building up at the lowest part of the embankment and causing a slope failure due to softening of the fill and foundation soils.

Figure 7.3 – Sausage Drain




To ensure proper functionality, daylight points need protection. Trash racks and head wall should be provided. Routine maintenance to clear the openings should be required.

## 7.5 Selective Grading

Depending upon the geology, project size, and observed soil and bedrock types, the project may benefit significantly from selective grading. This process involves simply replacing from 1 to as much as 10 feet of undesirable subgrade with on-site materials that are significantly better from the viewpoint of stability or support. Where the investigation indicates a significant deposit of granular or low plasticity soils, selective grading should be considered. Subgrades that are candidates for removal and replacement include soft soils, wet soils, expansive soils, collapsing soils, and soils with undesirable constituents, i.e., organics, sulfates, chlorides, etc.

## 7.6 Slope Stability

The stability of embankments or slopes should be thoroughly analyzed, since their failure may lead to accident and injury, as well as, significant economic loss. The failure of a slope takes place mainly due to the action of gravitational and/or seepage forces. A slope may also fail due to excavation or the undercutting of its toe or due to the gradual degradation of the soil strength. The internal surfaces of every mass of soil bounded by a sloping surface are subject to shearing because of gravitational forces that tend to pull the upper portions of the mass downward toward a more nearly level surface. A slope stability analysis consists of two parts:

1. The determination of the most severely stressed internal surface and the magnitude of the shearing stress to which it is subjected. (the driving forces)

2. The determination of the shearing strength along this surface. (the resisting forces)

The shearing stress to which any slope can be subjected depends on the unit weights of the material and on the geometry of the slope, while shearing strength that can be mobilized to resist the shearing stress depends on the character of the soil, its density, and drainage conditions. Once the slope geometry and subsoil conditions have been determined, the slope stability should be assessed using suitable computer analyses.

Slope stability analyses are sensitive to parameter selection and method of analysis. Engineers should have a significant amount of experience with slope stability analysis and design.

### 7.6.1 Slide Types

Terzaghi and Peck state "Slides may occur in almost every conceivable manner, slowly or suddenly, and with or without any apparent provocation." These slope failures are usually due to either sudden or gradual loss of strength of soil or a change in geometric conditions, such as steepening of an existing slope. There are several different types of slides and these different types of slides are characterized by their failure surfaces:

#### 7.6.1.1 Skin slides

Skin slides are relatively shallow slides that occur within 5 feet of the surface of embankments or within cuts composed of high-plasticity clay. While the overall stability of properly designed and constructed clay embankments is generally adequate, experience indicates that the outer layers of the embankments experience loss of strength over time. The loss of strength is due to the constant wetting and drying cycles, which cause sloughing and shallow slide failures. These shallow failures typically occur between 10 and 20 years after construction and represent a costly maintenance problem.

#### 7.6.1.2 Circular slides

Circular slides are caused by the rotation on a curved slip surface approximated by a circular arc. These are the most common slide surface which represents an estimated 80% of all slide geometries.

#### 7.6.1.3 Block failures

Block failures are caused by the displacement of a wedge-shaped mass along one or more planes of weakness. These slides are common on weakened planes and where clays seams cause a weak plane to form.

#### 7.6.1.4 Deep-seated failures

Deep seated failures occur when there is complete failure of the slope that extends deep into the foundation soils that support the embankment or slope. These are common where the underlying soils are soft and cannot resist the weight of the embankment.

#### 7.6.1.5 Rock Falls

Rock falls can occur due to both weak planes existing within a rock stratum and weathering of the rock after prolonged exposure to weathering elements. Normal failure mechanisms include dip-slip failures and slips along fracture and/or joints.

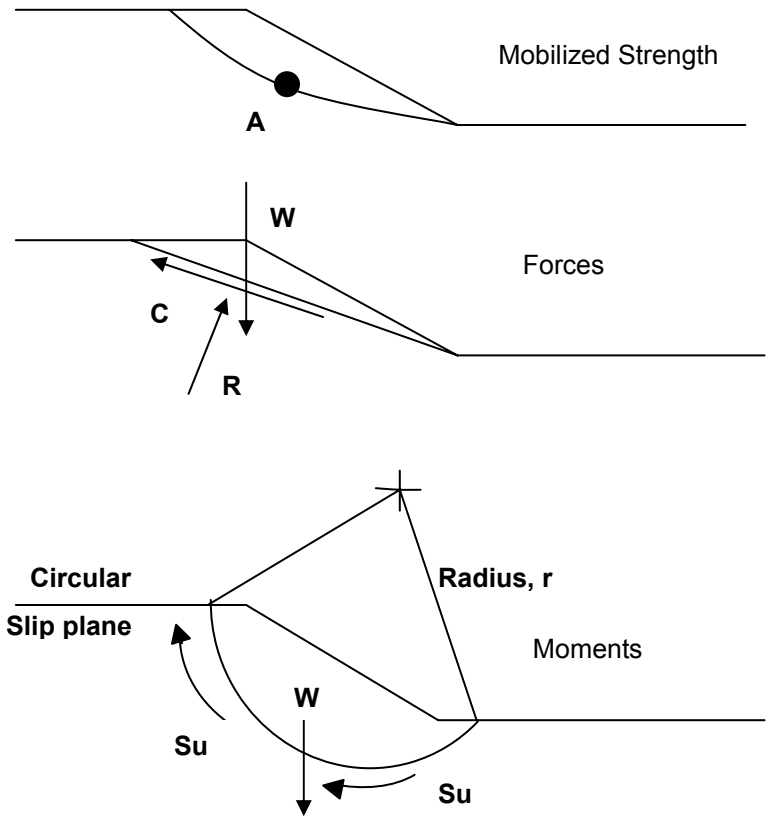
### 7.6.2 Modes of Failure

Factor of Safety (FS) is defined as:

$$FS = \frac{\text{Sum of Resisting Forces or Moments}}{\text{Sum of Overturning Forces or Moments}}$$

Factor of Safety can be based on the limit equilibrium method of mobilized strength, ratio of total existing forces, or ratio of total driving moments as shown below:

.Figure 7.4 – Failure Modes



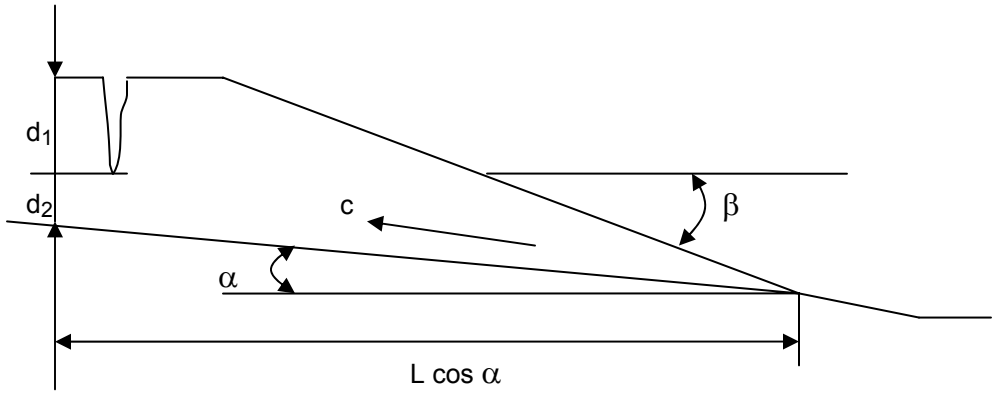

### 7.6.2.1 Noncircular Failure

Skin slides and block slides are types of noncircular failures. Block slides occur in soils with differing strengths and in soil subgrade overlying stronger rock-like material or wherever there is a weak zone in the profile.

Geologic conditions also govern noncircular failures. Cohesionless soils, colluvial soils over shallow rock, and fissured clays typically produce block failures. Block failures typically occur in faulted or slickensided soils, fissured and stratified soils, and interbedded dipping rock.

Block analysis is used to estimate FS as shown in Figure 7.5. The noncircular failure surface develops where the weak foundation is relatively thin. Block failure can be analyzed by calculating the Factor of Safety through the summation of forces.

Figure 7.5 – Block Analysis



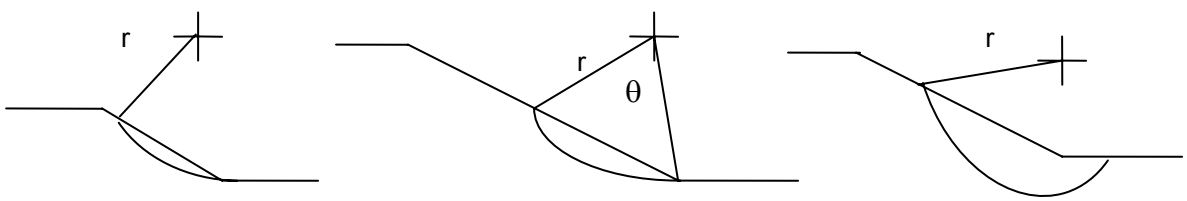

$$FS = \frac{cL + W \tan \phi + d_2 c}{W \sin \alpha}$$

where c = cohesion
$\phi$ = angle of internal friction
W = weight of block
L = length of slip surface

#### 7.6.2.2 Circular Failure

Circular failure occurs as a progressive mechanism where stresses concentrate then shed as the circle begins to fail. Failure typically occurs as shallow failure in homogeneous soil conditions. About 80% of slope failures are circular, and they fall into three classifications: skin, toe, and base (see Figure 7.6).

Figure 7.6 – Circular Failures




Skin Slide Toe Failure Base Failure

### 7.6.3 Approximate Analysis Method for Temporary Slopes

This method should be used only as a rough calculation to determine the stability of temporary, homogeneous slopes. Cohesive soils are modeled as having phi = 0. For temporary conditions, cohesion (c) of soil can be determined from an unconfined

compression test where c = qu/2. The factor of safety (FS) can be calculated as follows:

$$H_c = \frac{4c}{\gamma} \frac{(\sin \beta \cos \phi)}{(1 - \cos (\beta - \phi)}$$

$\beta$ = slope angle

$\phi$ = angle of internal friction

When $\beta$ = 90 and $\phi$ = 0;

$$H_c = \frac{4c}{\gamma} \qquad FS = 1 = \frac{c}{4\gamma H_c}$$

Although this is an approximate method, it can be used for several analyses, including skin slides, granular dry slopes, and temporary slopes.

Slopes with a homogeneous profile, which extends a relatively long distance can be evaluated with Infinite Slope Analysis. Infinite slope analysis is independent of slope height and depth, assume c = 0 and effective stress governs. Therefore FS can be defined as:

$$FS = \frac{\tan \phi}{\tan \beta}$$

$\phi$ = angle of internal friction
$\beta$ = slope angle

### 7.6.4 Circular Surface Analysis

There are two closed-form solutions that ignore all internal forces for circular failure surfaces. These solutions are very approximate and should be used only for initial calculations.

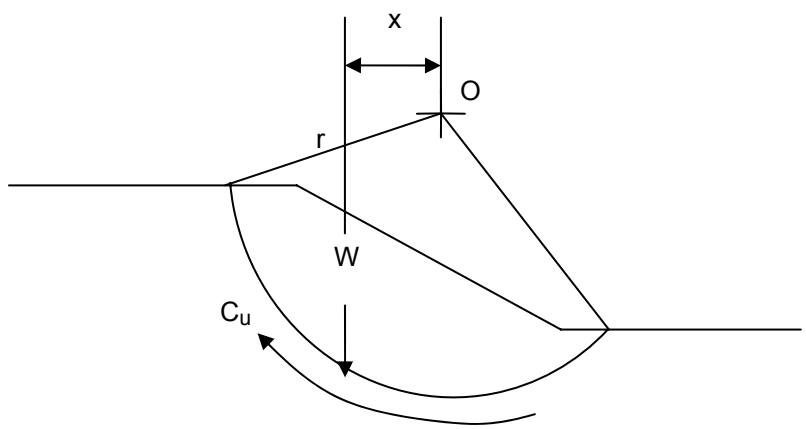
The next level of analysis involves dividing the failure circle into a number of slices. Each segment is analyzed, and the results are summed to calculate the Factor of Safety. Analyses of circular failure surfaces are complicated due to the fact that using static equilibrium and summing forces and movements yields an indeterminate solution for Factor of Safety. There are more unknowns than equations, and the solution cannot be obtained without making assumptions and fixing some variables. This leads to two types of solutions: “approximate” and “accurate.” Approximate solutions tend to be closed-form calculations which ignore the interstitial forces. Accurate methods use

an iterative solution and makes assumptions regarding the inclination of interstitial forces.

#### 7.6.4.1 Circular Arc ($\phi$ = 0 method)

This failure surface is a rigid cylindrical plane that rotates about its center and assumes no internal friction angle ($\phi$ = 0). The shear strength along the failure surface is defined by the undrained strength where only short-term total stresses are considered. Factor of Safety is calculated as a ratio of overturning against resisting moments:

Figure 7.7 – Circular Arc ($\phi$ = 0 method)




Factor of safety is calculated as a ratio of overturning against resisting moments:

$$FS = \frac{C_u L r}{Wx}$$

Where

$C_u$ = undrained shear strength
r = radius of circular surface
W = weight of sliding mass
x = horizontal distance between center of failure surface and center of the sliding mass
L = arc length

#### 7.6.4.2 Friction Circle Method

This method is applicable for homogeneous soils where the friction angle is greater than zero in soils and where cohesion and friction is present. The method is based on satisfying the requirements for complete equilibrium of the normal and frictional component due to the mobilization of the failure surface. The resulting normal stress assumes to be concentrated at one point. Stability charts (Taylor, Spencer, Janbu, etc.) can be used to determine the stability factor to use in calculating the factor of safety or to determine the critical height or depth.

Figure 7.8 – Friction Circle Method

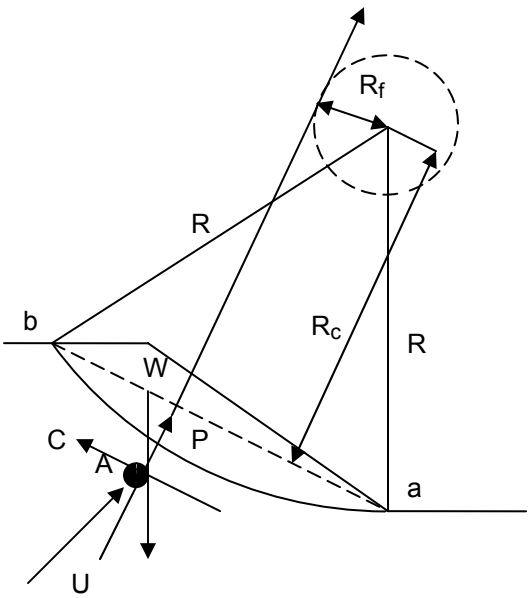
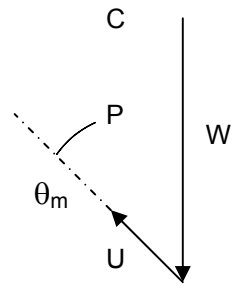



Figure 7.9 – Force Polygon




### 7.6.4.3 Method of Slices

The method of slices is based on static equilibrium. This method divides the failure mass into small slices and analyzes each slice (Fig. 7.10). The Factor of Safety is based on the sum of moments about the center of rotation and the sum of forces. The following diagram shows the forces acting on a typical slice (Fig. 11).

Figure 7.10 – Method of Slices

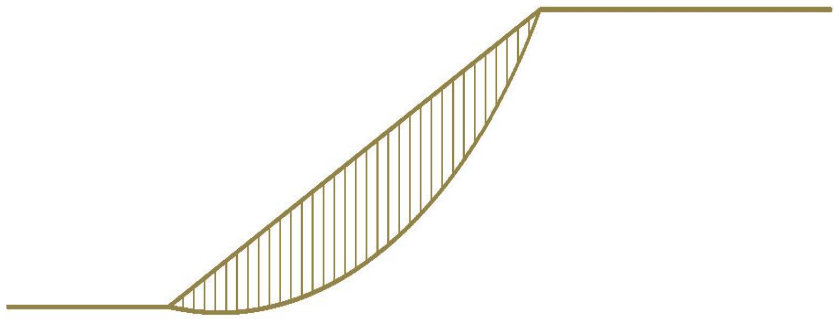
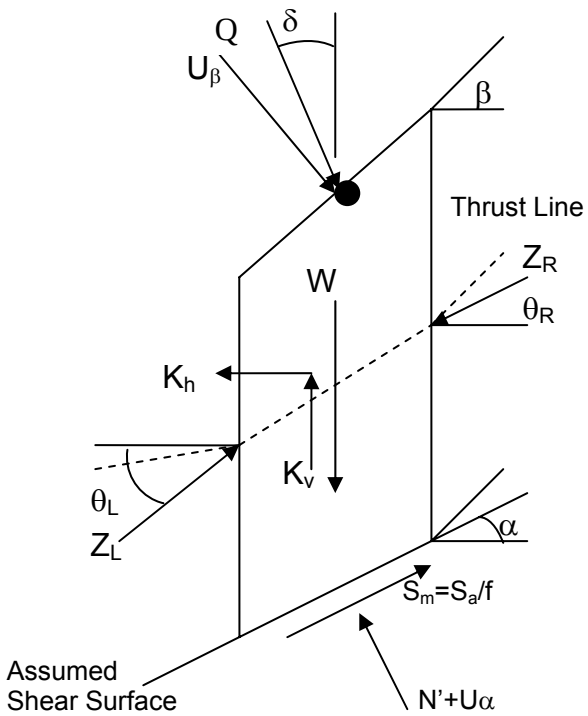


Figure 7.11 – Forces on a typical slice




$K_v$ = vertical seismic coefficient
$S_a$ = Available strength
= C + N' tan $\phi$
$S_m$ = mobilized strength
$U_\alpha$ = pore water force
$U_\beta$ = surface water force
W = weight of slice
N' = effective normal stress
Q = external surcharge

$K_h$ = horizontal seismic coefficient
$Z_L$ = left interstitial force
$Z_R$ = right interstitial force
$\theta_L$ = left interstitial force angle
$\theta_R$ = right interstitial force angle
$\alpha$ = inclination of slice base
$\beta$ = inclination of slice top
b = width of slice

A quick review shows there are too many unknowns to achieve a closed-form solution. Methods of solving this dilemma include ignoring some of the forces or assuming some values. This leads to the "approximate" and "accurate" or "rigorous" methods of solution.

The Ordinary Method of Slices and Simplified and Modified Bishop methods all ignore the side forces, or, more accurately, assume they are equal and opposite and therefore cancel out. As a result, these solutions are typically known as "approximate" solutions. Studies have shown that it is common for these solutions to vary as much as 30% from the more "accurate" solution methods.

#### 7.6.4.4. Ordinary Method of Slices (OMS)

OMS is an approximate method which neglects all interstitial forces and does not satisfy force equilibrium. This method considers only moment equilibrium. OMS assumes the resultant of interstitial forces is inclined at an angle parallel to the base of the slice. Side forces are assumed collinear and equal in magnitude. Since this approach ignores the presence of adjacent slices with different base inclinations, inconsistent effective stresses at the base of the slices occur. OMS can be used to calculate short term (total stress) and long term (effective stress) analysis. Pore pressures should be used to calculate effective stress.

#### 7.6.4.5 Approximate/Simplified Method

The Bishop Simplified Method assumes that all interstitial shear forces are zero. This method satisfies vertical force equilibrium for each slice and moment equilibrium of the entire circular failure surface mass. The Simplified Bishop Method is recommended for circular failure surface analysis only as a check or preliminary analysis.

This method can be hand calculated for most surfaces. The simplified method underestimates the FS by 30% and in other cases overestimates the FS by 5%, compared with the rigorous or "accurate" methods.

