Research Report
KTC-07-18/SPR 339-07-1F

KENTUCKY TRANSPORTATION CENTER

COMPACTION OF MIXTURES OF HARD ROCKS AND SOFT SHALES AND NON-DURABLE SHALES USING IMPACT COMPACTORS

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COMPACTION OF MIXTURES OF HARD ROCKS AND SOFT SHALES AND NON-DURBABLE SHALES USING IMPACT COMPACTORS

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in cooperation with the
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16. Abstract

Impact roller compaction has been used to improve embankment and highway subgrades in South Africa, Australia, Europe, and China and other areas of the world. In September of 2003, the International Technology Scanning Program, sponsored by the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the National Cooperative Highway Research Program of the Transportation Research Board, identified this technology as one of several foreign technologies and innovations that could significantly benefit U.S. transportation systems. The technology was high-lighted at the Fifth International Conference on the Bearing Capacity of Roads and Airfields in 1998 (Pinnard). To date, however, usage of these types of non-circular compactors is at an infant stage in the United States. The capability of this type of roller to compact soils and break-down and compact mixtures of hard rocks and clayey shales to a high percentage of maximum dry density obtained from modified compaction (AASHTO T-180) could provide many benefits. The main purposes of this report are to describe potential areas where this class of compactors might be applied in constructing transportation facilities and demonstrate some major potential benefits that could be obtained. Potentially, as shown in this report, this class of compactors could vastly improve the stability of many transportation facilities.

17. Key Words **Compaction, embankments, impact roller, non-circular, shales, stability, settlement, creep, bridge approaches, shear strength**		18. Distribution Statement Unlimited, with approval of the Kentucky Transportation Cabinet	
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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

EXECUTIVE SUMMARY

Rolling impact compaction has been used in a variety of applications over the last twenty years. These non-circular compactors have been manufactured in a variety of shapes which include three-sided, four-sided, and five-sided. This type of compactor has been used successfully in making ground improvements in South Africa, Australian, Europe, and China, for example. Impact roller compactors densify the ground to significant depths and have been used to break down concrete pavements and rocks. While this technology has been used in other parts of the world, it appears to be in an infant stage of usage in the United States. The International Technology Scanning Program, sponsored by the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the National Cooperative Highway Research Program of the Transportation Research Board, identified this technology as one of several foreign technologies and innovations that could significantly benefit U.S. transportation systems. The technology was high-lighted at the Fifth International Conference on the Bearing Capacity of Roads and Airfields in 1998 (Pinnard) and was shown to have made vast improvements in roadway construction.

The objective of this study was to examine the applicability and potential uses in Kentucky to improve roadway construction. In particular, the potential of the impact compactors to break down and compact mixtures to such a degree that embankment settlements are made very small. Failure to break down mixtures of hard rocks and soft shales and achieve good compaction in Kentucky led to numerous settlement and slope stability failures of interstate and parkway embankments in the seventies and eighties. Those failures required millions of dollars to repair. Moreover, the large embankment settlements led to numerous failures of pavements which eventually had to be replaced. The failures prompted numerous research studies to determine the causes and to develop new compaction specifications. Although the new specifications have been shown to have saved millions of dollars in maintenance expenditures, the new impact rollers could still vastly improve the compaction of highway embankments. The potential ability of these types of compactors to break down mixtures of hard rocks (durable) and shales (non-durables) and uniform shales and compact (including different types of soils) to dry densities approaching 95 to 100 percent of maximum dry density and optimum moisture obtained from AASHTO T-180 could yield numerous benefits. As shown in this study, the factors of safety against failure of embankments and subgrades increase significantly when the dry densities approach dry densities obtained from AASHTO T-180.

Specifications pertaining to the use of impact compactors have ranged from simple to complex (Avalle, 2004; Bouazza and Avalle, June 2006, and Avalle, December 2006). Based on one experience source, earthwork specifications may take the form of "method specifications" or "performance specifications". Method specifications specify the construction methods to be used while performance specifications specify that the "requirements to be met by test in the finished product." Accordingly, various hybrid specifications have been used for impact roller projects. An assessment of each situation must be made to determine the most appropriate method to use. For example, in some cases a detailed trial program (test pads for materials used on a particular project) may be performed in advance of the earthwork project to provide data for analyzing and assessing the effects of impact rolling. Consequently, both a method specification may be formulated based on the assessment and yet some testing (performance specification) may be performed to judge the final results of the impact roller. It is recommended that test pads be constructed at selected sites in Kentucky in order to build an experience base for formulating compaction specifications for various types of Kentucky soils and rocks using impact roller compactors.

It is also recommended that the goal of new specifications is to achieve dry densities approaching those obtained from AASHTO T-180 ("modified compaction"). As shown in this report, and based on laboratory triaxial tests, soils and clayey shales compacted at, or near, maximum dry density and optimum moisture content obtained from AASHTO-T-180, increases the cohesive component of strength significantly when compared to the cohesive strength component obtained when the materials are compacted at, or near, the dry density and optimum moisture content obtained from "standard compaction, or AASHTO T-99.

As shown herein, increasing the shear strength of compacted soils and rocks (and mixtures of hard rocks and soft clayey shales) using roller impact compactors can provide the following potential benefits as well as other potential benefits:

- The factor of safety against failure of an embankment increases. Consequently, embankment stability increases and many embankment failures could be prevented.

- Settlements (and differential settlements) of embankments, which can adversely affect pavement performance, decrease significantly. Large differential settlements of embankments, as shown by past research and experience, can cause premature pavement failures and require costly maintenance.

- The use of roller impact compactors could aid in mitigating, or decreasing, the magnitude of settlement of an embankment foundation, especially where shallow foundation soils occur.

- Improved compaction can significantly mitigate the differential settlement that occurs between bridge approach embankments and bridge abutments resting on piles that are founded on bedrock, or hard soils.

- Increasing the stability of soil subgrades by improving compaction can improve pavement performance. By increasing the density of the soil subgrade, the permeability of the subgrade decreases. This will aid in mitigating the depth of penetration of water flowing downward and through base materials. However, it will not prevent the eventual development of a soft zone at the top of untreated clayey subgrades. Chemical stabilization will still be needed to prevent the development of a soft zone in the top of clayey subgrades.

- Use of impact roller compactors could improve the use of the full depth reclamation of existing pavements. Currently, this concept has been used to renovate in place shallow flexible pavements—approximately 6 to 8 inches. Since the depth of compaction of the impact roller compactor is greater than depth of compaction of conventional, circular compactors, a deeper lift of pavement and soil subgrade could be pulverized and mixed with a chemical admixture to gain a very strong base layer for an asphalt overlay. Hence, thicker, existing flexible pavements could be renovated in place then is currently done.