#### 7.6.4.6 "Accurate" Methods

The accurate methods make assumptions regarding the angle the interstitial forces act on the slice. This side force inclination assumption allows for a more accurate method of calculating the Factor of Safety. Researchers have come up with different methods for estimating the angle of side force inclination, and this has led to several "named" analysis methods. The best known of these are "Morgenstern and Price," "Spencer," "Janbu," and "Lowe and Taylor". Comparisons of the results from these "accurate" or "rigorous" methods show values generally within ±5%. Given the variability in soil strength and engineering properties, this analysis accuracy is more than sufficient.

### 7.6.5 Approved Computer Programs

The rigorous or "accurate" methods of analysis are far too cumbersome for hand calculation. Computer codes are relatively inexpensive and provide a significant speed and accuracy advantage.

Approved computer programs include UTEXAS4, SLOPEW, and GSTABL7. There are numerous other programs available from individual suppliers and universities. If the consultant desires to utilize other programs, permission of NTTA must be obtained in advance.

It should also be noted that all these programs contain both "approximate" and rigorous, or "accurate," methodologies. Only the "accurate" methods are appropriate for the design of permanent slopes.

### 7.6.6 Analysis

The selection of analysis method is dictated by the soil within the slope, the geometric configuration of the slope, and the anticipated shape of the failure surface. The following guidelines are provided to assist in determining the appropriate analysis method based on the shape of the failure surface:

1. Long uniform slopes with failure surface parallel to the ground surface: use Infinite Slope Analysis method.
2. Shallow, long planar failure surface that is not parallel to the ground surface: use "accurate" methods.
3. Planar failure surfaces: use Block Analysis method.

4. Circular or arc failure surface: use simplified Bishop or Taylor's Chart as rough estimate; however, confirm with "accurate" methods.
5. Circular, arc or composite failure surface: use any of the "accurate" methods.

To determine the shear strength of fine-grained soil, a distinction must be made between the short-term (undrained) shear strength and long-term (drained) shear strength.

#### 7.6.6.1 Short-Term (Undrained) Shear Strength

The short-term, undrained shear strength is applicable to conditions in which the time between the construction of slope and its failure is short enough that the soil does not have sufficient time to drain, i.e. slope failures that occur during or very shortly after construction. The shear strength is determined from UU unconfined compressive strength tests. The short-term stress analysis is applied to the following situations:

1. Analysis of temporary cut slopes of normally consolidated or slightly pre-consolidated clays. In this case, little dissipation of pore water pressure occurs prior to critical stability conditions.

2. Analysis of embankments on a soft clay stratum.

3. Rapid drawdown of water level providing insufficient time for drainage, typical of cut slope conditions. Use the undrained strength that corresponds with the overburden condition within the structure prior to drawdown.

4. End-of-construction condition for fills built of cohesive soils. Use the undrained strength of samples compacted to field density and at water content representative of the embankment.

#### 7.6.6.2 Long-Term (Drained) Shear Strength

The shear strength used for a long-term slope stability analysis is the drained shear strength. Drained means the water has had the opportunity to move out of the soil. In the long-term analysis method, the pore pressures along the assumed failure surface are estimated based upon static equilibrium conditions. The analysis uses effective strength parameters c' and $\phi$' that have been obtained from a Consolidated-Drained shear test or a Consolidated-

Undrained test with pore pressure measurements. The effective stress analysis is used in the following situations:

1. Long term stability of clay fills. Use steady state seepage pressures where applicable.

2. Short term or end-of-construction condition for fills built of free-draining sand and gravel

3. Rapid drawdown condition of slopes in pervious, relatively uncompressible, coarse grained soils. Use pore pressure corresponding to new lower water level with steady state flow.

4. Long-term stability of cuts in saturated clays. Use steady state seepage pressures where applicable.

5. Cases of partial dissipation of pore pressures in the field. Pore water pressures must be measured by piezometers or estimated from consolidation data.

### 7.6.7 Slope Analysis

Slope stability analysis of fill slopes should be performed to determine the slope of the embankments to reduce the chances of skin, circular, and deep-seated failures. Both long-term and short-term stability analysis should be performed and be based on strength values obtained by suitable laboratory tests on representative material and fill samples. Analyses must be performed on cut and fill slopes for both short and long term conditions.

Consideration should be given to the geology of the site. Geologic information should provide guidance for the selection of soil parameters and the minimum slope requirement for an embankment or a cut. Strength data should come from testing performed on both natural (in-site) and compacted samples (fill). The configuration should model the site conditions at the alignment.

Groundwater and associated seepage pressures produce destabilizing forces, which are the most frequent cause of slope instability. Subsurface water seeping toward the face or toe of a slope can be evaluated by flow net diagrams. The stability analysis should also

consider groundwater that is present along the assumed failure surface. When compressible fill material is used for the construction of the embankment, excess pore pressure could develop. This must also be considered in the stability analysis. Similarly, when embankments are constructed on compressible soils, the foundation pore pressures must be considered in the stability analysis.

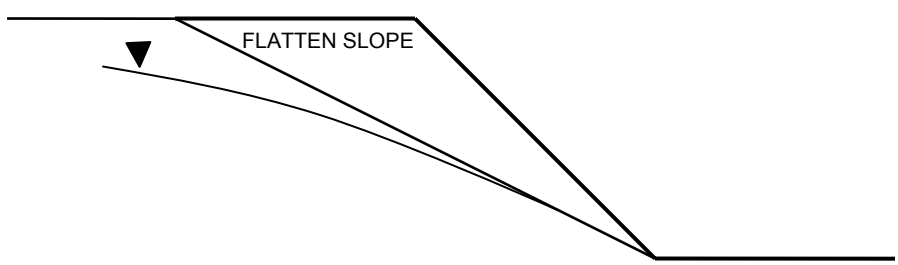
Stepped slopes are generally applicable on steep-cut slopes where flattening of the slope is difficult and surface sloughing occurs. Stepped slopes also help in controlling erosion and small failures. For rock slopes, steps help to catch and hold rockfall and reduce the probability of damage to the traveling public and the roadway.

### 7.6.8 Stabilization of Slopes

The stabilization of marginally stable slopes is limited to either unloading the slope to reduce driving forces, loading the toe to increase resisting forces, or draining the slope to increase soil strength. There are no other methods.

Flattening and/or benching of the slope, removal of a load from the head of the slope, or adding a load at the toe of the slope (as with the construction of an earth berm), will increase the stability of the slope. Slope stability analysis should be performed to determine the most effective solution.

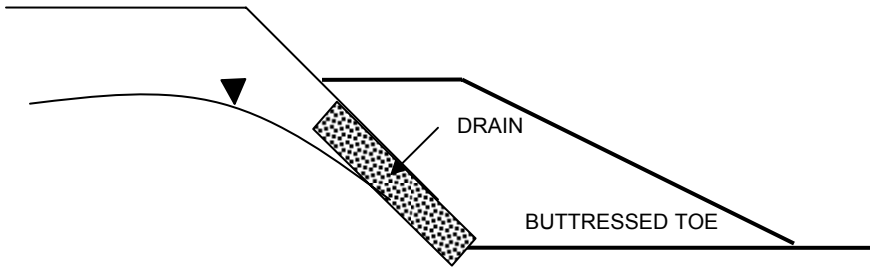
Figure 7.12 – Stabilization using Cut/Fill Geometry Change




Surface drainage decreases infiltration of water to potential weak planes within an embankment. Lowering of the groundwater increases the effective stress and increases

the strength of soils. Horizontal and/or interceptor drains are other effective means to control groundwater and improve the stability of slopes.

Figure 7.13 – Drain/Buttress




### 7.7 Skin Slides

Skin slides are typically small slides that occur in earth embankments; the local experience shows slides are typically less than 5 feet in depth and occur up to 20 years after construction. Skin slides frequently occur in embankments less than 30 feet in height constructed with clay soils having liquid limits 50 or greater and where slope ratios are as flat as 3 horizontal to 1 vertical.

The fundamental cause of the skin slides is the wetting/drying cycles on the highly expansive soils. These cycles cause a reduction in strength with each cycle of wetting and drying. When the slope becomes saturated and the forces increase enough, a small shallow slide occurs.

The Wright[2] study included various correlations between shear strength and simple index properties and to determine the applicability to these conditions in Texas. The same correlations can also be useful for making preliminary estimates for new slope design. The study also recommends that a water table coincident with the surface of the slope be assumed when the correlations are used. Only long-term strength conditions of highly plastic clays were examined.

[2] A.A. Saleh and Stephen G. Wright, "Shear Strength Correlations and Remedial Measure Guidelines for Long-Term Stability of Slopes Constructed of Highly Plastic Clay Soils," Center for Transportation Research, The University of Texas at Austin, October 1997.

The applicable shear strength for slopes in highly plastic clays that have not failed previously is the fully softened shear strength. Although the Mohr-Coulomb envelope for fully softened shear strengths is typically curved, the shear strength can be expressed using a 'secant' friction angle, $\phi_s$, and the equation;

$$s = (\sigma - u) \tan \phi'_s$$

where $\sigma$ = total normal stress, and

u = pore water pressure

The secant friction angle depends on the effective normal stress ($\sigma$-u) and the liquid limit of the soil. The relationship between the fully softened secant friction angle and the liquid limit and effective normal stress is shown on Figure 7.14. To determine the secant friction angle, the following steps are used:

1. Determine the liquid limit of the clay from laboratory tests.

2. Estimate the normal effective stress along the potential sliding surfaces. Typically the slides are shallow. A good approximation is that the slide depth will be 20% of the slope height. Thus:

   Slide Depth = 0.2 x H

   H = the slope height

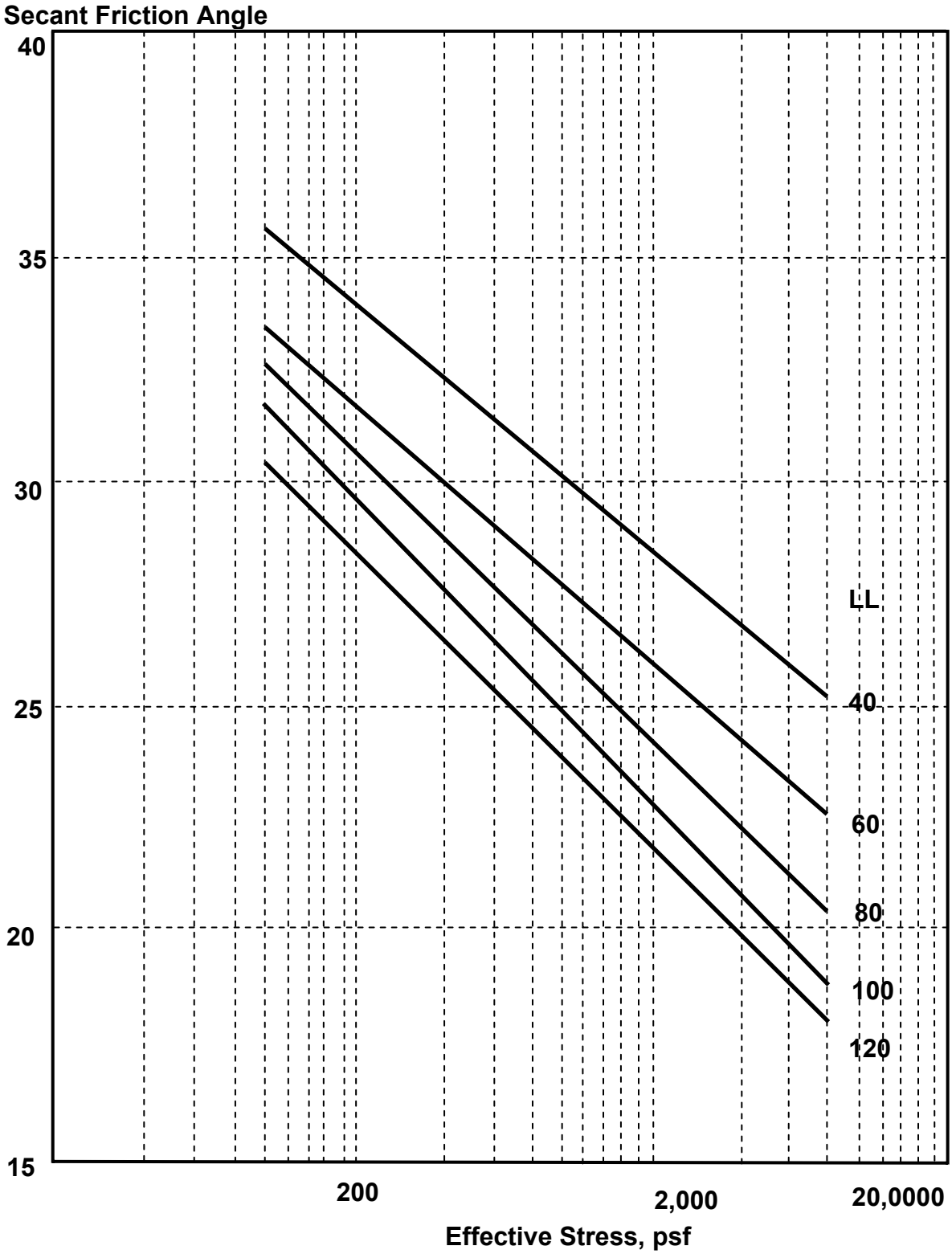
3. The effective normal stress can be estimated to be approximately 50 psf per foot of the slide depth. Thus, the nominal effective stress ($\sigma'$) along the slide surface would be:

   $$\sigma' = (50)\ (0.2)\ H = 10\ H$$

For a 20 foot high slope, the effective stress would be 200 psf. This is the maximum effective stress along the slide surface. Once the liquid limit and the effective normal stress are estimated, the secant friction angle can be estimated from the figure. For example, for a liquid limit of 80 and effective normal stress of 200 psf, the friction angle is estimated to be approximately 31 degrees. The friction angle estimated can be used in slope stability analyses

for new slopes and for existing slopes that have not failed previously using the infinite slope analysis.

FIGURE 7.14 – Secant Friction Angle And Effective Stress Relationship



### 7.7.1 Mitigation Measures

Some of the common remedial methods for skin slides include:

1. Flattening the slope to a minimum 4H:1V or flatter based on a stability analysis. Sufficient right-of-way must be available to flatten the slope.

2. Regrading the slope to provide better drainage of the surface water away from the face and behind the crest of the slope. This method is one of the most cost-effective ways of improving stability because it reduces the rate of degradation. It does not however, completely eliminate the risk of failure, only delays the failure

3. Removing a minimum of 5 feet of highly plastic clay soils (LL>50) for the slope surface and replace with low plasticity clay soils (PI<40). The clay soils (PI<40) should be placed at proper compaction and will protect the high plasticity softening susceptible soils.

4. During the construction of the embankment, avoid placing highly plastic clay soil (PI<50) near the surface of the slope.

## 7.8 Rock Fall

Rock fall can occur due to weak planes existing within a rock stratum and also due to weathered of the rock after prolonged exposure. A specialist should be consulted for projects with rock falls or potential rock fall hazards.

## 8.0 Pavement Design

Pavement design methodologies fall into two categories: empirical and theoretical. Empirical design methods are the most familiar and include the TxDOT, AASHTO, Asphalt Institute, Portland Cement Association, and most other common design methods. Empirical designs are not based on theory or mechanics, but rather on observations and historical reference. Typically these designs will say "based upon how road X performed over the past 20 years, road Y being designed should perform just as well since conditions are similar."

Unfortunately, conditions are never really similar. Subgrade soils are different; they have differing strength, swell/consolidation, and index properties. Construction materials are not the same today as they were 20 or more years ago; asphaltic cement is graded in a different manner and does not have the same adhesion of the cements of 40 or more years ago.

Aggregates being used today are often not as good as those of 1980s and 1990s. And most of all, traffic today bears little resemblance to the trucks and tires of 20 years ago. All of these problems are further exacerbated by the fact that most of the design methods are between 40 and 60 years old, not just 20 years old. Therefore, designs based upon empirical methods are certainly not guaranteed to perform well; in fact, they often fail within the first 5 to 10 years of service life. Some design methods that have been adjusted to compensate for traffic and materials produce unrealistically thick designs that cost too much and waste resources.

Beginning during the 1950s, a new approach to design appeared. This approach was based on "theoretical" stresses and strains in the pavement system imposed by real traffic loading conditions. These approaches first appeared as Portland cement concrete designs based on the Westergaard stress distribution. Since then, a significant effort has been made to refine and test the theory behind these methods using fatigue testing to model pavement systems as well as careful monitoring of full-scale test sections under live traffic. The result has been the development of design analysis and prediction tools that model the failure mechanisms found in real-world pavements and compare predictions with performance histories to verify the model.

These tools have given rise to a new design methodology known as “mechanistic designs.” Mechanistic Design Methods are actually comprised of a system of analysis tools that predict the performance of a given pavement system. Should the predicted performance be unsatisfactory, the design is changed and the analysis conducted again. This cycle is repeated until each design parameter is met and every performance measure is satisfied for the design life of the system. It is this systematic approach that is detailed in this design manual.

Mechanistic analysis mathematically models the behavior of the pavement system under imposed stresses. The resulting stresses and strains can be compared to fatigue models to determine the pavement fatigue life and rutting potential for flexible pavements as well as cracking and punchouts for rigid pavements. There are many different models currently available. The models use different assumptions for the material behavior, such as linear-elastic, viscoelastic, non-linear, dynamic, etc. Most of these models are still in the development stage and are too complex for design purposes. For the purposes of this manual, only the “Layered Elastic Analysis” model (LEA) and the “Finite Element Model” (FEM) are considered. These models can be run on a personal computer and are readily available either as freeware (WESLEA, EVERFE, MICHPAVE, LEDFAA) or for purchase (ISLAB2000, CIRCLY, SIGMAW and ABAQUAS).

The Layered Elastic Analysis computes stresses, strains, and displacements that result from surface loading. Analysis points can be located anywhere within the continuum. These models assume the pavement layers are homogeneous, isotropic, and linear-elastic. In other words, each layer is the same in all directions; strength (stress-strain) properties are the same in X, Y, and Z and will return to their original positions when the load is removed.

The Finite Element Method constructs a mathematical model of the pavement system that consists of elements that can have differing properties depending on the element’s location in the continuum and the stress applied. While this analysis technique is complicated, it is very useful for predicting behaviors not easily observed. It should be noted that FEM models must be calibrated and verified before they are used for design. Only the programs listed above are acceptable.

The model can also be linear elastic, viscoelastic or non-linear and can be non-isotropic and non-homogeneous depending upon the model and accuracy desired.

Each part of the field and laboratory investigation has been designed to pick design parameters to be incorporated into the final design. For example, a significant effort is expended to determine the movement characteristics of the subgrade soils supporting the pavement. Similar efforts are expended to determine the materials' strength in natural, compacted, treated, and stabilized states in order to obtain a rational basis for assigning strength and determination characteristics to the pavement supporting materials. At this point in the design process, we have determined the following:

1. Depth of moisture treatment necessary for expansive soil subgrades;
2. Stabilization method, additive, and concentration required for native subgrade, fill, and moisture treated subgrade;
3. Minimum design Resilient Modulus values for the subgrade, moisture treated subgrade, and stabilized zones supporting the pavement;
4. Grading requirements, including clearing and grubbing, compaction specifications, slope ratios for cuts and fills, and estimated bulking/shrinkage factors.

This final step in the process yields the actual pavement thickness design. In this section, we will detail minimum subgrade and intermediate layer thickness and minimum requirements, pavement design methodology, and specified input values.