INTRODUCTION

Objectives and Scope

Achieving good compaction of embankment soils and rocks has been and continues to be a major problem in Kentucky, as well as many other areas of the world. Impact roller compactors have been used widely in South Africa in road building and have been used in Europe and China. To date, this type of compactor has not been widely used in the Unites States. In September of 2003, the International Technology Scanning Program, sponsored by the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the National Cooperative Highway research Program of the Transportation Research Board, identified this technology as one of several foreign technologies and innovations that could significantly benefit U.S. transportation systems. The technology was high-lighted at the Fifth international Conference on the Bearing Capacity of Roads and Airfields in 1998 (Pinnard). The major objective of this discussion is to examine and describe potential geotechnical applications of non-circular, impact compactors to problems frequently encountered in the transportation industry, especially the compaction of mixtures of hard rocks and soft clayey shales.

A view of one type of impact roller compactor is shown in Figure 1. This particular impact compactor is four sided and pulled by a large tractor. One of the main current applications is breaking concrete pavements. Based on videos by the manufacturer, the four-sided impact compactor appears well suited for breaking solid concrete pavements into pieces that can be easily removed. Depending on the area of contact and contact stress, the impact compactor reportedly can compact a layer near or equal to dry densities obtained from Modified Compaction, AASHTO T-180. Achieving dry densities of this magnitude can significantly and greatly improve highway stabilities, as shown by the evaluations presented below.

Figure 1. View of one type of non-circular impact compactor (courtesy of Impact Roller Technology(IRT)).

Applications where the non-circular, impact compactors have been applied in the transportation industry and attempts to identify and discuss other potential applications are presented herein. Suggested research, applicable to Kentucky soils and rocks, that appears to be needed to maximize benefits of applying the impact compactor to other potential applications is briefly described and discussed.

Conventional Compaction Equipment and the Non-circular Impact Compactor

Typical equipment used in conventional compaction is circular. A sheepsfoot, self-propelled compactor is illustrated in Figure 2. This type of equipment is used to compact soils and clayey shales. Another type of compactor frequently used to compact granular soils is a self-propelled, vibratory (circular) roller, as depicted in Figure 2. Sometimes the different pieces of equipment, as well as other equipment, are used together. When a lift of material, such as a mixture of soft shales and hard rocks is placed, water may be applied to slake the soft shales and a disc may used to mix the material. A heavy sheepsfoot roller is used to further break-down the mixtures of hard and soft rocks; the vibratory roller is used to vibrate and densify the different rocks of the mixture. See current specifications of the Kentucky Transportation Cabinet in the Appendix.

Figure 2. Construction of three experimental shale embankments (after Hopkins and Beckham, 1998).

A basic difference between conventional compaction equipment and the impact compactor, as shown in Figure 3, is the area of contact each compactor makes with the material that is being compacted. The area of contact, A_{noncir}, is larger than the area of contact, A_{cir}, of the circular compactor. Consequently, the noncircular compactor has greater potential to compact at a larger depth, $d_{I_{noncir}}$, than the compaction depth, $d_{I_{circ}}$, of the circular compactor. Provided that the impact compactor has sufficient mass, as each side of the compactor impacts the lift of material, the impact energy created in this action may potentially be large enough to compact the material to a density state approaching modified compaction, as obtained from AASHTO T 180.

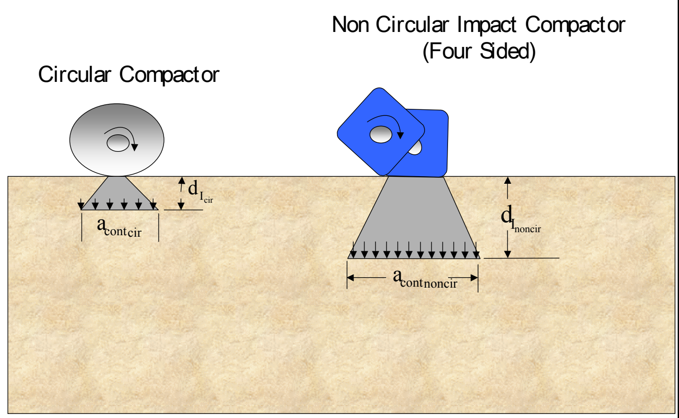


Figure 3. General comparison of contact areas of circular and non-circular contact areas and depths of influence

As the energy of compaction increases from some low-energy state to a high energy state (Figure 4), such as achieved from modified compaction (AASHTO T 180), the optimum moisture content decreases and the maximum dry density increases (Hopkins, January 1998). One of the feature applications of the impact compactor is to breakdown existing concrete pavements. Hence, it would appear that the impact compactor would have sufficient to compact soils and rocks to dry densities and moisture contents approaching values obtained from modified compaction. If this is the case, then several benefits would be derived from using this type of compactor.

By increasing the dry density obtained from standard compaction to a value approaching the dry density obtained from modified compaction, the shear strength of the soil, or rock mass, increases and the permeability decreases. As a result, the stability, or factor of safety, of an embankment slope increases and the ingress of subsurface and surface waters into the fill decreases. By compacting soils using a high level of energy, the material passes from a plastic state to an elastic-plastic state to an elastic state, as illustrated in Figure 5. As a result, settlement of the soil decreases. Settlement of the embankment after compaction will be less using the higher energy compactive effort (modified) than if standard compaction is used.

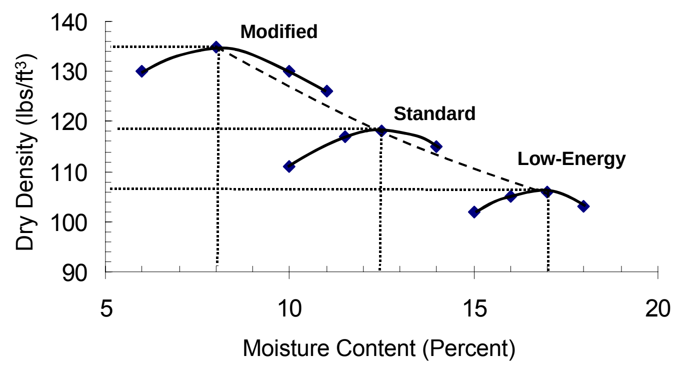


Figure 4. Relationship among maximum dry density, optimum moisture content, and compaction energy.

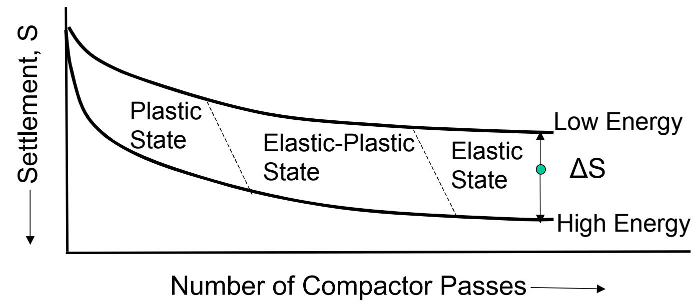


Figure 5. General relationships among settlement of soil layer, number of compactor passes, compactor energy, and stress-strain state of soil layer.