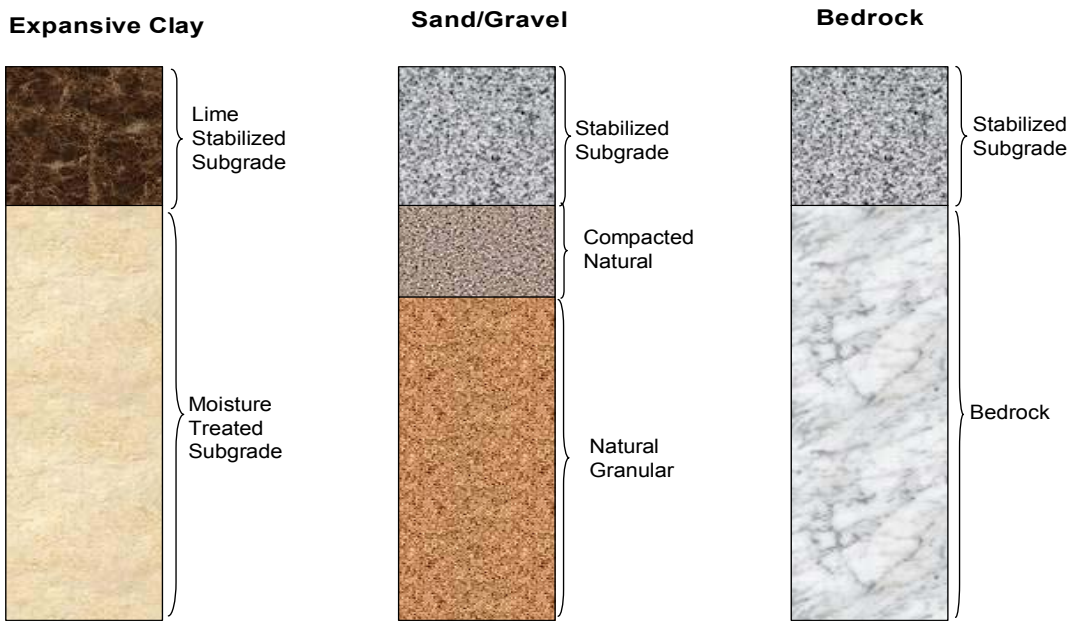
The pavement design process is designed to allow the engineer to select from several design sections, which could include asphaltic concrete, plain Portland cement concrete, and continuously reinforced concrete. For all three pavement types, both empirical and mechanistic design methodologies are included. The thickness of pavement layers and materials are largely a function of engineering judgment, availability, cost, contractor familiarity and experience. Pavement design can consist of as few as 2 materials or as many as 6. There is an infinite number of adequate designs. The trick is balancing initial cost, ease of construction, service life and maintenance.

## 8.1 Foundation of the Pavement System

The foundation system consists of the untreated subgrade, moisture treated subgrade, and stabilized subgrade. Cement-treated aggregate base course, aggregate base course, asphaltic concrete, or other materials above the foundation materials are considered intermediate layers.

In order to reduce the amount of variability in the design, standardized subbase and base sections have been developed and are to be used for all pavement sections. These sections are dependent upon the subgrade soils and pavement surface type. For all pavement types, a subgrade treatment consisting of moisture-treated subgrade or removal and replacement with non-swelling subgrade as defined in Section 6.2 is required for expansive subgrade. Likewise, all subgrade must be stabilized. In cases of very hard rock subgrade, it is necessary to have a stabilized layer to lessen the potential for a “hard spot” that can lead to premature failure of the pavement system. For the typical subsurface conditions, the foundation profiles will consist of the following:

Figure 8.1 Pavement Foundation




1. Stabilized subgrade shall have a minimum strength of 160 psi and >1% swell @ 200 psf
2. Clay soils shall be lime stabilized (cement may be added for strength).
3. Thickness of Moisture Treated Subgrade shall be determined from Section 6.2

## 8.2 Intermediate Layers

The type of intermediate pavement layers to be used will depend upon the surface type and the amount of traffic. Light traffic with asphaltic concrete surfaces could require no intermediate layers. Heavy traffic on a CRCP could require a layer of up to 12 inches of Cement Treated Base and a 1.5-inch layer of asphaltic concrete to achieve the design life and serviceability. The only acceptable intermediate layers for concrete-surfaced pavements are cement treated-aggregate base (TxDOT Item 276 Type N between 500 and 750 psi) and Type D asphaltic concrete (TxDOT Item 342 Binder Grading D PG 64-22). These materials should be analyzed assuming Resilient Moduli values of 50,000 psi for CTB and 300,000 psi for ACCP, or as measured.

For asphaltic concrete surfaced pavements, the intermediate layers could consist of these approved materials: flex base (TxDOT Item 247), Asphaltic cement base (TxDOT Item 340

Grading B PG 64-22), and Cement Treated Aggregate base (TxDOT Item 276 Type N). Design and analysis Resilient Moduli values for these materials are: Flex Base $M_R$= 25,000 psi; ATB Type B $M_R$=40,000 psi; and CTB $M_R$=50,000 psi, or as measured.

## 8.3 Surface Layers

Asphaltic Cement Concrete Pavement (ACCP) and Portland Cement Concrete Pavement (PCCP) are acceptable surfacing materials. ACCP is acceptable only for frontage roads and cross roads in the NTTA system. PCCP consists of three pavement types: Plain, Jointed Reinforced, and Continuously Reinforced. Only Continuously Reinforced is acceptable for mainlanes within the NTTA system. Plain PCCP can be used only for frontage roads and cross roads. Jointed Reinforced PCCP is discouraged.

## 8.4 Materials Design Properties

The following strength coefficients and Resilient Modulus values are to be used in the design of all pavement systems:

Table 8A Material Properties

| Material | Resilient Modulus ($M_R$) | Structural Coefficient (a) |
|---|---|---|
| Moisture Treated Subgrade | 3,000 psi | --- |
| Subgrade/Replacement Fill | As measured | --- |
| Stabilized Subgrade | 30,000 psi | 0.14 |
| Flexible Base (ABC) | 25,000 – 50,000 psi* | 0.12-0.17* |
| Cement Treated Base | 50,000 psi | 0.17 |
| High Asphalt ACCP Layer | 150,000 psi | 0.35 |
| Structural ACCP Layer | 300,000 psi | 0.40 |
| SMA-Wearing Course | 350,000 psi | 0.44 |

*Higher values must be totally confined and requires justification.

## 8.5 Pavement Traffic Matrix

Pavement thickness analysis for low, moderate and high risk traffic categories for trucks with 80 kips and 97 kips GVW should be performed using ESALs from the traffic matrix developed according to Section 5.2.3.3, unless the governing entity specifies a single design ESAL value. A pavement thickness matrix similar to the traffic matrix will be the result of the pavement thickness analysis is as follows:

Table 8B Thickness matrix

| Pavement Thickness (inches) | | |
|---|---|---|
| Risk | 80k GVW | 97k GVW |
| High | 9.0 | 10.0 |
| Moderate | 9.5 | 10.5 |
| Low | 10.5 | 11.0 |

The pavement thickness matrix should be developed for mainlanes and may be appropriate for other roadway categories. Life Cycle cost analyses can then be performed if different surface types are represented. Selection of the final design thickness can be made based on the risk and cost either initial or Life Cycles of the various sections.

## 8.6 Pavement Design Philosophy

The pavement surface selected for each roadway type is dependent on the expected performance and the maintenance agreement. Mainlane roadways and ramps that will be owned and maintained by NTTA shall be designed using the mechanistic approach as high performance Continuously Reinforced Concrete Pavement (CRCP). Frontage roads or cross roads will typically be ACCP, PCCP, or CRCP where the design methodology follows AASHTO or TxDOT requirements or mechanistic for NTTA projects. The entity controlling the roadway will provide design requirements, design methodology, and pavement type required. The following sections provide pavement design guidelines for empirical and mechanistic design procedures for Plain Portland Cement Concrete, Continuously Reinforced Concrete, and Asphaltic Cement Concrete Pavement.

### 8.6.1 Empirical Design Methodology

The approved empirical design methodologies are TxDOT and AASHTO (1993). For some entities, such as TxDOT, these designs serve as the final design for the pavement section. For NTTA, these serve only as the initial starting point of trial designs. In the following sections, each design method (ACCP, PCCP, and CRCP) will be discussed and protocols will be provided.

It should be noted that for TxDOT designs, moisture-treated subgrade may not be an acceptable mitigation method. In these cases, a removal and replacement option will be required. The depth of removal and replacement will depend on the analysis method accepted by the TxDOT district, usually the PVR (Potential Vertical Rise) TEX-124E, or the Effective PI methodology. Designer should check with the district Pavement Engineer for direction.

### 8.6.2 Mechanistic Design Methodology

The mechanistic design procedure is not truly a design since the result of a series of calculations is not a complete design as it is in the case of empirical design methodologies. Mechanistic designs are developed by beginning with an estimated design which is analyzed for specific performance criteria adjusted as necessary, reanalyzed and adjusted until all criteria are met. In this design procedure, the initial trial designs are typically developed using empirical design methods, usually AASHTO. The designs are then analyzed using appropriate tools and adjusted. The number of iterations is substantially reduced by using the foundations presented in Sections 8.1 and 8.2; therefore, adjustments to only 1 or 2 layers are necessary.

Analysis methods approved for mechanistic designs include Layered Elastic Analysis (LEA) packages WESLEA, CIRCLY, CHEVPC, LEDFAA, and MICHPAVE. Finite Element Method (FEM) packages ABAQUAS, EVERFE, SIGMAW, ISLAB 2000, and TTIPAVE are also accepted. Only the NTTA mechanistic design spreadsheet by Dr. Dan Zollinger is acceptable for Continuously Reinforced Concrete Pavements.

CRCP is Portland Cement Concrete Pavement (PCCP) constructed with continuous longitudinal steel reinforcement with no transverse contraction joints. The transverse cracks develop due to a stress-relief process from drying shrinkage and seasonal weather conditions. CRCP performance lies in the maintenance of uniform support and transverse crack widths sufficiently tight for providing adequate stiffness and load transfer.

Failures in PCCP are manifested as "D" or corner cracking and tensile failure of the concrete. CRCP failures are almost always manifested as a punch out, and the mechanism is completely different; the design/analysis should reflect the failure mechanism.

The Mechanistic design approach for Continuously Reinforced Concrete Pavement (CRCP) addresses the rational relationships between design parameters and pavement response under traffic and environmental conditions. The AASHTO empirical design ignores the failure mechanism and treats the design exactly the same as it does for plain or unreinforced Portland cement concrete. AASHTO gives no credit for the reinforcement.

## 8.7 Asphaltic Cement Concrete Pavement (ACCP)

The design philosophy for ACCP systems is to begin with the softest layer, the moisture treated or untreated subgrade, and to gradually increase stiffness to the final surface lift or wearing course. The design life of ACCP is 20 years. Regardless of the swell mitigation selected or the character of the subgrade, stabilization of the subgrade as outlined in Section 8.1 is required. Selection of materials for the intermediate layers shall be in accordance with Section 8.2. Thicknesses and type of the surface courses shall be dependent on the wishes of the controlling entity. Typical alternatives include:

a) TxDOT Type C or D full depth,
b) SMA surface over Type C or D,
c) SMA over Type B asphaltic base, or
d) "Perpetual Pavement"

### 8.7.1 ACCP Empirical Methodology

Traffic analysis is described in Section 5.23 ACCP Traffic Analysis. The ESALs should be derived using the Load Equivalency Factors for ACCP. The measured or lower bound subgrade Resilient Modulus ($M_R$) of 3,000 psi should be used to determine the minimum acceptable Structural Number (SN).

Given the Design ESALs or Traffic Matrix and the Subgrade Modulus, the design SN can be determined from the AASHTO charts or DARWIN. Using the SN, structural coefficients listed below and the maximum sections presented in Section 8.6.2, a design section can be easily calculated as follows:

1) The wearing course should be between 2" (minor cross roads) to 4" (moderate to heavy traffic) of SMA or Performance mixture SP-D or CMB-F mix using PG76-22 having an $M_R$=350,000 psi or more which provides for a structural coefficient of a=0.44.

2) The second ACCP layer should consist of a dense graded SHRP fine side mix utilizing PG 64-22 Asphaltic cement and having a minimum dynamic modulus of 300,000 psi. This layer of ACCP will have an assigned structural coefficient of a=0.40. In the "Perpetual Pavement" design, a third, high AC layer is provided. This layer must be at least 2" in thickness and have $M_R$=150,000 psi (a=0.35).

3) The layer immediately below the ACCP should be a stabilized layer consisting of either cement treated aggregate base, lime stabilized subgrade, or cement stabilized subgrade. Subgrade strength and support values for pavement design are determined using Resilient Modulus testing as described in Section 4.4.4 and summarized in Section 8.4 (Table 8A). The following minimum values are recommended to be used in the design:

Cement treated aggregate base ($M_R$=50,000 psi and a=0.17)
Lime stabilized subgrade ($M_R$=30,000 psi and a=0.14)
Cement stabilized subgrade ($M_R$=30,000 psi and a=0.14).

4) Flexible base between the ACCP and a stabilized subgrade can also be very effective, if properly designed. The thickness of this first underlying layer should be 1.5 to 2 times the combined thickness of the ACCP layers (maximum of 8 inches). When sandwiched between ACCP and stabilized subgrade, the resulting effective $M_R$ approaches 50,000 with an effective of a=0.17.

5) The upper 12 inches of any moisture-treated material must be stabilized with lime designed according to Section 4.4.6. Granular or non-swelling subgrade can be stabilized with cement (Section 4.4.6). These stabilized zones should be treated as discussed in Item 3 above.

6) The Design philosophy should follow these rules of thumb:

a. Thickness (D) must be equal or up to 1.5 times the thickness of the overlying layer.

b. Resilient Modulus ($M_R$) values should decrease with depth in the design section.

c. Minimum $M_R$ value is that of the natural or moisture treated subgrade.

d. The ACCP is a flexible pavement system and must be allowed to move slightly in order to transfer load, thus there shall be no hard layer in the system.

#### 8.7.1.1 ACCP Design Procedures

The SN value represents the sum of the various structural layers multiplied by the assigned structural coefficients. For example:

$$SN=a_1D_1, + a_2D_2 + a_3D_3 + \ldots$$

Where:

SN = Structural Number developed from design charts

$a_1, a_2, a_3$...= Structural coefficients for supporting layers

$D_1, D_2, D_3$…= Thickness in inches of the ACCP and supporting layers.

Start with a trial pavement section for the analysis with estimated thickness of each layer of the pavement section. Select the recommended structural coefficients, drainage

coefficient, traffic and one direction pavement width. Perform the analysis to calculate the SN. The design is adequate when the design chart SN is less than the calculated SN. If design is inadequate, modify the pavement section thicknesses and perform the analysis again until the design SN is met. Total thickness of the section should be no thicker than the sections shown in Section 8.7.2.

### 8.7.2 ACCP NTTA Methodology

The NTTA requires the use of a "Perpetual Pavement" design as prescribed by the Asphalt Pavement Alliance. This design philosophy is to provide a three-layer ACCP system consisting of a wearing course, a substantial structural layer and an "asphalt rich" layer to absorb elastic strains. The design limits the strain in the lowest asphalt layer to 70-200 microstrain units. The initial design can be developed using the PERROAD computer code. Intermediate layers and thicknesses should be designed using the AASHTO equation as presented in Section 8.7.1.

Once the trial pavement section has been determined, it should be subjected to mechanistic analyses to assess maximum stresses at the bottom of each layer. Layered Elastic Analysis (LEA) or Finite Element Method (FEM) can be used to provide these analyses. Estimated fatigue life of the pavement system based upon the generally accepted Finn fatigue equation.

$$\text{Log } N_f = 15.947 - 3.291 \log \left( \frac{\varepsilon_t}{10^{-6}} \right) - 0.854 \log \left( \frac{E_{ac}}{10^{3}} \right)$$

Where: $N_f$ = number of cycles to failure (fatigue)
$\varepsilon_t$ = horizontal tensile strain at the bottom of the ACCP layer
$E_{ac}$ = composite elastic modulus of the ACCP

$$N_f = 1.0787 \times 10^{18} \left( \frac{10^{-6}}{\varepsilon_v} \right)^{4.4843}$$

Where: $N_f$ = number of cycles to failure (rutting)
$\varepsilon_v$= vertical compressive strain at the bottom of the ACCP layer

After Finn (1977)

The Predicted Service life based on fatigue and rutting can be calculated by dividing the predicted fatigue and rutting pavement life by the number of cycles applied per year. The number of cycles applied per year is calculated by dividing the number of ESALs by the number of years in the design life. Although this ignores the growth in traffic, it is an adequate approximation given the other simplifying assumptions used in the analyses.

After the fatigue criteria is satisfied, stresses in the pavement system should be calculated for the design truck using the maximum applied axle load and tire pressure using a Layered Elastic Analysis (LEA) or Element Method (FEM) technique. This analysis should be performed for the worst-case design truck (22,000 lbs. load on a 150 psi radial tire). This analysis will result in a stress distribution such that the outer edges of the tire apply 450 psi to the pavement and the stress in the center is negative. This can be modeled using FEM codes. Where the code does not permit this, assume a uniform 150 psi tire stress. The load should be 11,000 pounds for a "Super Single" representing a 97 kips GVW.

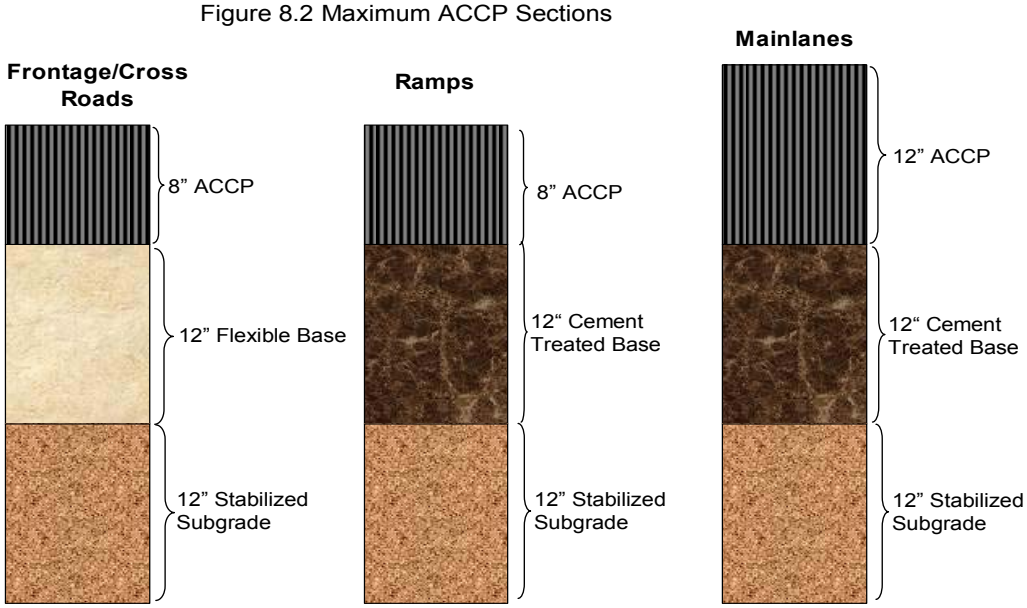
For layers that have high stresses at the bottom, the thickness should be increased and reanalyzed until all pavement layers have acceptable stress levels in the linear portion of the stress-strain curve (less than 40% of yield strain). The resulting pavement section becomes the design pavement system for the traffic and material properties are developed and analyzed. This design/analysis procedure is appropriate for the design of most low- to moderate-traffic-level roadways.

In many cases, the traffic loads will be high enough to result in unrealistically thick pavement systems. The root cause of this is the fact that the AASHTO equation was developed from a test using low traffic loading. Today, our trucks have 2.5 times the load and 2 times the tire pressure with 3 times the contact stress. This results in traffic loads not contemplated by the AASHTO road test. These very high traffic values, when used with the current AASHTO equations, result in the thick pavement systems.

When the maximum thickness pavement (Fig. 8.2) is exceeded by the design, a staged construction and maintenance program must be instituted. Typically, these programs require the pavement to be overlaid at a reduced interval, say 4–5 years with

mill and inlay at 5–9 years to maintain the integrity and service life of the pavement. The intervals are determined from mechanistic analyses, which can predict service life/distress as a result of load applications. Maximum thicknesses of ACCP are as follows:

Figure 8.2 Maximum ACCP Sections




## 8.8 Portland Cement Concrete Pavement (PCCP)

Portland Cement Concrete Pavement (PCCP) can be plain jointed (unreinforced), or jointed reinforced.

Jointed Reinforced Concrete Pavements (JRCP) have steel within the slab. The steel does not cross the joints and is light in comparison with Continuously Reinforced Concrete Pavement. While these pavements are used for municipal streets in the Dallas-Fort Worth area, neither NTTA nor TxDOT allows them on their systems. Therefore, JRCP will not be discussed in this Design Manual and PCCP will refer to Plain Jointed Pavement only.

Plain PCCP systems are slabs of unreinforced concrete with longitudinal and transverse joints typically spaced from 10 to15 feet on center. Dowels are used in the transverse joints to transfer load from one slab to the other. Dowels are smooth steel bars ranging from 1 to 2 inches in diameter and 12 to 24 inches in length that are typically placed across the joints at 9 to 18 inches on center. PCCP pavement details provide dowel bar size and spacing, joint spacing, and details for longitudinal joints based on the pavement thickness.

The most common failure mode for PCCP systems is fatigue cracking at the mid-slab location due to tensile stresses or corner cracking due to loss of support. A secondary but common failure is loss of support due to subgrade erosion, which can manifest itself with movement of the entire slab or rocking. PCCP will provide exceptional performance as long as the support is non-yielding and uniform. Having a non-erodible foundation system is critical to obtaining a long-life pavement with high serviceability. The PCCP system should have a stabilized subgrade and, where necessary for structural reasons, cement-treated aggregate base.

Plain PCCPs are useful and can be designed to take exceptional loads and high-traffic volumes. The best illustration is Denver International Airport, where all runways, taxiways, and aprons are plain PCCP. Dowels are used to transfer load between slabs. These slabs are 17 inches of plain PCCP supported by 8 inches of Cement Treated Base and 12 inches of Lime Stabilized Subgrade over a soft, moist clay subgrade. This pavement section was designed for a 40-year Service Life and supports aircraft weighing upwards of 1,200,000 pounds.

### 8.8.1 PCCP Empirical Design Methodology

PCCP is designed using conventional AASHTO methodology with a 30-year design Service Life. Conventional chart solutions from the 1993 Guide and computer programs as DARWIN and TSLAB86 can be used to determine the design thickness. The design 30-year ESALs can be determined using the computer program DARWIN or the Load Equivalency Factors for rigid pavements as presented in Section 5.

Depending upon the entities controlling the pavement, design parameter selection for traffic analysis, material properties and performance criteria should conform to either

the ASSHTO Guide for Design of Pavement Structure or the Texas Department of Transportation Pavement Design Guide (Revised October 2006). According to TxDOT, performance criteria and material properties for rigid pavement design are as follows:

Table 8C PCCP Design Minimums

| Design Parameters | Design Values |
|---|---|
| Reliability | 95% (Dallas District)<br>Or 99% (Fort Worth)* |
| Initial Serviceability | 4.5 |
| Terminal Serviceability | 2.5 |
| 28-day Mean PCC Modulus of Rupture | 620 psi |
| 28-day Mean Elastic Modulus of Slab | 5,000,000 psi |
| Mean Effective k-value | 300 psi/in** 120 psi/in |
| Overall Standard Deviation | 0.39 |
| Load Transfer Coefficient | 2.9 |
| Overall Drainage Coefficient | 1.0 |

NOTE: Values should be used as a default unless there is justification to change it (i.e. actual testing and measurement data).

*Verify with the respective TxDOT district that will be reviewing the project design.

**This value is highly suspect. Full-scale data load testing on Stabilized Subgrade suggests values of 120–160 psi are more representative of field conditions. DARWIN is capable of calculating an effective K value.

Using the 1993 AASHTO Guide, and DARWIN computer program, perform the analysis to calculate the required pavement thickness. Details such as dowel placement, dowel size, and joint details are found on TxDOT sheet CPCD-94. Jointing patterns must be carefully developed considering slab thickness and slab temperature shrinkage/expansion.

### 8.8.2 PCCP NTTA Methodology

The initial trial pavement thickness should be developed using the procedure outlined above in Section 8.8.1. This thickness, along with supporting layers, is to be

analyzed and adjusted to achieve the desired Service Life. The NTTA preference is for Portland cement Concrete Pavement systems to be designed for a 40-year Service Life rather than the TxDOT/AASHTO 30-year Service Life.

Once the initial design thickness of the pavement system is obtained, the fatigue life of the pavement system should be verified. The critical analysis is heavy wheel loads approaching mid-slab combined with environmental induced curling and warping stresses which leads to bottom-up cracking. These analyses are particularly important in cases where stiff subgrades and stabilized bases/subgrades are used to support the pavement.

Analysis of the trial pavement sections should be performed using FEM-based wheel load stress analysis to determine fatigue life and stress-strain levels in the materials. Generally, if stress levels are 40%–50% of capacity values, fatigue is generally not an issue.

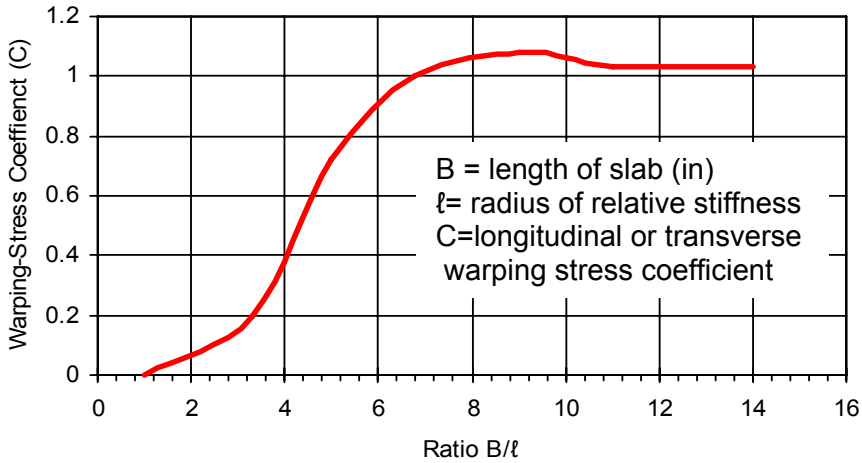
Design should also consider several other factors, such as curling and thermal stresses. Curling thermal stresses can be determined using ordinary calculations as shown below:

For edge stresses:

$$\sigma_t = \frac{cE\varepsilon(\Delta T)}{2}$$

Where: $\sigma_t$ = edge warping stress
c = coefficient from Fig 8.3
E = elastic modulus
ε = thermal coefficient (0.000005/F°)
ΔT = temperature differential between top and bottom of slab

Figure 8.3 Stress Coefficient




For interior stresses

$$\sigma_t = \frac{(E)(\varepsilon)(\Delta T)}{2}\left[\frac{C_1 + \mu C_2}{1-\mu^2}\right]$$

where: $\sigma_t$ = slab interior warping stress
$C_1$ = coefficient in direction of calculated stress
C2 = coefficient in direction perpendicular to $C_1$

For corner stresses:

$$\sigma_t = \frac{(E)(e)(\Delta T)}{3(1-\mu)}\left(\sqrt{\frac{a}{\ell}}\right)$$

where: $\sigma_t$ = interior warping stress
E = elastic modulus
e = thermal coefficient (0.000005/F°)
ΔT = temperature differential between top and bottom of slab
μ = Poisson's ratio (≈ 0.15)
a = radius of wheel load
ℓ = radius of relative stiffness

$$\ell = \sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}$$

h = slab thickness
k = modulus of subgrade reaction

After Bradbury, "Reinforced Concrete Pavements," Wire Reinforcement Institute, Washington, DC, 1938

## EXAMPLE OF CURLING AND WARPING STRESS CALCULATION:

E = elastic modulus – 5,000,000 psi
e = thermal coefficient - 0.000005/F°
ΔT = temperature differential between top and bottom of slab - 3°/in
μ = Poisson's ratio - 0.15
a = radius of wheel load – 5.9 inches
B = slab length – 14 ft
W = slab width – 12 ft
h = slab thickness – 9in
k = modulus of subgrade reaction – 200 pci

**1. Radius of Stiffness**

$$\sqrt[4]{\frac{(5{,}000{,}000)(9)^3}{12(1-0.15^2)(200)}} = 35.3 \text{ inches}$$

**2. B/ ℓ and C, $C_1$ & $C_2$**

B = 14x12 = 168 inches *longitudinal*
B = 12x12 = 144 inches *transverse*

B/ ℓ = 168/35.3 = 4.8 *longitudinal*
B/ ℓ = 144/35.3 = 4.1 *transverse*

From Fig 8.3
$C = C_1 = 0.68$
$C_2 = 0.45$

**3. Edge Stress**

$$\sigma_t = \frac{(.68)(5{,}000{,}000)(0.000005)(3x9)}{2} = 230 \text{ psi}$$

**4. Interior Stress**

$$\sigma_t = \frac{(5{,}000{,}000)(0.000005)(3x9)}{2}\left(\frac{0.68 + 0.15x0.45}{1-(0.15)^2}\right) = 258 \text{ psi}$$

**5. Corner Stresses**

$$\sigma_t = \frac{(5{,}000{,}000)(0.000005)(3x9)}{2}\left(\sqrt{\frac{5.9}{35.3}}\right) = 36 \text{ psi}$$

Load-related stresses are to be performed using FEM analysis with a load consisting of a Super Single radial tire at 150 psi with a 22,000 pound axle load (11,000 pound truck load). Load analyses should be performed for corner loading, edge loading, and interior loading. Combined curling and load/stress levels should be in the range of 40–50% of elastic limit stresses. For light duty pavements stresses up to 70% of the elastic limit for the high load (22,000 pound) condition is acceptable.

Designs should incorporate dowels in transverse joints. Dowels are to be at least 1.25 inches in diameter or 1/8 of the slab thickness, whichever is greater, and must be placed 12 inches on center. Longitudinal joints should be similar to details by TxDOT.

Joint details should be in accordance with TxDOT CPCD-94. Joint patterns should be developed for each pavement design. No Y-joints or T-joints are allowed.

Joint spacing is a function of the slab thickness, relative stiffness, aggregate type, curling stresses, lane width, and desired load transfer. The thermal differential and aggregate type have a most significant effect. Given the types of aggregate (granite and limestone) available, thermal expansion should be calculated from:

$$z = (\beta)(L)\,[\,(e)(\Delta T) + \xi\,]$$

Where:
- $z$ = Joint opening (change in slab length)
- $\beta$ = friction factor
  - 0.65 for stabilized bases
  - 0.80 for granular bases
- $L$ = slab length
- $\Delta T$ = temperature differential between top and bottom of slab
- $\xi$ = shrinkage coefficient = 0.0003 to 0.0010
- $e$ = concrete thermal coefficient
  - $6.6 \times 10^{-6}$/F° for quartz
  - $6.5 \times 10^{-6}$/F° for sandstone
  - $6.0 \times 10^{-6}$/F° for gravel
  - $5.3 \times 10^{-6}$/F° for granite
  - $4.8 \times 10^{-6}$/F° for basalt
  - $3.8 \times 10^{-6}$/F° for limestone

In addition, FHwA (1990) criteria for joint spacing should be calculated:

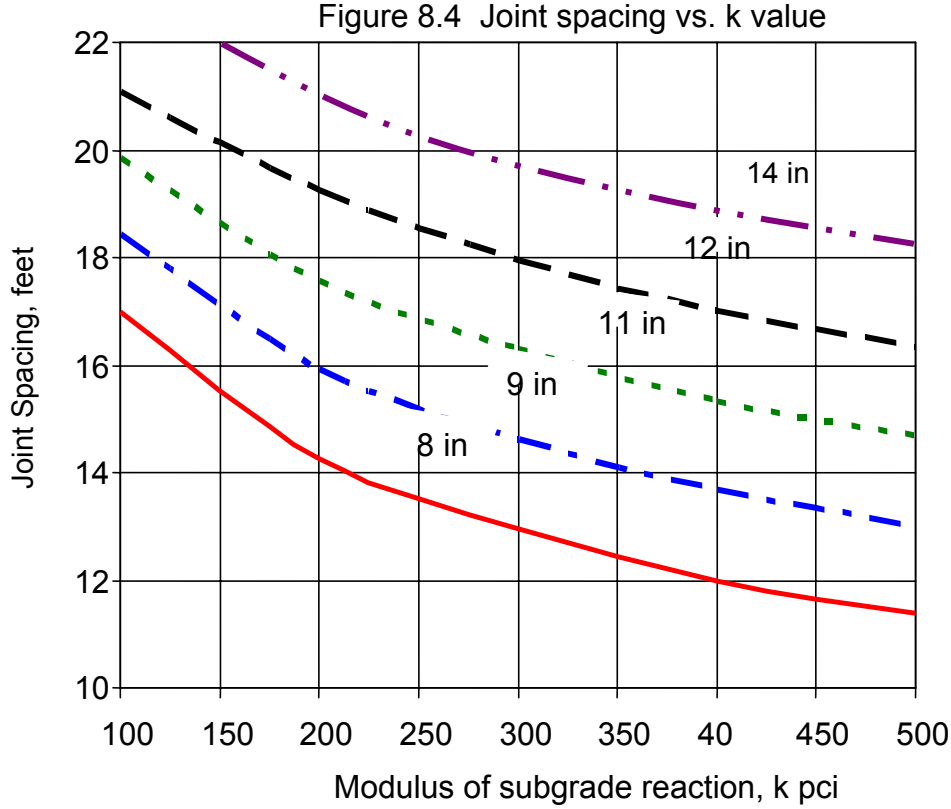
$$L/\ell = \frac{L}{\sqrt[4]{\frac{Eh^3}{12(1-\mu^2)k}}} < 5$$

Where:

L = length of slab (inches)
ℓ = radius of relative stiffness
E = elastic modulus
μ = Poisson's ratio (≈ 0.15)
h = slab thickness
k = modulus of subgrade reaction

The values are presented on Figure 8.4. It is good practice to conservative with this joint spacing.

Figure 8.4 Joint spacing vs. k value




The maximum joint spacing must be the lesser of the two values calculated. For most design thicknesses, joint spacing of 10 feet for thin slabs (8 inches) and 16 feet

for slabs up to 14 inches have been shown to be satisfactory; however, in the final analysis, subbase stiffness is the overriding factor affecting the design joint spacing.

Where the values of $\Delta L$ are between 0.035 inches and 0.045 inches, the joint sealant must be silicon. The size of the joint shall be calculated by:

$$W = \Delta L/\varepsilon_s = \text{width of joint reservoir}$$

Where:

$\Delta L$ = thermal length change between joints

$\varepsilon_s$ = allowable strain of sealant

**Note:** width to depth ratio = 1

Requirements for jointing, sealant, layout, etc. must be detailed in the report and provided on plans for the roadways.

Additional details for PCCP are contained in the NTTA Standards and Details. These should be reviewed and, if necessary, adjusted to fit the conditions and design assumptions.

## 8.9 Continuously Reinforced Concrete Pavement (CRCP)

Continuously reinforced concrete pavement (CRCP) is constructed with continuous longitudinal steel reinforcement, which requires no saw cutting of transverse contraction joints. Shortly after placement, CRCP begins to develop a transverse cracking pattern, ultimately spaced at 2–8 feet, that is typically fully developed within a 2-year period. CRCP is unique in that it is allowed to develop its own cracking pattern, which inherently minimizes curling and warping behavioral effects on performance while naturally promoting a smooth ride. The reinforcing steel used in CRCP plays an important role in the development of the crack pattern and in the limiting of the opening of cracks to certain levels to promote load transfer and stiffness of the cracks adjoining adjacent segments of concrete.

The behavior of the pavement system is governed by the crack spacing and the resulting Load Transfer Efficiency (LTE). The transverse crack pattern, when spaced closely, allows for very small cracks that have excellent load transfer. High LTE values lead to long pavement lives.

When the LTE is reduced or worn out, deterioration occurs and the cracks become more pronounced. Over time, the loss in LTE and increased crack width result in stress concentrations leading to longitudinally oriented fatigue crack. When coupled with erosion of the subbase, a punchout develops. The CRCP design process is based on this failure mechanism.

### 8.9.1 CRCP Empirical Methodology

The TxDOT methodology for CRCP is based on loaded slab behavior as it would pertain to jointed concrete systems. The resulting slab thickness using AASHTO or DARWIN is the same as plain PCCP. The resulting thickness is provided with specified longitudinal and transverse reinforcement in accordance with TxDOT Roadway Standards (CRCP(1)-03 and CRCP(2)-03. The CRCP (1)-03 sheet contains reinforcement plans for pavements up to 13 inches in thickness having a single layer of steel. The CRCP(2)-03 sheet applies to pavement systems more than 13 inches in thickness. The maximum pavement thickness is 15 inches.

The only other design data necessary is the thickness of the ACCP underlying the CRCP. The thickness varies from 4–6 inches of ACCP or 1 inch of ACCP over 6 inches of cement stabilized aggregate base. The option is normally a district decision.

While this completes the typical design process, additional work is required to determine the method of pavement terminus. There are several methods currently being used; however, performance has not been without problems. Additional discussion is presented in Section 8.9.3.

### 8.9.2 CRCP NTTA Methodology

The NTTA design philosophy is to use latest mechanistic design method. It is a trial-and-error method requiring iteration to solve for a specific level of performance in terms of punchouts per mile, which is a function of the pavement thickness. To simplify the design process has a significant number of variables that are fixed. The

result is an Excel spreadsheet having only a few variables—traffic, aggregate type, construction season, and slab thickness. The basis for the design and a discussion of the variables, theory, and methodology is presented in Section 8.9.2.1. Variables and specific spreadsheet instructions are presented in Section 8.9.2.2.

### 8.9.2.1 Basis of Methodology

The primary and predominant failure mechanism for CRCP is the punchout. The frequency of punchouts is regarded as the driving mechanism and the basis for design.

The heart of CRCP performance lies in the maintenance of uniform support and transverse crack widths tight enough to provide adequate stiffness across them. Deterioration of load transfer accelerates the fatigue of the concrete slab edge, leading to an edge punchout.

Consequently, the key design item is the behavior of the opening and closing crack movements over time. This item indirectly includes a broad range of design considerations that include:

- Reinforcement details;
- Traffic and loads;
- Concrete properties;
- Temperature and humidity conditions; construction season and interlayer friction;
- Foundation support, and
- Material variability.

#### 8.9.2.1.1 Slab Thickness

This is an important design feature from a performance standpoint. As the slab thickness increases the bending capacity and the shear stress resistance increase. In addition, the ability to transfer load across the transverse cracks also increases. Consequently, as slab thickness increases, performance improves.

Slab thickness must be selected within the context of other design features, including transverse crack width, percentage of steel reinforcement, concrete mixture

properties, and base type and stiffness—not to mention weather conditions at the time of construction. The goal is to select a minimum thickness that provides acceptable levels of aggregate interlock, wear-out resistance, punchout development, and smoothness (IRI) over the design period at the desired level of reliability.

#### 8.9.2.1.2 Transverse Crack Width

The width of the transverse crack is the primary design feature. It is the feature that is fundamental to many aspects of performance and is used to determine the amount of steel used for design. The narrower the crack the greater the capacity of the crack to carry shear stress between adjacent segments. Crack width is a key focus of the crack pattern since it has a dominant role in controlling the degree of load transfer. Ultimately, the crack width average (largely governed by the crack spacing) and distribution control the life and performance.

#### 8.9.2.1.3 Longitudinal Reinforcement

Longitudinal steel is important since it ultimately serves to control the opening of the transverse cracks. It is also critical from the standpoint of its effect on crack spacing. Field studies have shown that the greater the space between cracks, the wider the transverse cracks. The reinforcement in CRCP causes a restraining effect that increases as the percentage of steel increases and is one of the major steel-related design factors affecting crack development. Decreased crack spacing is associated with increased steel percentages. The q value is an important factor to consider for projects involving year-round paving due to the effect of the climate on the resulting crack pattern.