APPLICATIONS OF IMPACT COMPACTOR TO ROADWAY CONSTRUCTION

In addition to the use of the impact compactor to breakdown existing concrete pavements, some other potential applications of the impact compactor include the following:

- Compaction of mixtures of hard rocks and soft clayey shales, uniform shales, and soils.
- Bridge approach embankments.
- Improvement of highway subgrades using impact roller compaction.
- Full depth reclamation.

Although the first item listed above is the primary focus of this report, the other three situations are discussed herein because of their importance and the potential benefits that could be obtained using roller impact compactors.

Compaction of Mixtures of Hard Rocks and Soft Clayey Shales, Uniform Shales, and Soils.

Past Compaction Problems

Obtaining good compaction of embankments constructed of mixtures of hard rock and soft clayey shales is a major problem in Kentucky, as well as many other areas of the world. A prime example illustrating the large settlements resulting from inadequate compaction in past years is shown in Figure 6. During the seventies and eighties, highway embankments on long stretches of Interstates I 71 and I 75 in Kentucky were constructed using mixtures of hard limestone rock and soft clayey shales. Compaction lift thicknesses of some 30-36 inches were used and the mixtures were compacted using track dozers. The majority of the embankments were constructed using materials from the Kope and Fairview Geologic Units (Ordovician). Both units contain interbedded layers of limestone and clayey shales. The clayey shale in the Kope Formation is dominant while the limestone dominates the Fairview Formation.

Figure 6. Typical settlements of embankments observed on I 75 and I 71 shortly after construction.

Because of the loosely compacted state of the mixtures of the hard limestone rock and soft clayey shales, large voids were present in the embankment. As surface and subsurface water entered the embankment, the clayey shales in the matrix degraded (slaked) into weak soils. Slake-durability research conducted on the clayey shales showed that those materials have very low slake-durability indices (Hopkins and Gilpin, 1981; Hopkins and Deen, 1983). As the clayey shales in the embankment matrix degraded, large settlements occurred as shown in Figure 6. Eventually a large number of the embankments completely collapsed. Numerous stability analyses conducted on the failing embankments shown that the angle of internal friction, ϕ', from back analyses was only about 19-20 degrees, although consolidated-undrained triaxial compression tests (with pore pressure measurements) performed on well-compacted specimens yielded a value of about 26 degrees (Munson and Mathis, 1981-1983; Hopkins,1973). Because considerable movements had occurred in the embankments, the cohesive component, c’, was assumed to be near zero in the back analyses.

The large degree of slaking, soaking, and loss of strength of the Kope and Fairview clayey shales, as well as other Kentucky clayey shales, is illustrated in Figures 7 and 8 (Hopkins et al, 1983 and 1995). In Figure 7, laboratory CBR values of a number of compacted shales are compared. In the first series of tests, the CBR tests were performed on the shale specimens “as compacted”. That is, the specimens were not subjected to any soaking period. CBR values of the clayey shales in the series in an unsoaked state ranged from about 15 to 45. After soaking for several days, CBR values of the same clayey shale specimens (colored bars) ranged from about 0.5 to 6.

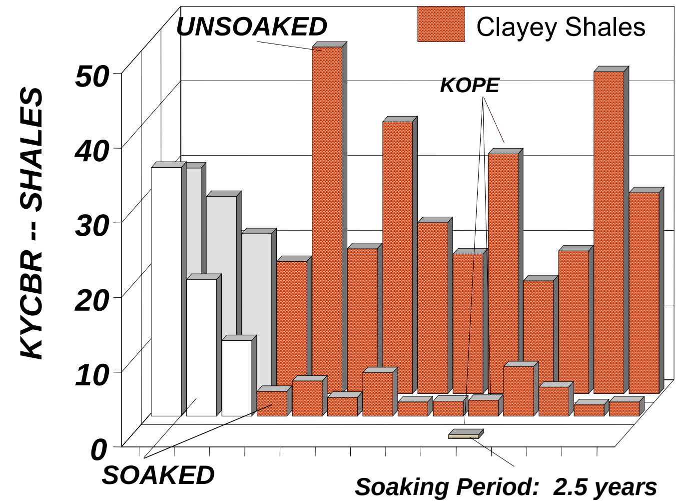


Figure 7. Effect of water on the CBR strength of clayey shales.

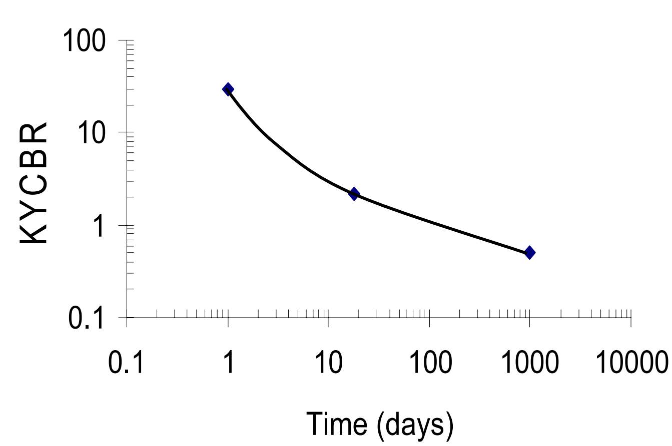


Figure 8. Effect of long-term soaking on the CBR strength of compacted Kope clayey shale.

The affect of long-term soaking on the CBR strength of Kope clayey shale is illustrated in Figure 8. The CBR of the unsoaked specimen was about 30. Based on the soaking period in the Kentucky CBR testing procedure, the CBR decreased to 2.5. The soaking period was about two weeks. A special test was conducted that included soaking the specimen for 2.5 years. The CBR value decreased to 0.5.

Development of Provisional and Permanent Compaction Specifications

Because of the many failures of embankments on I 75 and I 71, several research studies were conducted over about three decades in efforts to improve the compaction of shales and mixtures of soft shales and hard rocks, such as limestone (Hopkins et al, 1971, 1972, 1986, October 1986, Munson and Mathis 1981-1983). Provisional compaction specifications, which proposed using heavy compactors, were developed from those studies and they were used to build three experimental shale embankments (Hopkins and Beckham, 1998). Based on the success of the experimental embankments, the provisional shale specifications were used to construct about 85 miles of KY Route 9 (referred to originally as the AA- highway or Alexandria-Ashland Highway). Many of those embankments on this highway were built with mixtures of soft shale and hard limestone from the Kope Geological Formation. In conjunction with using the provisional shale compaction specifications, embankment slopes of 2.5 horizontal to 1.0 vertical and 3.0 horizontal to 1.0 vertical were recommended and used on most of the embankments on this roadway. The combination of those recommendations aided in avoiding major embankments failures on Ky Route 9. Based on the contract expenditures, it was estimated that about 2-3 million dollars per mile was spend on repairing embankment failures on I 75 and I 71. By preventing embankment failures, the savings to the Cabinet on KY Route 9 alone was estimated to be about 170 to 255 million dollars.