Experience in the United States has indicated that steel percentages of as low as 0.5 to as much as 0.70 percent have provided suitable cracking patterns depending upon climatic, design and construction conditions. In this regard, it is important to consider the effect of steel content within the context of several other design features, such as slab thickness, crack width, concrete materials properties, and base type and stiffness. Steel content is particularly sensitive with respect to different coarse aggregate types. The goal is to select a minimum steel content that provides an acceptable level of transverse crack widths as well as resistance to

punchout development over the design life at the desired level of reliability. For design purposes, the amount of steel for NTTA projects has been fixed at 0.6% or 0.0060.

#### 8.9.2.1.4 Transverse Crack Load Transfer Efficiency (LTE)

The load transfer of transverse cracks is a critical factor in controlling development of longitudinal cracking. Field studies have shown that close transverse crack patterns are associated with small crack widths that maintain a high resistance to wear-out of aggregate interlock. Maintaining load transfer of 92% or greater should minimize loss of load transfer over the design life of the pavement as well as limit the development of punchout distress, as long as the subbase support remains intact.

Crack load transfer is a function of slab thickness, percent steel, crack width, crack spacing, concrete materials properties, and base type and stiffness. The design goal is to select the crack width and slab thickness that corresponds to a load transfer of at least 92% which provides an acceptable level of punchout development and smoothness over the design life.

#### 8.9.2.1.5 Transverse Crack Spacing

One key calculated value in the design process is the average spacing between transverse cracks, which varies as a function of the reinforcement details. These details consist mainly of the diameter of the reinforcement and the amount of steel specified as a percentage of the slab cross-sectional area (which varies according to the selected design thickness). This value has been bracketed by using fixed input data to provide acceptable design values which relate to the Load Transfer Efficiency.

#### 8.9.2.1.6 Traffic and Other Factors

Traffic data input consists in part of the percentage of trucks and the average daily traffic. Truck traffic is characterized in terms of percent single, tandem, and tridem axels. Annual growth and Lane Distribution factors are included in the analysis.

The key design input relative to the properties of concrete is the strength the other properties have been fixed. The temperature and season of construction inputs

affect the prediction of the expected cracking interval. The subbase friction coefficient also affects that prediction; however, most asphaltic bond breakers have a coefficient of friction between 3 and 5 and a value of 4 has been selected and fixed.

The foundation support is provided in terms of k-value for the subgrade. It does not include the effect of the subbase on the pavement support. The effects of material variability are based on standard values of coefficient of variations for the materials-related parameter. The effects are imbedded in the code of the design spread sheet and not available for user input.

#### 8.9.2.1.7 Predetermined Design Parameters

Design considerations that influence performance and the development of punchout distress include foundation support, material variability, and traffic and loads discussed in the previous sections. Other integral design factors include reinforcement details, concrete properties, and temperature and humidity conditions. Construction season and interlayer friction play a role, but given the limited geographical areas covered by this manual, they can be fixed in the computer code. The variables and their values are:

1. Transverse crack spacing – 6 inches
2. Slab Thickness – 9 to 13 inches
3. Longitudinal Reinforcement – 0.6 percent
4. Transverse Crack LTE->92%
5. Concrete strength – 4,000 psi (compressive strength)
6. Foundation Modulus – 150 psi
7. Reliability – 95%
8. Design Life – 40 years
9. Base Type Modulus – 1.5” ACCP – TxDOT Type D - 500,000 psi

#### 8.9.2.1.8 Material Properties/Construction Season

The critical nature of the construction season and aggregate type requires additional consideration. The selection of these inputs determines the minimum bar size to be used. While these values are selected in the spreadsheet, the designer should make a note of the specified bar size for the construction season and aggregate type. The

NTTA has elected to standardize this value at a No. 6 bar (Grade 60) for all seasons and aggregate types.

### 8.9.2.1.9 Input Parameters Selection

The following table indicates the recommended design input for the spreadsheet:

Table 8D: PCCP Design Values

| Design Parameters | Design Values | Units |
|---|---|---|
| **Roadway Type** | **Punchout per lane-mile (R=95%)** | |
| Slab Thickness (Trial)[1] | 9–13 | Inches |
| Percent Steel | 0.006 | Fraction |
| Reliability | 95 | Percent |
| Design Life | 40 | Years |
| Total 18 kip ESAL [2] | Calculate or Spreadsheet | -- |
| Compressive strength | 4,000 | psi |
| Aggregate Type[3] | Gravel, Limestone or Granite | -- |
| Construction Season[4] | Spring, Summer, Fall or Winter | -- |
| Base Modulus of HMA | 500,000 | psi |
| Base Thickness | 1.5 | Inches |
| Mean Effective k-value | 150 | psi |

Notes:

1) Select trial slab thickness between 9–13 inches. If the level of punchout performance is not met by selecting a thickness of 13 inches, further analysis should be performed by modifying the selection of pavement base and subbase.
2) After selecting the trial pavement thickness, calculate the total 18 kip.
3) Select the aggregate type based on the aggregate for the Portland cement concrete mix.
4) Input the environmental parameter by selecting the season when the Portland cement concrete will be placed.

After all the input has been entered, perform analysis using the CRCP spreadsheet by selecting the Mechanistic Analysis option. After numerous iterations, the spreadsheet provides a chart showing punchout prediction based on 95% reliability.

The following table indicates the recommended level of performance based on the roadway types:

Table 8E. Punchout criteria

| | |
|---|---|
| Main Lane | <3 |
| Ramps | <6 |
| Frontage and Cross Roads | <6 |

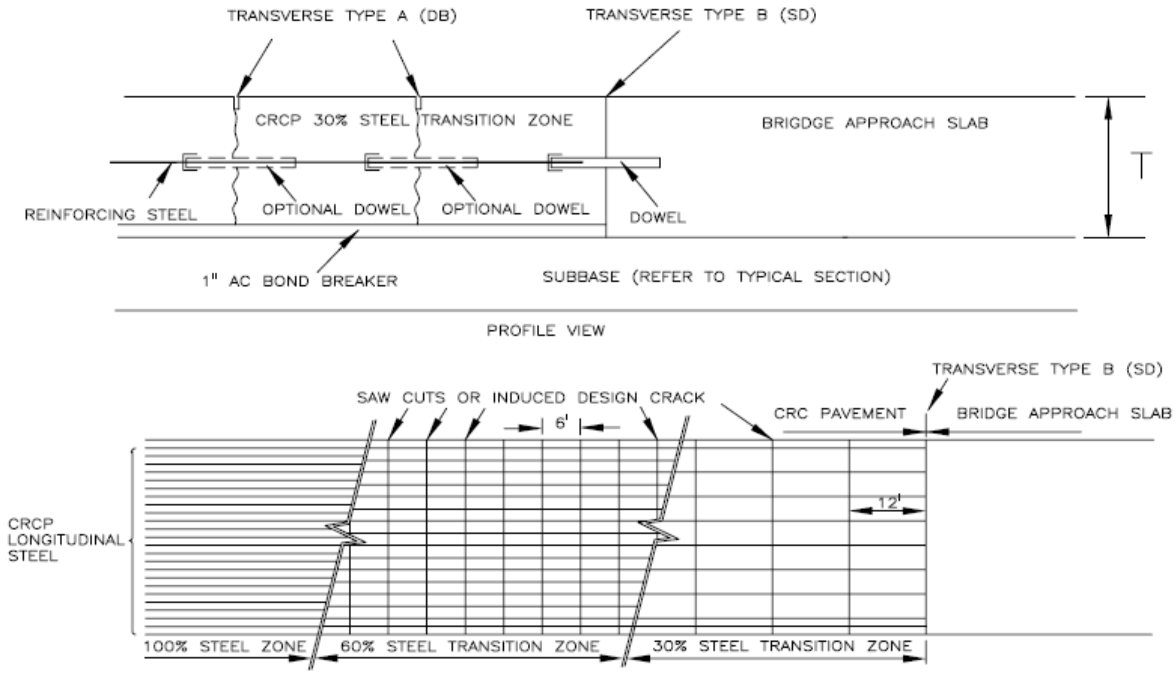
If the trial design does not meet the punchout per lane-mile design criteria, then pavement thickness should be increased and the analysis should be performed until the desired punchout per lane-mile is achieved.

### 8.9.3 Design Details

The NTTA has design standard details for CRCP. These details provide location and distribution of steel. Longitudinal joint details and terminus construction details are contained on the sheet.

The most critical detail to be decided by the designer is the terminus design. CRCP must terminate at the beginning of the project, at each bridge, and at the end of the project. Terminus details are important, since the restraint of the pavement at the terminus allows the crack pattern to fully develop as designed. Unrestrained pavement will expand and contract with thermal plastic shrinkage, curling and loading stresses. This movement will significantly wear and reduce the load transfer in the aggregate and result in premature failure. Typically, this occurs in the 120–150 feet from the end of the pavement. Usually after this length, there is sufficient restraint to allow normal crack development. See Fig. 8.5 CRCP Transition Terminus, which follows.

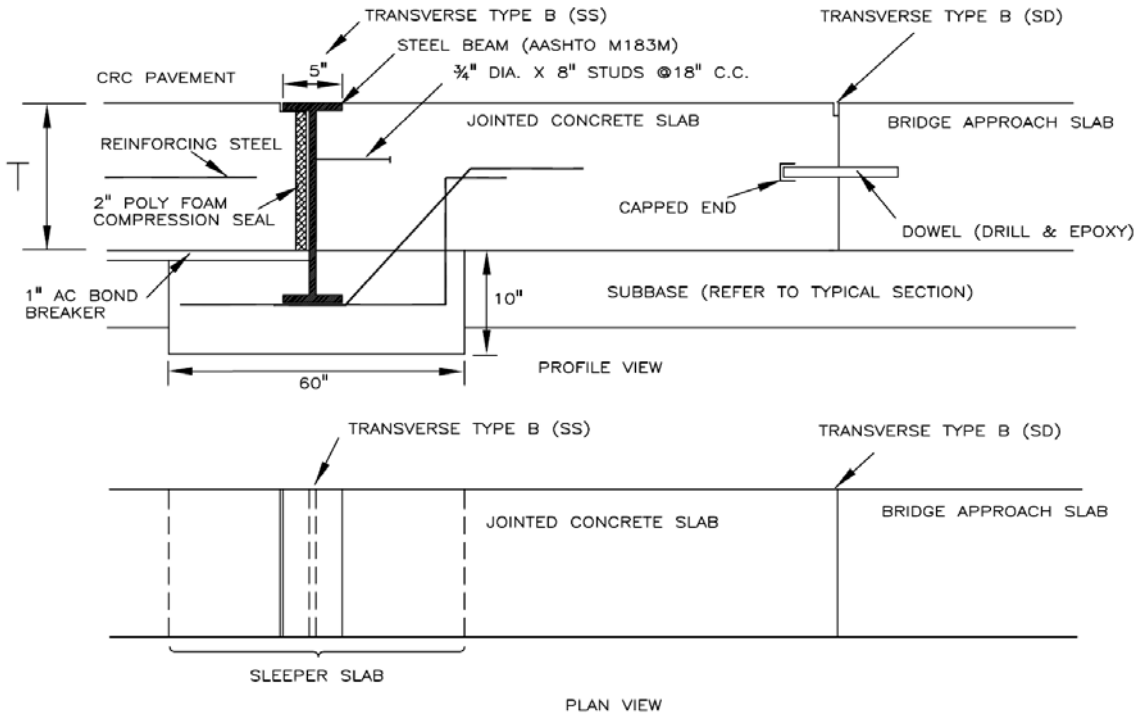
Figure 8.5: Transition Terminus




Terminus details are designed to either restrain or transition the pavement to reduce the uncontrolled cracking problem. Restraint details are typified by the TxDOT CRCP using lugs to restrain the pavement. This particular detail depends on the passive earth pressure on the lugs being fully developed; and this may require several inches of movement to fully mobilize the strength of the soil. The designer is cautioned that movement of this nature may be far too much to control the pavement system.

There are several other terminus option details that can be used. Both Austin and Houston have details that are believed to control cracking. A simplified example is presented in Fig. 8.6.

Figure 8.6 TxDOT Austin/Houston Simplified Terminus




Designers should be aware of these considerations and options. The selection of the most appropriate details is critical to the performance of the terminus pavement.

## 9.0 Reporting Requirements

The final subgrade investigation and pavement design report should present a complete picture of the investigation, analyses, results, and recommendations. The data presented should include the graphical boring logs, laboratory test results, analyses (including calculations), and any supporting references or publications. The data must be organized and presented in such a manner that the logic and analysis leading to the conclusions and recommendations can be easily followed by knowledgeable engineers.

Typically, the report is organized into at least three parts: introduction, construction recommendations, and pavement recommendations. The sections generally follow the course of the investigation and project construction. Introductory material should consist of contract scope, existing site and geologic conditions, methods of investigation and subsurface conditions, and properties of the subgrade soils.

The second part of the report should be organized in the order of construction beginning with clearing and grubbing requirements followed by subgrade preparation and fill placement and compaction requirements. Slope recommendations for both cut and fill slopes should be presented in this section. The embankments which support paving should be given special attention and, if necessary, figures detailing the zones of select or treated fill should be provided.

The final section of the report will provide pavement recommendations and requirements. The pavement section should be detailed enough to describe the design limitations and any alternatives to be considered. Discussions should include number of lanes in each direction, potential expansions, pavement design limitations, transition details, special support characteristics (i.e., retaining walls, bridges, tunnels, etc.), and other features that impact or could impact the design.

### 9.1 Report Introduction

The introductory parts of the report should include the Executive Summary, which provides a casual review of the scope of work and results. This should be limited to one page in

length. The next sections include the Scope of Work and (if desired) Summary of Conclusions. The Scope of Work merely summarizes the tasks involved in the contract and explains the limitations and assumptions involved in the work. This section will also include any changes, additions. or omissions to the work as contracted. Reference should be made to the contract parties, proposal or contract number, and date.

The first part of the report should describe the project location and scope. Reference should be made to the length, width, and location by either latitude/longitude, Township, Range and Section, or state plane coordinates. The location should be graphically presented on Figure 1 as the Vicinity Map, which should have sufficient detail to show the location and allow anyone to find the project. The project scope should include references to the plan and profile drawings provided and describe the project in general terms, i.e., 12 miles long, 6 bridges, 4 lanes each direction, 4 lane frontage roads, 5 intersections, 2 Direct Connectors, etc. Very specific descriptions should be provided for grading and pavement features. The typical and extremes of cut and fill should be presented and description should include discussions of balanced, import, or export. Abutment heights, anticipated slopes, drainages, etc. should be also discussed. All detail should be referenced to the plan and profile sheets by station number. A section should be included which references the preliminary investigation or document review phase of the work and include a brief summary of the results

Figures in support of this section of the report should consist of:

- Vicinity Map
- Plan and profile of the Alignment (note: these figures also normally contain boring logs and boring locations.

### 9.2 Site and Geologic Conditions

The investigation portion of the report should include published data and research results and sources obtained in the preliminary investigation. A detailed description of site conditions, features, vegetation, structures and any other noteworthy observations should be presented in this section. Any anomalous conditions should be given special consideration in this section of the report. The surficial and bedrock geology are discussed

and their impact on construction presented. These data should be organized into the introductory sections in the beginning of the report and will usually contain:

Site Conditions and Features

- Topographic conditions (high/low elevations) – general discussion of topography
- Features of interest – hills, valleys, rivers, lakes, bluffs, etc.
- Structures/Land Uses – historical, current
- Vegetation – crops, trees (type-density aerial extent, pheatophyte vegetation, plantings, etc.
- Water features – lakes, rivers, streams, springs, seeps, drainage; Figures supporting this part of the report should consist of:

Geology

Surficial Geology

- Soil types (USDA)
- General properties of interest

Bedrock Geology

- Formations/Locations/Depths
- Properties
- Impacts of Geology

Figures in support of this section of the report should consist of:

- Tree copse locations shown on the Plan and Profile
- Surficial and Bedrock Geology

### 9.3 Field Investigation

Presentations of the field investigation should include rationale of boring location and depth, for example, "*borings were located at 500 foot intervals along the alignment in alternating lanes and in the unusual high spots located at station 29+45.*" It should also include information about additional borings located in depressions, vegetation changes, springs or seeps, changes in soil/bedrock color, etc. Drilling and sampling methods and the techniques used in locating and elevating borings should be presented as well. This section of the report might be organized as follows:

Field Investigation

Boring locations

- Selection criteria
- Descriptions

Methods of locating and elevating the borings

- Survey
- GPS
- Taping from centerline
- Accuracy ± location ± elevation

Drilling Methods

- Drill rig type/make/model
- Drilling method
    - Auger-Rotary-Hollow Stem Auger ,etc.
    - Bit type (if appropriate)

Sampling Method

- Undisturbed Sampling
    - Shelby Tube
    - Pitcher Sampler
    - Foil Sampler, etc.
- Relativity Undisturbed
    - California Sampler
    - Standard Penetration
- Disturbed Sampling
    - Grab samples
    - Bulk samples

Figures in support of this section of the report should consist of:

- Graphical boring logs shown on the Plan and Profile
- Graphical "fence" logs plotted to elevation and presenting the finish subgrade elevation
- Individual logs in Appendix (if desired)

## 9.4 Laboratory Investigation

The general laboratory testing philosophy and rationale should be presented in a summary format to illustrate the reasoning behind the test and test locations selected. This introductory discussion should contain logic supporting the frequency of swell/consolidation profiles, materials selected for stabilization, design of moisture treated zones, etc. data presented should include properties of interest to designers and those detailed in this manual. Discussions should be organized by testing methodology, i.e., classification testing, in-situ engineering properties of the subgrade, modified engineering properties (compacted and stabilized) and support properties of natural, compacted and stabilized subgrade.

All testing should be conducted in accordance with AASHTO or ASTM standards, except the occasional TxDOT specific tests, such as TEX-113-E, TEX-114-E and TEX-145-E. The specific standards used in the testing should be listed in the text or appendix. The testing should be presented in the figures for testing of some importance and in a table summarizing results; testing reporting figures can be placed in an Appendix.

Testing Philosophy and Methodology

- Undisturbed
    - Moisture and Density
    - Engineering Properties
        - Strength
        - Swell-Consolidation
        - Permeability
        - Support Characteristics, etc.
- Disturbed
    - Classification Testing
    - Proctor Compaction
- Modified
    - Lime/Cement design
        - Strength
        - Swell
        - Resilient Modulus

## 9.5 Subsurface Conditions

Depending upon the geologic setting(s) and the length of the alignment, this section can be quite complex and involve a significant effort. It is recommended that for complex sites, discussion of the areas are divided by geology and discussed individually.

The description of each geologic section should use a top-down approach, beginning with the topsoil. Each layer should be characterized using its Unified Soil classification System and the AASHTO classification, as appropriate. Natural and engineering properties should also be presented. The properties of subgrades modified by either manipulation, moisture treatment, or stabilization should also be presented. Presentation of data can be organized either by soil type and include all data or by general soil descriptions followed by presentations of engineering properties for natural and compacted soils.

Summaries of testing data should be presented in figures that follow the graphical fence-type boring logs or in the Appendix and on the attached Summary Table. If the "fence" logs are on a large enough scale; some laboratory can be presented on these figures. Testing data should be presented by test type. For small projects, testing presented as figures in the report are preferable. Longer projects may require several appendices for "detailed" boring logs and test data organized by type.