Although the provisional specifications have been adopted as permanent shale compaction specifications (See Appendix), there are still improvements to be made in compacting mixtures of soft shales and hard rocks. As shown in Figure 4, the approach to breaking down shales involves using several different types of equipment. Initially, water is added to the loose lift to slake the shales and then a disc is used to bring about more breakdowns. More degradation occurs using a self-propelled sheepsfoot. This action is followed by using a vibratory compactor to cause more breakdown and to compact the mass tightly. In efforts to improve the compaction of mixtures of hard rocks and soft shales, provisional standards were developed in the eighties. Numerous research studies were conducted. To check the provincial compaction specifications, three shale embankments were constructed. The provincial specifications (See Appendices A and B) involved using several pieces of compaction equipment. As noted in the provisional specifications, two different heavy-duty compactors were specified to break down the mixtures of hard rock and soft shales and when the fill material may consist of a uniform type of shale.

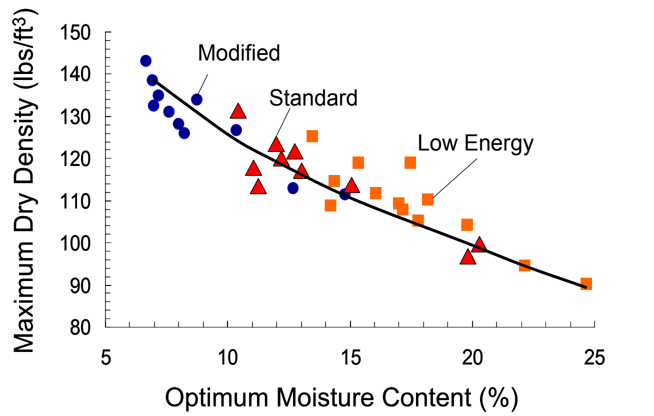


Figure 9. Increase of dry density and decrease of optimum moisture content as compactive energy increases.

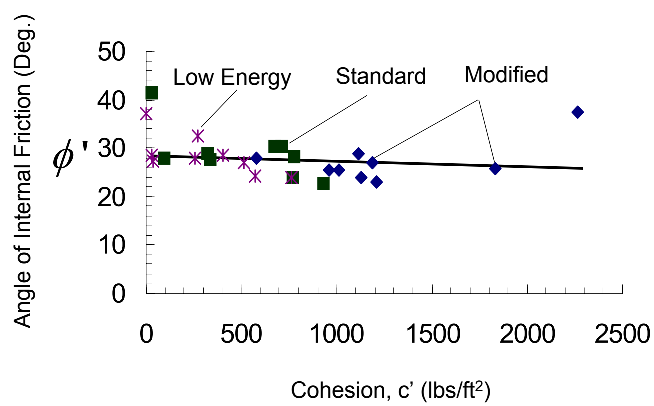


Figure 10. Angle of internal friction as a function of cohesion of shale specimens remolded at different compactive energies (After Hopkins, 1998).

Importance of Compaction Energy

The importance of compactive energy, or the parcel of energy transferred by the compactor to a roadway layer of clayey materials, is illustrated in Figures 4, 9, and 10. Maximum dry density of compacted clayey material increases as compactive energy increases. The optimum moisture content decreases.

The effect of increasing compactive energy on the shear strength of clayey materials is illustrated in Figure 10. Shear strength parameters obtained from triaxial tests performed on Kentucky shale specimens and remolded at different compactive energies are shown in Figure 10 (Hopkins, January 1998) and Table 1. In this series of tests, consolidated-undrained, (isotropic) triaxial tests with pore pressure measurements were performed on nine selected different types of shales that are found abundantly in Kentucky. The triaxial specimens were remolded to maximum dry densities and optimum moisture contents obtained from three different compactive energies. Compactive energies used were “modified–AASHTO T-180” (56,000 ft-lbf/ft^3), “standard-AASHTO T-99)”, {12,400 ft-lbf/ft$^{3)}$, and a low-energy effort (about 8500 ft-lbf/ft^3-created).

Results of the triaxial tests are summarized in Table 1. As compactive energy increases, dry densities of the different types of compacted shales increase and range from about 90 lbs/ft^3 (low energy compaction) to 140 lbs/ft^3 (Modified compaction). As the compactive energy increases

Table 1. Results of consolidated (isotropic)–undrained triaxial compression tests with pore pressure measurements obtained for specimens that were remolded to three different compaction energies.

Shale Name	Modified Compaction T 189		Standard Compaction T 99		Low-Energy Compaction *	
	Effective Stress Parameters					
	ϕ'	c'	ϕ'	c'	ϕ'	c'
	(Degrees)	(lbs/ft^2)	(Degrees)	(lbs/ft^2)	(Degrees)	(lbs/ft^2)
New Albany	37.4	2269	41.4	28	37.2	0
Hance	25.9	1833	30.4	678	27.1	512
Drakes	28.8	1114	30.4	710	32.6	268
Nancy	25.5	962	28.7	320	28	260
Osgood	27	1186	28.2	776	28.5	402
New Providence	25.5	1013	27.5	335	27.2	40
Kope	27.8	576	28	92	28.4	27
Crab Orchard	23	1211	23.9	768	24.2	572
Newman	23.9	1130	22.7	932	24	765

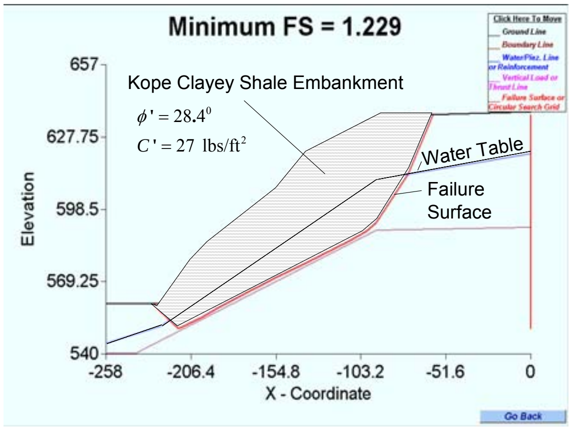
**Devised*

Note: Triaxial specimens were remolded to 100 percent of maximum dry density and optimum moisture content obtained for each selected compaction energy.

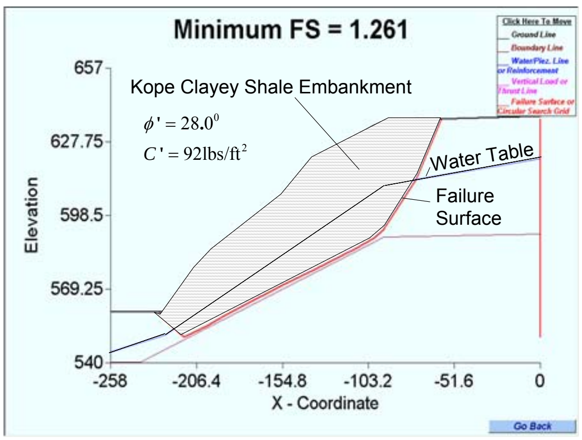
from the low-energy effort to modified compaction, the cohesive component of the strength increases significantly. However, the angle of internal friction, φ', changes very slightly.

By increasing compactive effort, stability of the embankment increases because the cohesive strength component increases. Moreover, by increasing the compactive energy to a level approaching that used in modified compaction, the stability of a clayey slope is affected significantly. This aspect can be illustrated by the results of slope stability analyses using the two examples shown in Figures 11 and 12 and in Table 2. The slope selected for illustrating the effect different compactive energies has on stability was an actual case embankment failure that occurred in Kentucky on I 75. The shear surface of the slope was non-linear. Analyses were performed using two stability models developed by Hopkins (1991) and Slepak-Hopkins (1995). Effective stress parameters (Table 1) of the two selected clay shales—Kope and New Providence clayey shales-- were used to illustrate the effect of compactive energy on the magnitude of the factor of safety, as shown in Table 2. As the compactive energy increases, the factor of safety increases. Based on the strength parameters of the Kope shale, the factor of safety increases from a value of 1.19, at a low energy compactive effort, to 1.63, at modified compaction. The factor of safety increases about 37 percent. Similarly, the factor of safety increases from 1.17 to 1.88, or about 60 percent, using the shear strength parameters of the New Providence clayey shale. Relationships between the factor of safety and the effective stress parameter, c', and compactive energy is shown in Figures 13 and 14, respectively. These figures illustrate the benefit of using modified compaction to increase slope stability of an embankment.

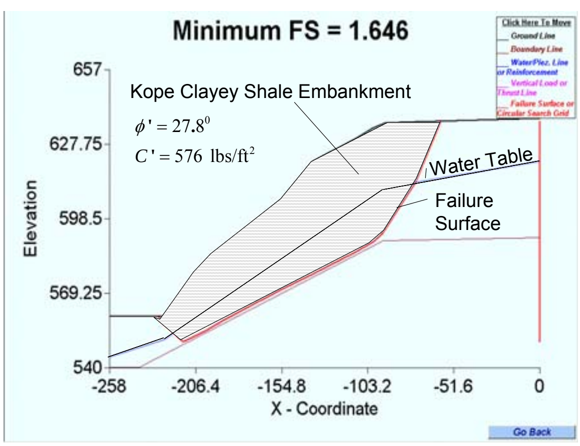
Specifying modified compaction also provides another potential benefit. As shown in Figures 4 and 5, the range of optimum moisture contents of the shales decrease as the compactive energy increases. At modified compaction, the range of optimum moisture contents (about 6 to 15 percent approach the range of natural moisture contents of the shales (1.7 to 12 percent).



(a). Factor of safety using low-energy compaction shear strength parameters.

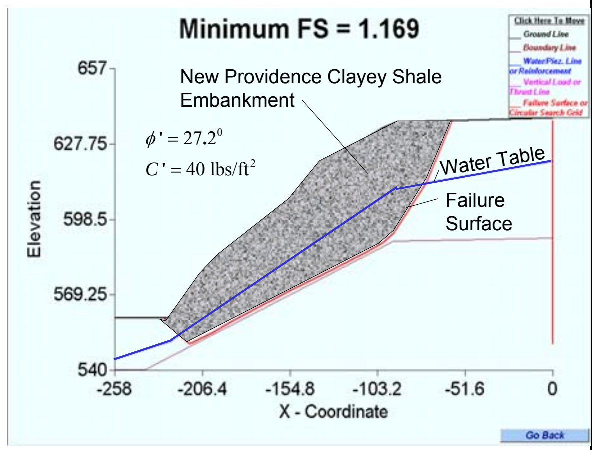


(b). Factor of safety using standard compaction shear strength parameters.

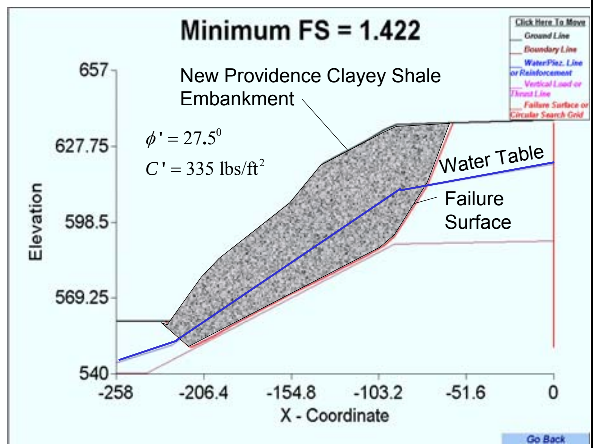


(c). Factor of safety using modified compaction shear strength parameters.

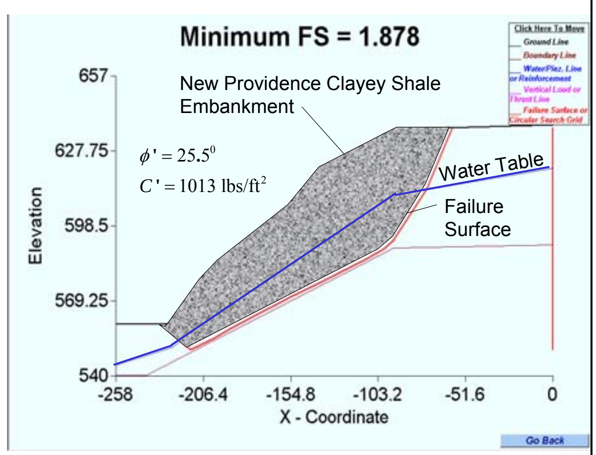
Figure 11. Typical embankment (Kope Shale) failure encountered on I 75 in Grant County, Kentucky.



(a). Factor of safety using low-energy compaction shear strength parameters.



(b). Factor of safety using standard compaction shear strength parameters.



(c). Factor of safety using modified compaction shear strength parameters.

Figure 12. Assumed embankment using shear strength parameters of the New Providence Shale.

Table 2. Results of slope stability analyses using different shear strength parameters obtained from triaxial tests conducted on specimens remolded at different levels of compactive energies.