For each geologically similar zone, the report should be organized as follows:

Typical/Generic Subsurface Profile

0-2' Topsoil

2-7' Clay, sandy, stiff, moist, dark brown

7-25' Bedrock Limestone, cemented, very hard moist, gray

- Gradation or % passing No. 200
- Atterberg limits (LL & PI)
- Unified and AASHTO Classifications

Engineering Properties

- Strength
    - Unconfined compressive strength

    - Undrained strength
    - Drained strength
- Swell/Consolidation
- Permeability
- Moisture/Density
- Resilient Modulus

Compacted Properties (figures included)

- Maximum Density/Optimum Moisture Content
- Strength
    - Unconfined
    - Undrained
    - Drained
- Swell/Consolidation (may require moisture content vs. swell for expansive soils)
- Resilient Modulus

Stabilized Properties (figure included)

- Design Data (stabilizer, percentage, test results)
- Maximum Density/Optimum Moisture Content
- Resilient Modulus

## 9.6 Subgrade Support Analyses

This portion of the report should provide data and recommendations for necessary subgrade treatments, including moisture treatment for expansive subgrade, removal and re-compaction for soft (settlement prone) subgrade, and/or replacement for high sulfate or high organic soils. The analyses should present specific areas (by station) for this treatment.

Sufficient data and analyses should be presented so that the engineering logic, as well as any necessary calculations, can be followed. Should the treatment require specialized treatment, information about this must be included (usually in Appendices). Information about subgrade stabilization—lime or cement—should be presented and should include the requirements by station.

## 9.7 Site Grading

Clearing and grubbing, embankment preparation, fill compaction requirements, and settlement considerations are covered in this section. In general, the sequence follows the construction planning.

### 9.7.1 Clearing and grubbing

The section about clearing and grubbing should cover the removal of organics and vegetation from areas to be either cut or filled. The recommendations should include the minimum stripping thickness (organic or topsoil zone) and information should be provided about areas with greater thicknesses of organics, such as farmed land, which should be listed by stationing. Organics should be removed to where no vegetation or root is greater than 1 inch except where the stripped subgrade is to be stabilized. Since the stabilizing agents (lime and cement) react with organics, all organic materials should be removed.

This section of the report should also include specific recommendations for clearing trees if they are present. Copse of trees should be shown on the plan and profile sheets and should be specifically referenced in this section. In most cases, TxDOT standards are satisfactory and should be referenced. Where the finished subgrade is at or near the natural elevation, extra care is required to remove the entire root zone and fill the excavation with soils similar to the native soils but with no organics.

### 9.7. 2 Compaction

Subgrade compaction requirements are presented in this section and should include subgrade preparation requirements and fill placement specifications. The section should also include requirements for subgrade preparation, including scarification and compaction of areas to receive fill as a minimum. In some cases, it will be necessary to present information about additional subgrade preparation as presented in Section 9.6 and to describe moisture treatment in cut areas, transition areas, and locations near the surface of fill sections, as well as the removal and replacement or re-compaction of soft subgrade.

The section should also cover fill placement and include requirements for high fills and moisture treatment of the upper 10 feet of expansive soil fills. Where fills exceed 20 feet in height, a zoned fill compaction specification is recommended. Use of modified compaction in lower parts of the fill will increase the strength of fill sections and moisture treatment (if necessary) will reduce swell in the upper 10 feet. Where large fills (>20 feet) are required, the design should also provide recommendations for overbuild (to allow for settlement) and crown (differential settlement of sloped fill).

### 9.7.2 Stabilization

This section should cover subgrade stabilization and show the type of stabilization agent (lime or cement), concentration, and aerial extent (by stationing). Should specialized or double treatment be required, the methodology should be presented here and the (locations shown on the plan and profile sheets).

### 9.7.3 Drainage

Drainage is one of the most critical components of the design, yet it is the component most often overlooked or ignored. This section should provide foundation recommendations for all drainage structures, culverts, and boxes. The foundation recommendations should consist of allowable bearing capacity, foundation requirements (removal and replacement), surface and subsurface drainage, lateral pressures, and wingwall and headwall design data.

Also include information about subsurface drainage, including location, plan and profile of drain systems, drain schematics and cross-section, material details, and installation requirements. Drain locations must be clearly marked on the plan and profile sheets. Discussion should anticipate drainage for perched or water table conditions encountered. Subdrain for all locations where fill is located with within drainages. Drain details are to be provided in the figures along with gradation, minimum slope recommendations, cleanouts (if a permanent drain), fabric, headwall details, and other appropriate design and construction requirements.

This section of the report might be organized as follows:

Clearing and Grubbing

- Topsoil striping depth and extent
- Tree removal
- Location and size
- Filling method
- Special conditions at finished subgrade

Subgrade Compaction

- Embankment preparation
- Fill compaction
- High fills
- Special conditions
- Removal and replacement
- High sulfates, soft soils, etc.
- Moisture treatment procedure

Stabilization

- General consolidations
- Special conditions

Drainage

- Surface
- Subsurface structures
- Culverts
- Outlets

## 9.8 Slope Stability

The discussion of slope stability should include short-term stability for cuts in which life is measured in days, long term stability in which life is measured in decades, and skin slides. The analyses should be based on measured strengths obtained from laboratory investigations using acceptable “accurate” methods of analysis.

Both cut and fill slopes must be evaluated. Typically, a parametric analysis is useful for fill slopes, which have more predictable properties based on compacted soil strength values and dry conditions. For slopes of consistent soil types, these analyses are usually

presented in terms of slope angle vs. safety factor. Where several soil types make up a given fill slope, then the analysis should be site specific.

Cut slopes require that the analysis be based upon the site-specific conditions using in situ soil-strength values measured on undisturbed samples. Ground water conditions, as existing or as anticipated, are critical to the analyses and require careful modeling and explanation. The minimum static Factor of Safety is 1.5.

Generally, the DFW area does not have a significant risk of seismic activity, and any required analyses can be performed using quasi-static methods. Local requirements and TxDOT specific values should be followed. The minimum dynamic Factor of Safety is 1.3.

The skin slide is a special problem unique to high plasticity clays. The problem has been studies by Saleh Wright (1997) and analytical methods based upon plasticity indices developed. The analysis is straight forward and should be presented in the text or as a figure in the appendix containing the slope stability calculations. The mitigation method suggested should be clearly presented along with minimum slope ratios and drainage recommendations for all slopes by station number.

This section of the report might be organized as follows:

Slope Stability

- Short term analysis
    - Temporary excavations
    - Slope ratios
    - Bracing requirements/lateral pressures
- Long term analyses
    - Fill slopes
    - Analysis results (static/dynamic)
    - Recommendations – slope ratio – drainage
    - Cut slopes
    - Analysis methods and results

- Site specific slope ratios and drainage recommendations (tabular form) include specific drainage recommendation (type and location)

- Skin slides
  - Analyses and results
  - Recommendations

## 9.9 Traffic

Using the base-line traffic projections, the analyses in Section 5 should be explained and presented to determine the design traffic matrix. The traffic analyses should be based upon the adjusted demographic projections and conservative truck projections from the traffic engineers. Analyses should be presented in an Appendix that can be easily followed by a knowledgeable engineer. If DARWIN calculations are performed, the data should be summarized in the text and presented in an Appendix. The text should explain the process and assumptions used in the analysis.

## 9.10 Pavement Design

The pavement design should include at least two pavement design alternatives for all pavements except mainlanes. Mainlanes and Ramps are required to be Continuously Reinforced Concrete Pavement (CRCP). These designs should consist of required subgrade treatment (removal/replacement or moisture treatment), Stabilized Subgrade, Cement Treated Base, ACCP, and CRCP. For mainlanes and ramps, the Risk Matrix should be completed for the CRCP. The recommended thicknesses should be based on the design with the minimums as listed in Section 8 for all pavement types.

Pavements recommended should include at least 2 alternatives for the frontage and cross roads based upon the design traffic, unless agency specifies a particular type of pavement. Graphical and Tabular pavement thickness recommendations should be presented in the text or on figures. The calculations or summary sheets should be presented in an Appendix. Calculations must be presented in a form that can be easily followed and understood by a

knowledgeable engineer. Where appropriate, assumptions are to be completely delineated and explained.

The final step in the pavement design process is to analyze the Life Cycle Cost Analysis (LCCA) of the pavement design alternatives. This analysis is necessary only when competing designs have different surface courses ACCP and PCCP. Where the surface courses are the same, the LCCA will provide the same result. For the analysis to be valid, the treatment methodology and the interval must be in accordance with the recommendations of either TxDOT or FHwA. Both these agencies have LCCA software which is acceptable to the NTTA. In order for the analysis to be considered definitive, there must be at least a 3% difference in the 30-year cost.

The basis for the LCCA will be at least a 40-year life cycle and an interest rate of 4%. Results should be present in Tabular form. For the Traffic Matrix used for the mainlanes, only the initial cost need be considered. The cost differential between Low, Moderate and High risk should be reported along with the thicknesses of each alternative.

A typical pavement design section will contain the following:

Introduction

- Traffic classification
- Design Assumption
- Design Analyses/Methodology

Calculated Pavement Sections/Recommendations

- Frontage roads
- Cross roads
- Other pavements designed
- Mainlanes and ramps

Pavement Alternatives

Life Cycle Consideration

- Treatment Intervals/Types
- ACCP
- PCCP
- CRCP

Life Cycle Analyses

## 9.11 Materials and Construction Requirements

Many of the design material properties are contained in the NTTA standard specifications. There are conditions that occasionally arise which require additional explanation and specifications. Some special conditions occur when materials cannot meet or are clearly superior to the standard specifications and adjustments are required. In addition, some new materials and treatments are developed and require revision of standard specifications. It should be noted that the use of any non-standard materials must be tested and approved prior to use in design. Chemical additives used to "stabilize" expansive soils must have a proven track record and a basis in chemistry acceptable to the NTTA. No non-standard stabilizers will be approved without at least a 10-year provable performance record.

In the event that specifications require revisions or a new specification, the details including item number, section, and versions must be presented and justified. Justification must be based on engineering reasons and supported by laboratory testing data. Any changes must be fully justified in the report.

## 9.12 Design Details

Design details must be provided for each type of pavement. At a minimum, the details must conform to NTTA design standards. Joint details and joint patterns, along with dowel configuration, must be presented for PCCP pavements. CRCP must have steel placement details and terminus details. If the standard details are not accepted, additional details must be provided in an appendix.

## 9.13 Report Checklist

In order to provide a quick, easy, and complete report review, a report checklist has been provided below. The checklist is provided in the preferred order of the "Subgrade Investigation and Pavement Design" report.

## General Report Checklist:

- ☐ Executive Summary
- ☐ Scope of Work
- ☐ Summary of Conclusions (optional)
- ☐ Project Location
- ☐ Project Description
- ☐ Site Conditions
  - ☐ Land Use and Features
  - ☐ Vegetation
  - ☐ Drainages, Ponds, Dams and other drains Features
  - ☐ Structures
  - ☐ Environmental considerations
- ☐ Geologic Conditions
  - ☐ Surficial Geology
  - ☐ Bedrock Geology
  - ☐ Geologic Impacts
- ☐ Field Investigation
  - ☐ Boring Location Selection
  - ☐ Boring Methodologies
  - ☐ Groundwater Measurements
- ☐ Laboratory
  - ☐ Testing Philosophy
  - ☐ Techniques including AASHTO, ASTM, or TxDOT Designations
- ☐ Subsurface Conditions
  - ☐ Soil Profiles
  - ☐ Engineering Properties

- ☐ Classification
- ☐ Shrink-swell Potential
- ☐ Subgrade Support Characteristics
- ☐ Subgrade Support bases on Stabilization Series
- ☐ Resilient Modulus

☐ Subgrade Support Analyses

- ☐ Moisture Treatment Analyses
- ☐ Recommendations for Stabilization
  - ☐ Meets Criteria
  - ☐ List Station, Concentration, Stabilizer
  - ☐ Special Construction (Double Treatment)

☐ Site Grading

- ☐ Clearing and Grubbing
- ☐ Subgrade Compaction
  - ☐ Cut
  - ☐ Fill
  - ☐ Removal/Replacement
- ☐ Bulking/Shrinkage

☐ Drainage

- ☐ Surface Drainage
  - ☐ Drainage Features
  - ☐ Pavement crown/cross slopes
  - ☐ Runoff protection
  - ☐ Slopes
- ☐ Subsurface Drainage
  - ☐ Pavement/edge Drains

- ☐ Interceptor Drain
- ☐ Sausage Drain
- ☐ Selective Grading
  - ☐ Soft and wet soil areas
  - ☐ Expansive soil areas
  - ☐ Backfill criteria
- ☐ Slope Stability Analyses
  - ☐ Cuts
    - ☐ Analyses Results & Recommendations
  - ☐ Fills
    - ☐ Analyses Results and Recommendations
- ☐ Traffic Analyses
  - ☐ Traffic data
    - ☐ Source
    - ☐ Current and future ADT
    - ☐ Vehicle type and distribution
  - ☐ Traffic Calculations
    - ☐ Growth factor and LEF
    - ☐ ESAL calculations
  - ☐ Traffic matrix
- ☐ Pavement section
  - ☐ Pavement system foundation
  - ☐ Intermediate and surface layer
  - ☐ Surface layer
  - ☐ Design methodology
    - ☐ Empirical – PCCP

- ☐ Mechanistic – high performance CRCP
- ☐ Empirical – ACCP and JPPCP
- ☐ Material Considerations
- ☐ Pavement Details
  - ☐ ACCP, JPPCP, CRCP
  - ☐ Terminus
- ☐ Life Cycle Analysis

**Figures and Appendix Checklist:**

- ☐ Vicinity Map
- ☐ Plan and Profiles
  - ☐ Boring logs
  - ☐ Tree copse
  - ☐ Subsurface drain locations
  - ☐ Special features
- ☐ Geology (surficial if necessary)
- ☐ Bedrock Geology
- ☐ Graphical Boring logs—plotted to elevation with finished subgrade
- ☐ Slope Stability Recommendations
- ☐ Laboratory Test Data (can be placed in Appendix B)
  - ☐ Swell-Consolidation Testing
  - ☐ Proctor Compaction
  - ☐ Stabilization Test Results
  - ☐ Triaxial Test Results
  - ☐ Resilient Modulus Test Results
  - ☐ Other
- ☐ Fill Details

- ☐ Drain Details
- ☐ Tables
  - ☐ Summary of groundwater conditions
  - ☐ Summary of pavement section limits
  - ☐ Summary of subgrade treatment and stabilization limits
  - ☐ Maintenance program
  - ☐ Summary of life cycle analysis
  - ☐ Laboratory Test Results
- ☐ Appendix A: Detailed Boring Logs (if necessary)
- ☐ Appendix B: Laboratory Test Results
- ☐ Appendix C: Slope Stability Calculations
- ☐ Appendix D: Pavement Design Calculations
- ☐ Appendix E: Design Details
- ☐ Appendix F: Material Considerations

Appendix A

# GLOSSARY OF TERMS

**Adhesive Failure**: Loss of bond between the joint sealant and the joint, or between the aggregate and the binder.

**Aggregate Base (base course)**: Crushed stone or gravel, immediately under the surfacing material.

**Aggregate Interlock**: Interaction of aggregate particles across cracks and joints to transfer load.

**Air Trucking Association (ATA):** The trade group for the trucking industry.

**Alligator Cracks**: Interconnected cracks forming a series of small blocks resembling an alligator's skin or chicken wire.

**American Association of State Highway and Transportation Officials (AASHTO):** Formerly known as AASHO – changed name in about 1973 **-** Association to foster the development, operation and maintenance of an integrated national transportation system governing highway and transportation departments in the 50 states, the District of Columbia and Puerto Rico in all five transportation modes.

**American Association of State Highway Officials (AASHO):** Early organization of state officials known for conducting a study to determine pavement performance of known thickness under applied loads of known magnitudes and frequencies; resulted in the performance equations and nomographs used in AASHTO guide.

**American Society for Testing and Materials (ASTM):** Standards development organization providing source of technical standards for materials, products, systems, and services to fulfill industrial, governmental and environmental standardization requirements.

**Analysis Period**: The period of time for which the economic analysis is to be made; ordinarily will include at least one rehabilitation activity.

**Asphalt Cement Concrete Pavement (ACCP)**: High-quality, thoroughly-controlled hot mixture of asphalt cement and well-graded, high quality aggregate, thoroughly compacted into a uniform dense mass.

**Asphalt Cement Concrete Pavement Structure**: A pavement structure, placed above the natural subgrade, with courses consisting of asphalt-aggregate mixtures, untreated aggregate courses, improved or stabilized subgrade, drainage layers, etc. which act in a structural capacity.

**Asphalt Cement Concrete Pavements**: Pavements consisting of a surface course of mineral aggregate coated and cemented together with asphalt cement on supporting courses such as asphalt bases; crushed stone, slag or gravel; or on Portland Cement Concrete Pavement, brick, or block pavement.

**Asphalt Emulsion Slurry Seal**: A mixture of emulsified asphalt, fine aggregate and mineral filler, with water added to produce slurry consistency.

**Asphalt Leveling Course**: A course (asphalt-aggregate mixture) of variable thickness used to eliminate irregularities in the contour of an existing surface prior to superimposed treatment or construction.

**Asphalt Overlay**: One or more courses of asphalt construction on an existing pavement. The overlay generally includes a leveling course, to correct the contour of the old pavement, followed by uniform course or courses to provide needed thickness.

**Asphalt Tack Coat**: A light application of emulsified asphalt applied to an existing asphalt or Portland cement concrete pavement surface. It is used to ensure a bond between the surface being paved and the overlying course. Typically 0.10 gals/yd$^2$ of CSS1h.

**Atterberg Limits:** The upper and lower plastic limits of fine-grained soil. It defines the water content boundaries between non-plastic, plastic and viscous fluid states.

**Axle Load:** The maximum allowable weight that is applied by one axle, on a carriage or engine, to the pavement.

**Bias Ply Tire:** A type of tire construction utilizing plies that run diagonally from one bead to the other. One ply is set on a bias in one direction, and succeeding plies are set alternately in opposing directions crossing each other. Also called a cross-ply tire.

**Benkelmann Beam:** A long, pivoted bar used to measure the rebound of a pavement system under an 18 kip axle load. The only direct measure of structural capacity of a pavement.

**Binder**: Asphalt Cement used to hold stones together for paving.

**Binder Course**: The layer of asphalt cement concrete pavement underlying the surface course.

**Bituminous**: Like or from asphalt.

**Bleeding or Flushing**: The upward movement of asphalt in an asphalt pavement resulting in the formation of a film on the pavement surface. It creates a shiny, glass-like, reflective surface that may be tacky to the touch in warm weather.

**Block Cracking**: The occurrence of cracks that divide the asphalt surface into approximately rectangular pieces, typically one square foot or more in size.

**California Bearing Ratio Test (CBR)**: An empirical measure of bearing capacity used for evaluating bases, subbases, and subgrades for pavement thickness design.

**Centerline**: The painted line separating opposing traffic lanes.

**Channels**: See *Rutting*.

**Chip Seal**: A thin layer of emulsified asphalt cement in which aggregate is embedded. The seal is placed to improve the texture of the pavement surface to increase skid resistance and decrease permeability of the surface.

**Chipping**: Breaking or cutting off small pieces from the surface.

**City Street**: A street whose traffic is predominantly local in character.

**Classification Test:** Test to determine specific group of soil based on its gradation and Atterberg limits. Common methodologies include AASHTO and United Soil Classification System.

**Cohesive Failure**: The loss of a material's ability to bond to itself or its substrate; results in the material splitting or tearing apart from itself or its substrate (i.e. joint sealant splitting).

**Composite Pavement**: A pavement structure composed of an Asphalt Cement Concrete Pavement wearing surface and Portland Cement Concrete Pavement slab; an Asphalt Cement Concrete Pavement overlay on a PCC slab is also referred to as a composite pavement.

**Continuously Reinforced Concrete Pavement (CRCP):** Portland Cement Concrete Pavement constructed with sufficient longitudinal steel reinforcement to control transverse crack spacing and openings in lieu of construction joints for accommodating volume changes and load transfers.