Type of Compacted Shale Embankment	Compaction Energy Level[1]	Angle of Internal Friction[2], φ' (Degrees)	Cohesion[1], c' (lbs/ft^2)	Factor of Safety[3], FS	
				Hopkins (Method of Slices)	Slepak-Hopkins (Perturbation Model-Free ody))
Kope Clayey Shale	Low	28.4	27	1.19	1.23
	Standard	28.0	92	1.24	1.26
	Modified	27.8	576	1.63	1.65
New Providence Clayey Shale	Low	27.2	40	1.17	1.18
	Standard	27.5	335	1.42	1.43
	Modified	25.5	1013	1.88	1.89

1. *Compactive energy levels:*
   Low Compaction—8,500 ft-lbf/ ft^3
   Standard Compaction (AASHTO T-99)—12,400 ft-lbf/ ft^3
   Modified Compaction (AASHTO T-180)—56,000 ft-lbf/ft^3
2. *Effective stress parameters obtained from consolidated-undrained isotropic triaxial tests with pore pressure measurements (after Hopkins, xxxx). Triaxial specimens compacted to 100 percent of maximum dry density and optimum moisture content .*
3. *Slope stability analyses performed using slope stability models by Hopkins (1991) and Slepak-Hopkins (1995).*

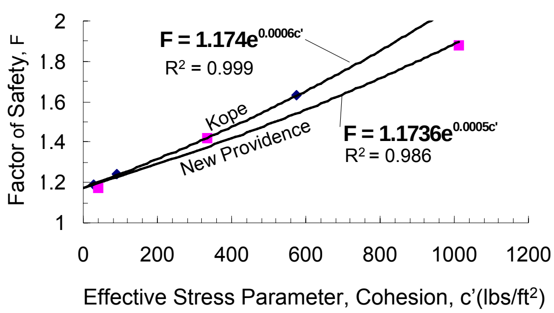


Figure 13. Factor of safety as a function of the effective stress parameter, c'.

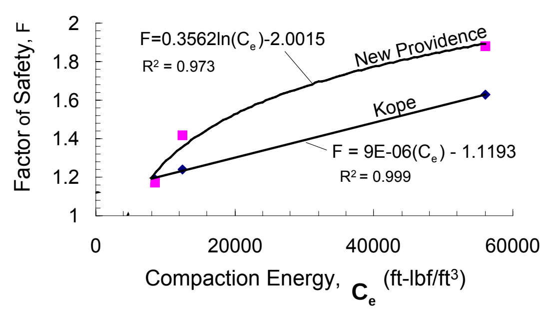


Figure 14. Factor of safety as a function of compaction energy.

Bridge Approach Embankments

Numerous research studies have been conducted to determine the causes of the settlement of bridge approach embankments. A number of factors cause the "bump at the end of the bridge" (Hopkins, February 1969; Hopkins and Deen, 1970; Hopkins and Scott, 1970; Hopkins, 1985) or differential settlement that may occur between the bridge and approach embankment. In many situations in Kentucky, pile-end-bend abutments are used, as shown in Figure 15. H-piles are usually driven through the approach fill to bedrock. In this case, the bridge cannot settle but the approach embankment is free to settle. Consequently differential settlement between the bridge and the approach embankment develops.

Factors Leading to Differential Settlement

To mitigate the differential settlement requires that each individual case be evaluated and the factors that can lead to settlement must be addressed. Major factors that contribute to the differential settlement are depicted pictorially in Figure 15 and may be listed as follows:

- Poor compaction of the approach embankment soils and rocks
- Primary and secondary settlement of the foundation of the approach embankment
- Primary compression and secondary compression of the embankment
- Creep of the embankment
- Loss of material from behind the abutment due to erosion caused by a lack of drainage measures to control the flow of surface waters from the bridge or from the approach pavement flowing water because of a lack of drainage
- Toe erosion by stream and loss of embankment support
- Loss of material from the face of the abutment due to poorly designed drainage.
- Lack of bridge design coordination between the bridge designers and geotechnical engineers
- Dynamic forces acting on the bridge approach pavements when large loaded trucks “drop” off the edge of the bridge onto the approach pavement.
- Rapid Drawdown

In designing measures to mitigate the differential settlement between the bridge and the approach pavement, the above factors much be considered. Several different combinations of those factors may combine to cause the approach settlement or only one factor may act individually to cause

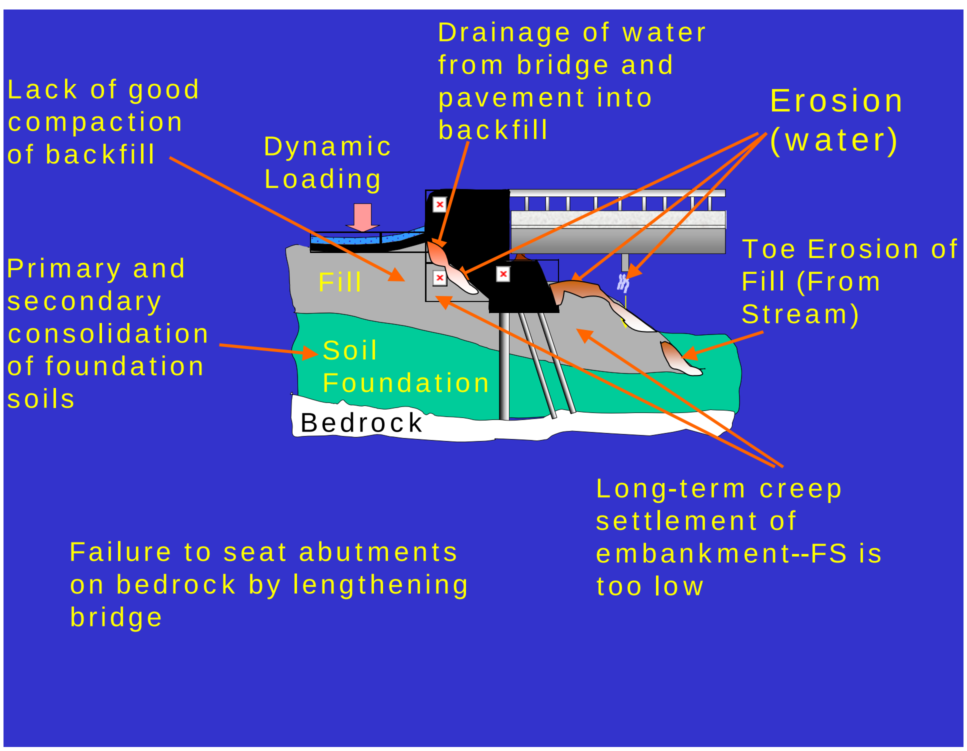


Figure 15. Summary of important factors that cause the differential settlement between bridge approach pavements and the end of the bridge.

the settlement. The situation at each bridge much be considered individually.