**Corrugations (Washboarding)**: A form of plastic movement typified by ripples across the pavement surface. Most common in aggregate surficial pavements but occurs in Asphalt Cement Concrete Pavements as well.

**Crack**: Approximately vertical random cleavage of the pavement due to thermal or load action.

**Cut:** Excavation of the current topography to achieve proposed final grades.

**Deflection**: The amount of downward vertical movement of a surface due to the application of a load to the surface.

--Rebound Deflection: The amount of vertical rebound of a surface that occurs when a load is removed from the surface.

--Representative Rebound Deflection: The mean value of measured rebound deflections in a test section plus two standard deviations, adjusted for temperature and most critical period of the year for pavement performance.

--Residual Deflection: The difference between original and final elevations of a surface resulting from the application to, and removal of one or more loads from, the surface.

**Design ESAL**: The total number of equivalent 80kN (18,000 lb) single-axle load applications expected during the Design Period.

**Design Lane**: The lane on which the greatest number of equivalent 80kN (18,000 lb) single-axle loads is expected. Normally this will be either a lane of a two-lane roadway or the outside lane of a multi-lane highway.

**Design Period**: The number of years from initial construction or rehabilitation until terminal service life. This term should not be confused with pavement life or Analysis Period. By

adding asphalt overlays as required, pavement life may be extended indefinitely, or until geometric considerations or other factors make the pavement obsolete.

**Diamond Grinding**: Process of grinding the surface of a pavement to restore smoothness

**Disintegration**: The breaking up of a pavement into small, loose fragments due to traffic or weathering.

**Distortion**: Any change of a pavement surface from its original shape.

**Drainage Coefficients**: Factors used to modify layer coefficients in flexible pavements or stresses in rigid pavements as a function of how well the pavement structure can handle the adverse effect of water infiltration.

**Dynaflect:** A commercial product used to determine the structural capacity of a pavement system. Similar to the Falling Weight Deflectometer except the loading is lighter and dynamic (vibration) rather than impact.

**Edge Cracking**: Fracture and materials loss in pavements without paved shoulders which occurs along the pavement perimeter; caused by soil movement beneath the pavement.

**Effective Thickness**: The thickness that a pavement would be if it could be converted to Full-Depth Asphalt Cement Concrete Pavement.

**Embankment (embankment soil)**: The prepared or natural soil underlying the pavement structure.

**Embrittlement**: Premature (surficial) cracking of an asphalt concrete pavement due to oxidative aging of the asphalt cement.

**Empirical Design:** Method to determine pavement thickness based on traffic load using mean values based on experience in certain conditions.

**End Result Specifications**: Specifications that require the contractor to take the entire responsibility for supplying a product or an item of construction. The highway agency's responsibility is to either accept or reject the final product or apply a price adjustment that compensates for the degree of compliance with the specifications. (End result specifications have the advantage of affording the contractor flexibility in exercising options for using new materials, techniques, and procedures to improve the quality and/or economy of the end product.)

**Equivalent 80kN (18,000 lb) Single-Axle Load (ESAL)**: The effect on pavement performance of any combination of axle loads of varying magnitude equated to the number of 80kN (18,000 lb) single-axle loads required to produce an equivalent effect.

**ESAL to Failure**: The number of design 18 kip axle load cycles required to produce approximately 40 percent fatigue cracking as calculated using AAMAS equations based on Asphalt Cement Concrete Pavement Resilient Modulus and tensile strain at the bottom of the ACCP layer.

**Falling Weight Deflectometer**: A trailer mounted device which uses a falling weight to impact the pavement and transducers to pick up reflected waves. The purpose of the

device is to determine the structural capacity of the pavement system and its individual layers.

**Fatigue Cracking**: A series of small, jagged, interconnecting cracks caused by failure of the Asphalt Cement Concrete Pavement surface under repeated traffic loading (also called alligator cracking.)

**Fault**: Difference in elevation between opposing sides of a joint or crack.

**Fill:** Material used to achieve final proposed grades.

**Finite Element Method (FEM):** A method of solving the basic governing equations of a numerical model by dividing the problem area into elements in each of which the solution of the governing equations is approximated by some continuous function.

**Flexible Pavement**: See *Pavement*.

**Free Edge**: Pavement border that is able to move freely.

**Full-Depth Asphalt Pavement**: The term FULL-DEPTH (registered by the Asphalt Institute with the U.S. Patent Office) certifies that the pavement is one in which asphalt mixtures are employed for all courses above the subgrade or improved subgrade. A Full-Depth Asphalt Pavement is laid directly on the prepared subgrade.

**Functional Classification**: A method of separating and classifying streets according to their purpose or function in the network of streets, i.e. residential collectors, commercial collectors, residential locals.

**Gradation:** Soil or aggregate distributed by mass in specified particle-size ranges. It is usually expressed in percent of mass of sample passing a range of sieve sizes.

**Grade Depressions**: Localized low areas of limited size which may or may not be accompanied by cracking.

**Gross Vehicle Weight (GVW):** Total of the weight of the vehicle including its load when stationary.

**Hairline Crack**: A fracture that is very narrow in width, less than 3mm (0.12 in.).

**Heave:** A bump feature in the subgrade or reflected in the pavement that is caused by the change in volume of the soil.

**Heavy Trucks**: Two axle, six-tire trucks or larger. Pickup, panel and light four-tire trucks are not included. Trucks with heavy-duty, wide base tires are included.

**Hot Mix Bituminous Pavement:** A pavement mixture comprising an upper layer or layers of aggregate mixed with a bituminous adhesive binder such as asphalt cement, modified asphalt emulsion or asphalt rubber.

**Hydroplaning**: The dangerous action of a vehicle being driven on a pavement over which a film of rain or other water has formed; on reaching a certain speed, the vehicle's tires tend to

ride upon the water surface rather than the pavement, drastically reducing the driver's control of the vehicle.

**Incentive/Disincentive Provision (for quality)**: A pay adjustment schedule which functions to motivate the contractor to provide a high level of quality. (A pay adjustment schedule, even one which provides for pay increases, is not necessarily an incentive/disincentive provision, as individual pay increases/decreases may not be of sufficient magnitude to motivate the contractor toward high quality.)

**Instability**: The lack of resistance to forces tending to cause movement or distortion of a pavement structure.

**International Organization for Standardization (ISO):** A non-governmental international organization consisting of a federation of national standard institutes of individual countries with the purpose of identifying and developing international standards in product development and testing.

**International Roughness Index (IRI):** A measure of pavement roughness obtained from physical measurement with lasers, profilogragh or interpreted value base upon accelerometer testing. Measurements are presented in terms of inches per mile.

**Jointed Reinforced Concrete Pavement (JRCP):** Reinforced concrete pavement where reinforcement does not extend across joints.

**Lane Line**: Boundary between travel lanes, usually a painted stripe.

**Lane-to-Shoulder Drop-off**: The difference in elevation between the traffic lane and shoulder.

**Lane-to-Shoulder Separation**: Widening of the joint between the traffic lane and the shoulder.

**Layer Coefficient**: The empirical relationship between structural number (SN) and layer thickness which expresses the relative ability of a material to function as a structural component of the pavement.

**Life Cycle Cost Analyses (LCCA):** An analysis tool to calculate all costs anticipated over the life of a pavement including initial costs, maintenance estimates, periodic rehabilitation, pavement user and delay costs, salvage value, inflation, discount rate, and the analysis period.

**Lime Stabilization:** A process that involves a permanent chemical and pozzolonic reaction of the mixture of lime, water and soil followed by subgrade compaction to achieve the desired pH, swell and strength requirements.

**Lime Stabilized Subgrade**: A prepared and mechanically compacted mixture or lime, water and soil below the pavement system.

**Lime Treatment:** A process of chemically and mechanically altering the soil structure to provide a temporary stable working platform.

**Lime-Fly Ash Base**: A blend of mineral aggregate, lime, fly ash and water, combined in proper proportions which, when compacted, produces a dense mass.

**Liquid Limit:** Defines the boundary between plastic and viscous fluid states in a cohesive soil.

**Load Equivalent Factor (LEF):** A factor used to convert applications of axle loads of any magnitude to an equivalent number of 80 KN (18,000 lb) single axle load.

**Longitudinal**: Parallel to the centerline of the pavement.

**Longitudinal Crack**: A crack that follows a course approximately parallel to the center line.

**Maintenance**: The preservation of the entire roadway, including surface, shoulders, roadsides, structures, and such traffic control devices as are necessary for its safe and efficient utilization.

**Materials/Methods Specifications**: Specifications that direct the contractor to use specified materials in definite proportions and specific types of equipment and methods to place the material.

**Mechanistic Design:** An elastic or nonlinear analysis to calculate the stresses and strains in the pavement caused by traffic loads. Variables are determined by factors that vary based on subgrade and environmental conditions.

**Method Specifications**: See *Materials/Methods Specifications.*

**Milling:** Process of removing a layer of pavement using carbide tipped teeth in order to replace the surface material with out change the lines and grades of the pavement.

**Moisture Treatment:** A process involving the addition of moisture to at least 3 percent above the soil standard Proctor optimum moisture content and compaction to 95 percent density to reduce the soil's swells potential.

**Montmorillonitic Clay:** The most active principal clay mineral able to water. It has a very high shrink-swell potential due to its high affinity for water.

**North American Free Trade Agreement (NAFTA):** A free trade agreement comprising the U.S.A., Canada, and Mexico.

**Parametric Analysis**: A study of a set of physical properties whose values determine the characteristics or behavior of something; used to isolate the significance of individual variables.

**Patch**: An area where the existing pavement has been removed and replaced with a new material.

**Patch Deterioration**: Distress occurring within a previously repaired area.

**Pavement Condition Indicator (PCI)**: A measure of the condition of an existing pavement section at a particular point in time, such as cracking measured in feet per mile, or faulting measured in inches of wheel path faulting per mile. When considered collectively,

pavement condition indicators provide an estimate of the overall adequacy of a particular roadway.

**Pavement Design (design, structure design)**: The specifications for materials and thicknesses of the pavement components.

**Pavement Distress Indicator**: See *Pavement Condition Indicator*.

**Pavement Performance**: The trend of serviceability with load applications.

**Pavement Rehabilitation**: Work undertaken to extend the service life of an existing facility. This includes placement of additional surfacing material and/or other work necessary to return an existing roadway, including shoulders, to a condition of structural or functional adequacy. This could include the complete removal and replacement of the pavement structure.

**Pavement Structure (pavement)**: A combination of subbase, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed.

**Pavement, Flexible**: A pavement structure generally consisting of Asphalt Cement Concrete Pavement surfacing, base and/or subbase.

**Pavement, Rigid**: A pavement structure consisting of Portland Cement Concrete Pavement surfacing, with or without subbase.

**Performance Period**: See *Design Period*.

**Performance Specifications**: Specifications that describe how the finished product should perform over time. For highways, performance is typically described in terms of changes in physical condition of the surface and its response to load, or in terms of the cumulative traffic required to bring the pavement to a condition defined as "failure". Specifications containing warranty/guarantee clauses are a form of performance specifications. (Other than the warranty/guarantee type, performance specifications have not been used for major highway pavement components (subgrades, bases, riding surfaces) because there have not been appropriate nondestructive tests to measure long-term performance immediately after construction. They have been used for some products (e.g., highway lighting, electrical components and joint sealant materials) for which there are test of performance that can be rapidly conducted.)

**Performance-Based Specifications**: Specifications that describe the desired levels of fundamental engineering properties (e.g., Resilient Modulus, creep properties, and fatigue properties) that are predictors of performance and appear in primary prediction relationships (i.e., models that can be used to predict pavement stress, distress, or performance from combinations of predictors that represent traffic, environmental roadbed, and structural conditions.) [Because most fundamental engineering properties associated with pavements are currently not amenable to timely acceptance testing, performance-based specifications have not found application in highway construction].

**Performance-Related Specifications**: Specifications that describe the desired levels of key materials and construction quality characteristics that have been found to correlate with fundamental engineering properties that predict performance. These characteristics (for example, air voids in asphaltic pavements, and strength of concrete cores) are amenable to

acceptance testing at the time of construction. True performance-related specifications not only describe the desired levels of these quality characteristics, but also employ the quantified relationships containing the characteristics to predict subsequent pavement performance. They thus provide the basis for rational acceptance and/or price adjustment decisions.

**Planned Stage Construction**: The construction of roads and streets by applying successive layers of Asphalt Cement Concrete Pavement according to design and a predetermined time schedule.

**Plant-Mix Base**: A foundation course, produced in an asphalt mixing plant, which consists of a mineral aggregate uniformly coated with asphalt cement or emulsified asphalt.

**Plastic Limit:** Defines the boundary between non-plastic and plastic states for a cohesive soil.

**Plasticity Index:** Defines the complete range of plastic state between liquid and plastic limits for a cohesive soil.

**Polyvinyl chloride (PVC):** A hard non-transparent material which is made useful by the incorporation of plasticizers. It can be flexible or rigid and is also called vinyl.

**Portland Cement Concrete Pavement (PCCP):** Controlled mixture of Portland cement, water, and well-graded aggregate, thoroughly mixed and placed as a uniform dense pavement mass unreinforced with or without dowels to transfer load.

**Potential Vertical Rise (PVR):** A crude estimate of soil heave value as a function of plasticity index, density and moisture content.

**Pothole**: A bowl-shaped depression of varying sizes in the pavement surface, resulting from localized disintegration.

**Prepared Roadbed**: In-place roadbed soils compacted or stabilized according to provisions of applicable specifications.

**Prescriptive Specifications**: See *Materials/Methods Specifications.*

**Present Serviceability**: The ability of a specific section of pavement to serve, for the use intended, mixed traffic on the day of rating.

**Present Serviceability Index (PSI)**: A mathematical combination of values, obtained from certain physical measurements of a large number of pavements, so formulated as to predict, within prescribed limits, the Present Serviceability Rating (PSR) for those pavements.

**Present Serviceability Rating (PSR)**: The mean of the individual ratings made by the members of a specific panel selected for the purpose.

**QA/QC Specifications**: See *Quality Assurance Specifications.*

**QC/QA Specifications**: see *Quality Assurance Specifications.*

**Quality Assurance**: All those planned and systematic actions necessary to provide confidence that a product or facility will perform satisfactorily in service. Quality assurance addresses the overall problem of obtaining the quality of service, product, or facility in the most efficient, economical, and satisfactory manner possible. Within this broad context, quality assurance involves continued evaluation of the activities of planning, design, development of plans and specifications, advertising and awarding of contracts, construction, and maintenance, and the interactions of these activities.

**Quality Assurance Specifications**: A combination of end result specifications and materials and methods specifications. The contractor is responsible for quality control (process control), and the highway agency is responsible for acceptance of the product. (Quality assurance specifications typically are statistically based specifications that use methods such as random sampling and lot-by-lot testing, which let the contractor know if his operations are producing an acceptable product.)

**Quality Control**: Those quality assurance actions and considerations necessary to assess production and construction processes so as to control the level of quality being produced in the end product. This concept of quality control includes sampling and testing to monitor the process but usually does not include acceptance sampling and testing.

**Radial Tire:** A type of tire that's constructed with the reinforcing belts sideways under the tread rather than lengthwise. The tire is more flexible which reduces rolling resistance and improves fuel economy.

**Raveling**: The wearing away of the pavement surface caused by the dislodging of aggregate particles.

**Recipe Specifications**: See *Materials/Methods Specifications.*

**Reflection Cracking**: Cracks in asphalt overlays that reflect the crack pattern in the pavement structure underneath.

**Resilient Modulus:** It is the ratio of axial stress to the axial strain due to repeated load of triaxial compression test on soil samples or other pavement materials designed to simulate traffic loading.

**Resilient Modulus Test**: A measure of the modulus of elasticity of roadbed soil or other pavement material.

**Resistance Value (R-value)**: A test for evaluating bases, subbases, and subgrades for pavement thickness design.

**Roadbed**: The graded portion of a highway between top and side slopes, prepared as a foundation for the pavement structure and shoulder.

**Roadbed Material**: The material below the subgrade in cuts and embankments and in embankment foundations, extending to such depth as affects the support of the pavement structure.

**Roadway**: All facilities on which motor vehicles are intended to travel such as secondary roads, interstate highways, streets and parking lots.

**Roadway Land Use**: A classification based on the use of land adjacent or serviced by the street. The classification is used to separate streets for different volume assumptions.

**Roughometer**: A single-wheeled trailer instrumented to measure the roughness of a pavement surface in accumulated millimeters (inches) per mile or International Roughness Index (IRI).

**Rubberized Asphalt Cement**: Blend of asphalt cement and pre-vulcanized rubber.

**Rutting**: Longitudinal surface depressions in the wheel paths.

**R-Value:** Resistance value determined by a stabilometer and used for evaluating asphalt, base, subbase, and subgrade for use in pavement thickness design.

**Selected Material**: A suitable native material obtained from a specified source such as a particular roadway cut or borrow area, of a suitable material having specified characteristics to be used for a specific purpose.

**Service Life:** The number of years from initial construction to the first rehabilitation or the number of years to failure requiring major rehabilitation or reconstruction.

**Serviceability**: The ability at time of observation of a pavement to serve traffic (autos and trucks) which use the facility.

**Shoving**: Permanent, longitudinal displacement of a localized area of the pavement surface caused by traffic pushing against the pavement.

**Shrinkage Limit:** The maximum water content at which a further reduction in water content will not cause a decrease in volume of the soil mass for a cohesive soil.

**Single Axle Load**: The total load transmitted by all wheels of a single axle extending the full width of the vehicle.

**Single Axle Load Limit:** The load limit of the total load transmitted by all wheels of a single axle extending the full width of the vehicle.

**Skid Hazard**: Any condition that might contribute to making a pavement slippery when wet.

**Skid Tester** Either a small portable device or a locking wheel trailer designed to determine the skid resistance of a wetted pavement surface. Results are typically given in terms of a friction coefficient.

**Slippage Cracks**: Cracks, sometimes crescent-shaped, that point in the direction of the thrust of wheels on the pavement surface.

**SMA (Stone-Matrix Asphalt, Split-Mastic Asphalt)**: An asphalt mix design composed of large stones creating a stone to stone matrix, often containing large percentages of asphalt cement and fillers.

**Soil Cement Base**: A hardened material formed by curing a mechanically compacted intimate mixture of pulverized soil, Portland cement and water, used as a layer in a pavement system to reinforce and protect the subgrade or subbase.

**Soil Suction:** Is a capillary measure of soil's affinity to water. ASTM test method to determine total suction in units of tons per square foot

**Splitt Mastix Ashphalt (SMA):** A gap graded asphalt mix developed in Bavaria that contains crushed stone material of near uniform size along with performance enhanced binder, fibers and additives; also known as Stone Mastic Asphalt.

**Stabilization:** A process designed to increase or maintain the stability of soil by chemical agent for subgrade construction and pavement support.

**Stabilized Subgrade**: A subgrade soil that has been altered by a chemical agent to make suitable for subgrade construction and pavement support.

**Standard Deviation**: The root-mean-square of the deviations about the arithmetic mean of a set of values.

**Statistically Based Specifications**: Specifications based on random sampling, and in which properties of the desired product or construction are described by appropriate statistical parameters.

**Strategic Highway Research Program (SHRP):** A multi-year in depth federal research project provides much of the data used in determining current paving and pavement maintenance designs, materials, and methods.

**Structural Number (SN)**: An index number derived from an analysis of traffic, roadbed soil conditions, and environment which may be converted to thickness of flexible pavement layers through the use of suitable layer coefficients related to the type of material being used in each layer of the pavement structure.

**Subbase**: The layer or layers of specified or selected material of designed thickness placed on a subgrade to support a base course.

**Subbase (subbase course)**: The layer of graded sand-gravel or stabilized subgrade material between the surface of the embankment soil and the base course (or surfacing course when there is no base course).

**Subgrade**: The soil prepared to support a structure of a pavement system. It is the foundation for the pavement structure. The subgrade soil sometimes is called "basement soil" or "foundation soil".