Importance of Compaction

One of the most important factors leading to differential settlement is the magnitude of compaction of the approach fill. This may be illustrated by the approach embankment shown in Figure 16. This approach fill consisted of a conglomerate of weathered, soft shale and hard durable rock, Ohio black shales, Bedford and Borden shales, and sandstones. When this fill was constructed, most lift thicknesses were two feet or more. Little compaction was used because, initially, the fill rocks appeared to be sound and the specifications at that time allowed thick lift thicknesses. A detailed geotechnical investigation of this site was performed as part of ongoing research at that time to study the causes of settlements of approach pavements. Thin-walled tube samples of the foundation and embankment were obtained. Consolidated-undrained triaxial tests with pore pressure measurements and consolidation tests were performed on the undisturbed tube samples.

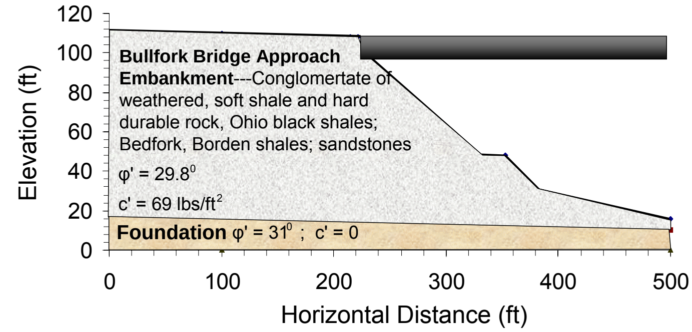


Figure 16. Eastern bridge approach embankment at Bullfork Creek, I 64

Settlement gages were installed on the top of the foundation soils to monitor foundation settlement as the embankment was constructed. As shown in Figure 17, the measured, or observed, settlement was about 17 inches. Consolidation tests were performed on samples obtained from the foundation before construction. A consolidation analysis was performed to predict the magnitude of primary and secondary consolidation. As shown in figure 17, the predicted settlement was about 14 inches. In this case, primary settlement did not affect the settlement of the approach pavements because it was completed before the pavements were constructed. The secondary compression did not affect the approach settlement because it was extremely small.

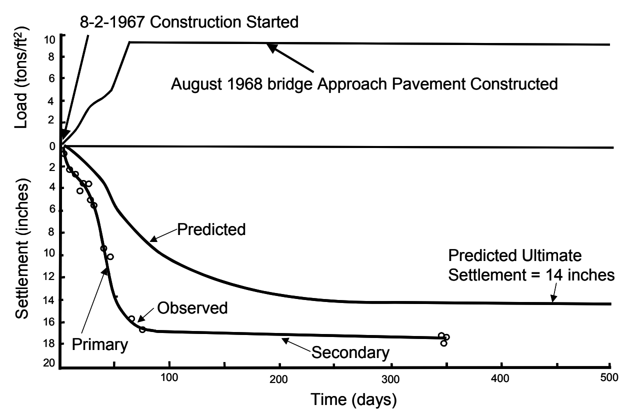


Figure 17. Settlement of the foundation of the Eastern bridge approach embankment at Bullfork Creek, I 64.

Based on measurements at other sites, primary settlement of approach foundations is frequently completed before the construction of the approach pavements. In some cases, however, secondary foundation settlement (Hopkins, 1969; Hopkins and Scott, 1970) may occur after the bridge approach pavements have been constructed, as illustrated by measurements obtained at the I 24 bridge site over Eddy Creek (Lake Barkley) in Western Kentucky. The primary consolidation of the foundation ended near the time that fill loading was completed. Secondary consolidation continued, although the loading had reached a constant value.

Secondary consolidation as a function of the logarithm of time is linear, as shown in the figure 18, and decreases with each log cycle. If the approach pavements been constructed near the end of primary consolidation, then the approach pavements settlement would have settled about 6 inches by the end of 27.4 years (10,000 days). Fortunately, at this site early construction had been specified and the approach pavements were built nearly 9 years (3,285 days) after construction. Hence, secondary settlement was a primary factor to consider in this case. However, by the time the approach pavements were constructed, secondary settlement was a small amount of the total settlement.

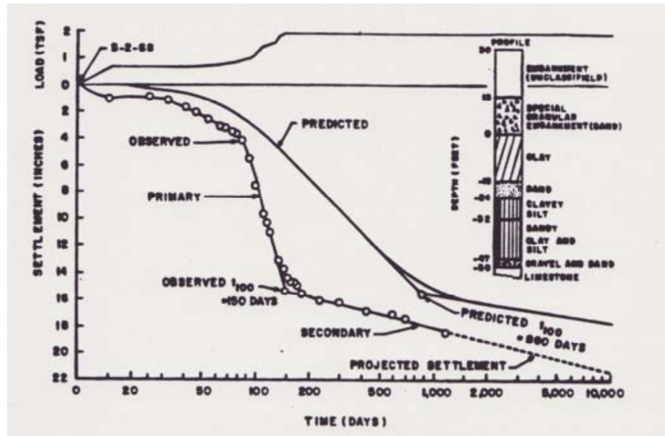


Figure 18. Primary and secondary consolidation of the foundation soils at the I 24 crossing across Eddy Creek at Lake Barkley.

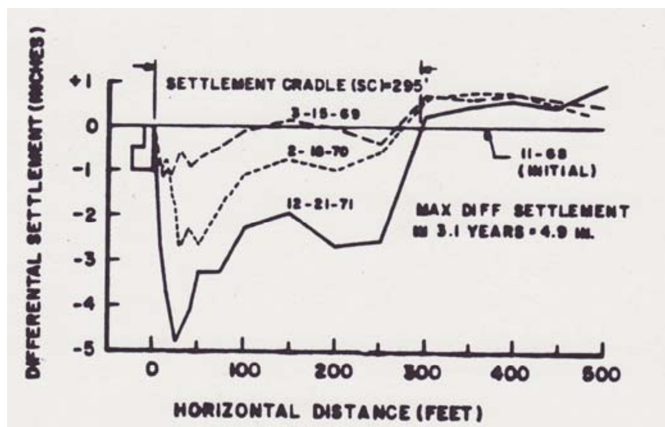


Figure 19. Settlement crater at the outside edge of the eastern approach pavement of the I 64 bridges across Bullfork Creek. In Rowan County.

Although primary and secondary settlement of the foundation soils had ended at the I 75 Bullfork Creek site before construction of the approach pavements, settlement of the approach pavements occurred as shown in Figure 19. The approach settlement crater extended about 300 feet from the end of the bridge. When the settlements (on an arithmetic scale)) were graphed as a function of the logarithm of time, the relationship is linear (Hopkins 1985) as shown in Figure 20. The settlement may be described as embankment creep as verified by slope inclinometers installed in the approach embankment showed horizontal movement, which caused downward movement of the embankment. The measurements at this site continued for about 4 years.