**Subgrade Resilient Modulus**: The modulus of the subgrade determined by repeated load triaxial compression tests on soil samples. It is the ratio of the amplitude of the accepted axial stress to the amplitude of the resultant recoverable axial strain.

**Subgrade Support Value:** Is the ratio between load applied and the measured deflection.

**Subgrade, Improved**: Any course or courses of select or improved material between the subgrade soil and the pavement structure.

**Super Single:** Extra-wide, single tires that can be used as a replacement of the dual sets that have been the norm throughout the trucking industry, turning 18-wheelers into 10-wheelers.

**Surface (Surface Course)**: One or more layers of a pavement structure designed to accommodate the traffic load, the top layer of which resists skidding, traffic abrasion, and the disintegrating effects of climate. The top layer of flexible pavements is sometimes called the "wearing course".

**Surface Thickness (surfacing thickness, surface, slab thickness (rigid))**: The thickness of surfacing material, usually expressed in inches.

**Swell:** Change in volume when soil became wetted.

**Swell Test:** Test to determine heave potential of soil by measuring the odometer for percent change in thickness of the preloaded sample that was saturated with water.

**Tandem:** Two consecutive axles extending across the full width of the vehicle.

**Tandem Axle Load**: The total load transmitted to the road by two consecutive axles extending across the full width of the vehicle.

**Thermal Cracking**: Cracking occurring in pavement material introduced within the material resulting from a change in temperature.

**Traffic Equivalence Factor**: A numerical factor that expresses the relationship of a given axle load to another axle load in terms of their effect on the serviceability of a pavement structure.

**Transverse Crack**: A crack that follows a course approximately at right angles to the centerline.

**Treatment:** A process to create a temporary improvement in subgrade condition during construction.

**Tridem:** Three consecutive axles extending across the full width of the vehicle.

**Triple (Tridem) Axle Load**: The total load transmitted to the road by three consecutive axles extending across the full width of the vehicle.

**Truck Factor**: The number of equivalent 80kN (18,000 lb) single-axle load applications contributed by one usage of a vehicle. Truck Factors can apply to vehicles of a single type or class or to a group of vehicles of different types.

**Twenty-Year ESAL**: ($ESAL_{20}$) The Equivalent Single Axle Load application for a twenty-year design. The value is the product of the Load Equivalency factor for each vehicle type, the number of each particular vehicle per day, 365 days per year, and a twenty-year period.

**Unconfined Compressive Strength (UNC):** Is a measure of the soil's ability to carry load under an unconfined environment.

**Upheaval**: The localized upward displacement of a pavement due to swelling of the subgrade or some portion of the pavement structure.

**Washboarding**: See *Corrugations*.

**Water Bleeding**: Seepage of water from joints or cracks.

**Weathering**: The wearing away of the pavement surface caused by the loss of asphalt binder.

**Xeriscape**: Landscaping technique which eliminates or significantly reduces irrigation needs.

Appendix B

# Laboratory Procedures

Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (ASTM D 2216-05)

Summary of Test Method – A test specimen is dried in an oven at a temperature of 110 +/- 5°C to a constant mass. The loss of mass due to drying considered to be water. The water content is calculated using the mass of water and the mass of the dry specimen.

Report moisture content to the nearest 0.1%,

Standard Test Methods for Amount of Material in Soils Finer than the No. 200 Sieve (ASTM D 1140-00)

Summary of Test Method - This test method determines the amount of materials finer than a No. 200 sieve by washing. Two methods for determining the amount of material finer than the No. 200 sieve are as follow:

Method A- Test specimen is not dispersed prior to wash sieving.

Method B- Test specimen is dispersed by soaking in water containing a deflocculating agent prior to wash sieving.

*A dispersing agent shall be used for clays, sandy clays and clayey sands.*

Standard Test Method for Density of soil in place by the Drive-Cylinder Method (ASTM D 2937 – 04)

Summary of Test Method - This test method covers the determination of in-place density of soil by the drive-cylinder method. The test method involves obtaining a relatively undisturbed soil sample by driving or pushing a thin-walled cylinder. Field samples obtained from Shelby tubes or California barrels are acceptable.

Procedure - Carefully trim both edges of the sample so that the edges are parallel to each other. With a caliper, determine the height and diameter of the sample to the nearest 0.01 inch. Place the sample in a tared (calibrated/weight) container and determine the weight of the sample to the nearest 1 gram. Determine the moisture content of the sample in accordance with ASTM D 2216.

Standard Test Method for Unconfined Compressive Strength of Cohesive soil (ASTM D 2166-00)

Summary of Test Method - This test method covers the determination of the unconfined compressive strength of cohesive soils in the undisturbed, remolded or compacted condition, using strain-controlled application of the axial load.

This test method provides an approximate compressive strength value of the cohesive soils in terms of total stresses.

Sample Preparation:

Undisturbed Specimens

Trim both the edges of the sample so the ends are parallel to each other and are perpendicular to the longitudinal axis of the sample. Measure sample diameter and height to the nearest 0.1 inch. The sample height shall be between 2 and 2.5 times the sample diameter.

Remolded Specimens / Compacted specimens

Specimens shall be prepared to the predetermined water content and density prescribed by the engineer assigning the test. After the specimen is remolded, trim the ends so they are parallel to each other and are perpendicular to the longitudinal axis. Carefully remove sample from the mold, and determine the mass and dimensions of the test specimen as described previously.

Typically, specimens are remolded and re-compacted using ASTM laboratory compaction procedures (ASTM D 698 or D 1557). A trial and error method is necessary to determine the compaction energy required to achieve a specific density at the specified moisture content.

Standard Test Method for Liquid Limit, Plastic Limit and Plastic Index of Soils (ASTM D 4318-05)

These test methods cover the determination of liquid limit, plastic limit and plasticity index of soils. ASTM provides two (2) methods for preparation of test samples. The standard of practice in North Texas is to prepare the clay and sandy clay samples in accordance with the wet method. For consistency we recommend the wet method to be followed. However, all samples involving stabilizing agents such as lime, Portland cement and fly-ash shall be prepared in accordance with the dry method.

Standard Test Methods for One-Dimensional Swell or Settlement Potential of Cohesive Soils (ASTM D 4546-03)

ASTM provides three alternative methods for determination of swell or settlement of relatively undisturbed or compacted cohesive soil. The test methods can be used to determine the magnitude of swell or settlement under known vertical (axial) pressure or the magnitude of vertical pressure needed to maintain no volume change of laterally constrained, axially loaded specimens.

These test methods are applicable to undisturbed or remolded specimen, as follows:

Method A – Measures the free swell, percent heave for vertical confining pressures up to the swell pressure and the swell pressure.

Method B – Measures the percent heave or settlement for vertical pressure usually equivalent to the estimated in-situ vertical overburden and other vertical pressure up to the swell pressure and the swell pressure.

Method C – Measures the swell pressure, preconsolidation pressure and percent heave or settlement within the range of applied vertical pressures.

*For purposes of the subgrade investigation and pavement design, Method B shall be used.*

It is important that specimens remain inundated in water under the prescribed vertical pressure until the primary swell has been achieved. In general, tests are set up in the laboratory in such a manner as to allow the specimens to remain inundated under the prescribed vertical pressure over night. Primary swell shall be defined when, after the overnight inundation, three successive readings at two hour intervals indicate less than 0.1% swell.

Standard Test Methods for Laboratory Compaction Characteristics of Soil using Standard Effort (ASTM D 698 – 00)

These test methods cover laboratory compaction methods used to determine the relationship between water content and dry unit weight of soils compacted in a 4 or 6 inch diameter mold with a 5.5 pound rammer dropped from a height of 12 inches producing a compactive effort of 12,400 ft – lbf/ft$^3$.

Three alternative methods are provided. The method used shall be as indicated in the specifications for the material being tested. If no method is specified, the choice should be based on the material gradation.

Method A –
Mold - 4 in diameter
Material - Passing number 4 sieve
Layers - Three
Blows per layer - 25
Use – May be used if 20% or more by mass of the material is retained on the No. 4 sieve.
Other use – If this method is not specified materials that meet these gradation requirements may be tested using method Methods B or C.

Method B –
Mold - 4 in diameter
Material - Passing 3/8 inch sieve
Layers - Three
Blows per layer - 25

Use – May be used if 20% or less by mass of the material is retained on the No. 4 sieve and 20% or less by mass of the material is retained on the 3/8 inch sieve.
Other use – If this method is not specified materials that meet these gradation requirements using method Methods C.

Method C –
Mold - 6 in diameter
Material - Passing ¾ inch sieve
Layers - Three
Blows per layer - 56
Use – Shall be used if more than 20% by mass of the material is retained on the 3/8 inch sieve and less than 30% by mass of the material is retained on the ¾ inch sieve.
The 6 inch diameter mold shall not be used with method A or B.

The required sample mass for Methods A and B is approximately 35 lbs and for Method C is approximately 65 lbs of dry soil.

Sample Preparation –

ASTM specifies a minimum standing time of moisturized samples of 16 hours for CH, CL and SC soils. In North Texas, moisturized samples comprised of weathered or un-weathered Marl or Shale or soils described as “shaley” or “marly” clays shall be allowed to stand for a minimum of 72 hours.

Reporting – Laboratory compaction reports shall include description and classification (USCS) of the soil, compaction method and a graphical presentation of the data points obtained and the 100% saturation curve. The compaction curve shall be drawn based upon the engineer’s judgment and experience. *Computer generated compaction curves shall not be used as this method does not account for single point errors nor the engineer’s judgment and experience.*

Standard Test Methods for Laboratory Compaction Characteristics of Soil using Modified Effort (ASTM D 1557 – 02)

These test methods cover laboratory compaction methods used to determine the relationship between water content and dry unit weight of soils compacted in a 4 or 6 inch diameter mold with a 10 pound hammer dropped from a height of 18 inches producing a compactive effort of 56,000 ft – lbf/ft3.

Three alternative methods are provided. The method used shall be as indicated in the specification for the material being tested. If no method is specified, the choice should be based on the material gradation.

Method A -

Mold - 4 in diameter
Material - Passing No. 4 sieve
Layers - Five
Blows per layer - 25
Use – May be used if 20% or less by mass of the material is retained on the No. 4 sieve.
Other use – If this method is not specified materials that meet these gradation requirements may be tested using method Methods B or C.

Method B -

Mold - 4 in diameter
Material - Passing 3/8 inch sieve
Layers - Five
Blows per layer - 25
Use – May be used if 20% or more by mass of the material is retained on the No. 4 sieve and 20% or less by mass of the material is retained on the 3/8 inch sieve.
Other use – If this method is not specified materials that meet these gradation requirements may be tested using Method C.

Method C –

Mold - 6 in diameter

Material - Passing ¾ inch sieve

Layers - Five

Blows per layer - 56

Use – Shall be used if more than 20% by mass of the material is retained on the 3/8inch sieve and less than 30% by mass of the material is retained on the ¾ inch sieve.

The 6inch diameter mold shall not be used with method A or B.

Test Sample

The required sample mass for Methods A and B is approximately 35 lbs and for Method C is approximately 65 lbs of dry soil.

Reporting – Laboratory compaction reports shall include description and classification (USCS) of the soil, compaction method and a graphical presentation of the data points obtained and the 100% saturation curve. The compaction curve shall be drawn based upon the engineer's judgment and experience. *Computer generated compaction curves shall not be used as this method does not account for single point errors nor the engineer's judgment and experience.*

Standard Test Method for using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization (ASTM D 6276-99)

This test method provides a means for estimating the soil lime proportion requirement for stabilization of a soil. This test method is performed on soil passing the No. 40 sieve. The optimum soil lime proportion for the soil stabilization is determined by tests of specific characteristics of stabilized soil such as unconfined compressive strength or plasticity index.

This test method is used to determine the lowest percentage of lime that results in a lime pH of 12.4. Lime is not an effective stabilizing agent for all soils. Some soil components such as sulfates, phosphates, organics and so forth can adversely affect soil-lime reactions and may produce erroneous results using this test method.

ASTM specifies the test be performed in increments of 1 percent lime, starting at 2 percent and ending at 6 percent. Some North Texas soils may require lime contents greater than 6

percent to achieve a pH of 12.4, thus the engineer should be prepared to perform this test at higher lime contents.

Reporting – Results of the test shall be reported in graphical and tabular form, presenting the measured pH versus lime content, expressed as hydrated lime by dry weight of soil. The minimum lime content necessary to achieve a pH of 12.4 shall be estimated to the nearest 1%.

**LIME STABILIZATION OF SUBGRADE SOILS**

To estimate the soil-lime requirements for soil stabilization, start with Eades Grim pH method (ASTM D 6276-99). This test method is used to determine the lowest percentage of hydrated lime necessary to achieve a soil-lime pH of 12.4. Round of to the nearest 1 %.

The lime used shall come from a single source and shall meet the requirements of ASTM C977, contain at least 92% calcium and magnesium oxide, and display at least moderate reactivity for rate of slaking per ASTM C110. Obtain quick lime directly from the manufacturer no earlier than 2 weeks prior to testing. Quick lime shall be slaked in the laboratory just prior to mixing with soil. Quick lime content shall be converted to hydrated lime by multiplying the weight of quick lime by 1.28.
Potable water shall be used.

Sample Preparation
Air-dry a sufficient quantity of the soil (75 to 100 pounds) to be tested and screened through a No. 4 sieve. Store the sample in a closed container under room temperature to maintain uniform moisture.

Preparation of soil-lime Proctor material
Prepare 5 specimens with the selected percentage of lime (provided by the engineer) by dry weight. Selected percentage of hydrated lime for the standard Proctor testing is typically two to three percent above the required hydrated lime to achieve a soil-lime pH of 12.4. Mix-soil lime specimens thoroughly in different percentages of water as required. At least two specimens shall be mix on the dry side of optimum and two specimens shall be mix on the wet side of optimum. After initial mixing allow the soil-lime mixture to mellow for at least 72 hours under room temperature. Moisture condition of the soil-lime mixture shall be checked once every 24 hours after the initial mixing.

Additional water shall be added as needed to the specimens every 24 hours to maintain the original moisture condition of the mixture.

After 72 hours of mellowing time, perform the standard Proctor on the soil-lime mix. Record the unit weight and moisture content for each specimen. Draw the proctor curve between each point of moisture content and dry density. Only a hand drawn Proctor curve is acceptable. A computer generated curve shall not be used. Record the optimum moisture content to the nearest 1.5% and maximum dry density to the nearest to 0.5 lbs.

**Preparation of Unconfined Compressive strength specimens and One-Dimensional Swell**

Prepare samples with three different percentages of lime by weight. Use the results of the lime pH series to determine the initial hydrated lime percent for the unconfined compressive strength sample. Add 2 percent for the second soil-lime sample, and add 4 percent of hydrated lime for the third soil-lime sample.

**Lime**

Prepare enough samples to remold three sets of samples for each percentage of the soil-lime mixture. A total of nine remolded samples will be needed. Mix soil with the assigned percentage of hydrated lime at +3% of its optimum moisture content. Place the soil-lime mixture in a sealed bag labeled with the corresponding percentage of hydrated lime. Mellow the soil-lime samples for at least 72 hours under room temperature. Moisture content of the soil-lime mixture shall be checked once every 24 hours. Additional water shall be added to the specimens as needed every 24 hours to maintain the moisture content of the soil-lime mixture at +3% of its optimum moisture content.

After 72 hours of mellowing time, remold the samples to 95% compaction of soil-lime standard Proctor ASTM D 698. The samples shall be remolded in a split Proctor mold. Remold three sets of samples for each of the soil-lime mixtures. Two of the samples from each set will be tested for unconfined compressive strength and one sample will be tested for swell. After the compactive effort has been applied, trim the remolded

specimen and open the split Proctor mold to extract the sample. Immediately test one of the samples for swell in accordance with ASTM D4546, with an overburden of 200 psf.

The rest of the samples shall be completely wrapped at least three times with saran wrap to maintain its moisture content. After the samples have been wrapped, they should be taped using masking tape on the top, bottom and around the center. Label the samples to indicate the date of remolding, sample number and percentage of hydrated lime used in the soil-lime mixture.

Cure the remolded samples in a temperature controlled oven under a temperature of 100°F for 5 days. Remove one of the samples from each set after the 5 days curing, unwrapped and let it stand for at least 1 hour prior to testing for unconfined compressive strength in accordance with ASTM D 2166. A minimum unconfined compressive strength of 160 psi (ASTM D 2166) is desired. If the unconfined compressive strength of 160 psi was not achieved, allow the other sample to cure in the 100°F oven for another 2 days for a total of 7 days of curing. After the 7 days of curing, test the remaining samples for unconfined compressive strength.

If the unconfined compressive strength of 160 psi was not achieved after 7 days of curing, repeat the process by remixing the soil with higher percentages of hydrated lime at one percent intervals until the required strength is achieved or up to a maximum of 12 percent of hydrated lime was used.

**Lime and Portland cement or lime and fly-ash**

If the unconfined compressive strength of 160 psi is not achieved with 12 percent hydrated lime, then a combination of lime-Portland cement or lime and fly-ash shall be used. The percentage of hydrated lime from lime-pH series test result shall be used for mixing with either Portland cement or fly-ash. Repeat the process of remolding the different sets of lime-Portland cement or lime-fly ash mixtures as described in the previous section using a fixed percentage of hydrated lime combined with two percent increases in Portland cement or fly-ash. The minimum amount of Portland cement or fly-ash used as a starting point shall be 3 percent.

If the unconfined compressive strength of 160 psi was not achieved after 7 days of curing, repeat the process by remixing the soil with hydrated lime and higher percentages of Portland cement or fly-ash at one percent intervals until the required strength is achieved.

## CEMENT STABILIZATION OF SUBGRADE SOILS

Soil materials with Plasticity Index of 20 or less may be stabilized with Portland cement. Portland cement used for testing shall conform to the requirements of ASTM C 114. Obtain the Portland cement no earlier than 2 weeks prior to testing.

The minimum amount of Portland cement used shall be 3 percent. Prepare enough samples to remold three sets of samples for each percentage of the soil-cement mixture. A total of nine remolded samples will be needed. Mix soil with the assigned percentage of Portland cement at +3% of its optimum moisture content.

Remold the samples within 30 minutes of mixing to 95% compaction of soil-cement standard Proctor ASTM D 698. The samples shall be remolded in a split Proctor mold. Remold three sets of samples for each of the soil-cement mixtures. Two of the samples from each set will be tested for unconfined compressive strength and one sample will be tested for swell. After the compactive effort has been applied, trim the remolded specimen and open the split Proctor mold to extract the sample. If the soil is clay or very clayey, immediately test one of the samples for swell in accordance with ASTM D4546, with an overburden of 200 psf.

The rest of the samples shall be completely wrapped at least three times with saran wrap to maintain its moisture content. After the samples have been wrapped, they should be taped using masking tape on the top, bottom and around the center. Label the samples to indicate the date of remolding, sample number and percentage of Portland cement used in the soil-cement mixture.

Cure the remolded samples in a temperature controlled oven under a temperature of 100°F for 5 days. Remove one of the samples from each set after the 5 days curing,

unwrapped and let it stand for at least 1 hour prior to testing for unconfined compressive strength in accordance with ASTM D 2166. A minimum unconfined compressive strength of 300 psi (ASTM D 2166) is desired. If the unconfined compressive strength of 300 psi was not achieved, allow the other sample to cure in the 100°F oven for another 2 days for a total of 7 days of curing. After the 7 days of curing, test the remaining samples for unconfined compressive strength.

If the unconfined compressive strength of 300 psi was not achieved after 7 days of curing, repeat the process by remixing the soil with higher percentages of Portland cement at two percent intervals until the required strength is achieved or up to a maximum of 8 percent total of Portland cement was used.

**Determining Sulfate Content in Soils – Calorimetric Method**
(Tex – 145 E)

This method shall be used to determine the soluble sulfate content of soil by turbidimetric techniques.