The linear relationship shown in Figure 20 was observed at six other instrumented bridge approach embankment sites. Long-term measurements of approach pavements at those sites continued for several years. At one site the measurements of the creep settlement continued for some 9 years, as shown in Figure 21. Again, settlements were linear with the logarithm of time.

Development of a Procedure for Estimating Creep Settlement

The linearity of the relationships formed the basis of developing a procedure for estimating creep settlement. By projecting the linear relationship to the end of 27.4 years, an approximate value could be obtained of the total creep settlement that occurs at an approach embankment site. After 27.4 years, creep settlement becomes almost insignificant as it proceeds into the next

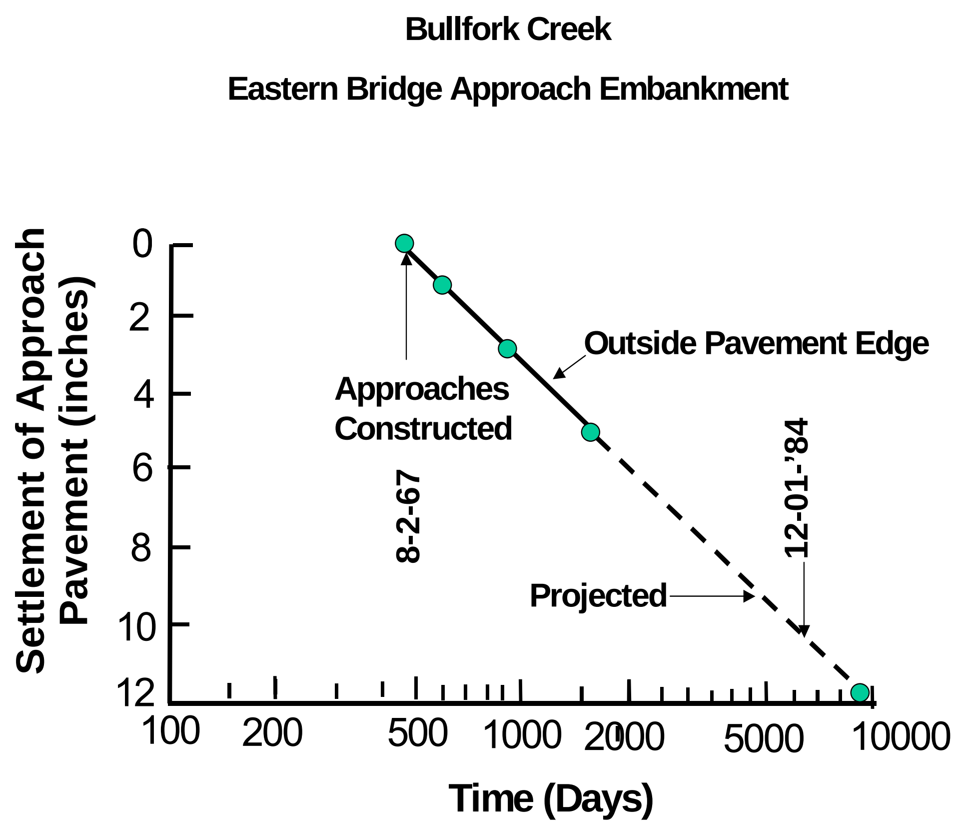


Figure 20. Settlement (creep) of the approach pavements as a function of the logarithm of time.

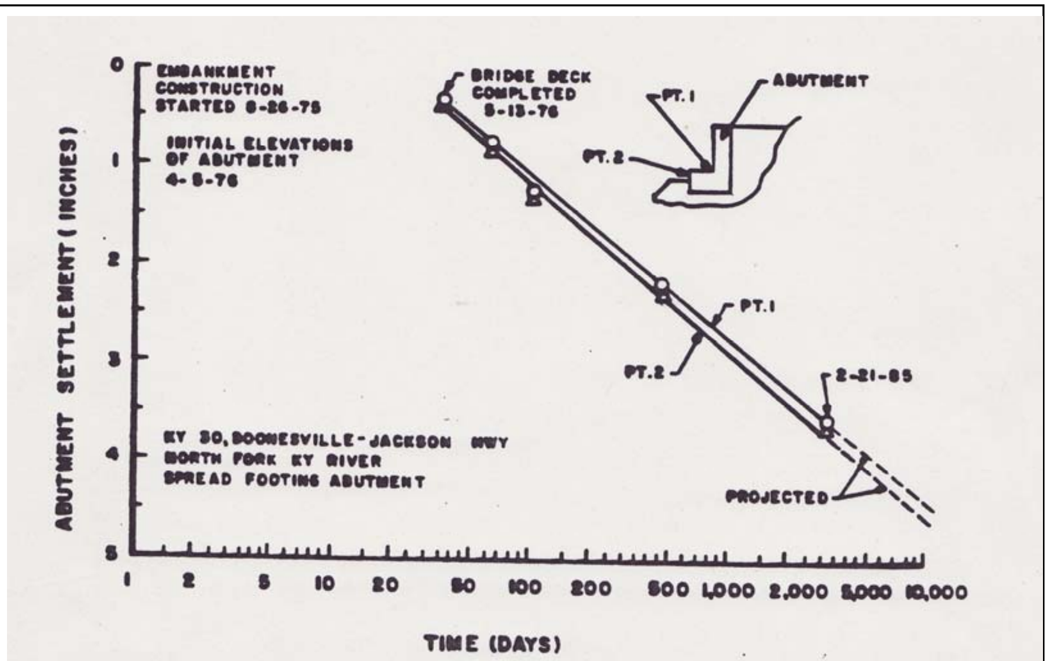


Figure 21. Settlement (creep) of the left side of abutment as function of logarithm of time , Ky 30, Bonnesville-Jackson Road (spread footing abutment).

logarithm cycle. In an attempt to relate the factor of safety of the approach embankment, to creep settlement, detailed slope stability analyses of the seven study sites were performed using effective stress parameters obtained for each approach embankment and foundation soils at each site. Reciprocal values (an index, c_{ss}) of the linear slope relationships (see example in Figures 20 and 21) were graphed as a function of the ratio of the embankment height of the approach embankment to long-term factor of safety. This relationship is illustrated in Figure 22, or

$$\frac{1}{c_{ss}} = 47370 F_r^{-1.5013}, \tag{1}$$

where

c_{ss} = slope of the settlement–logarithm of time curve (coefficient),
F_r = ratio of the embankment height, H_e, to the long-term factor of safety, F_{lt}.

Because the relationship of approach embankment settlement as a function of the logarithm of time is linear (see Figures 20 and 21), then the coefficient of secondary settlement and shear

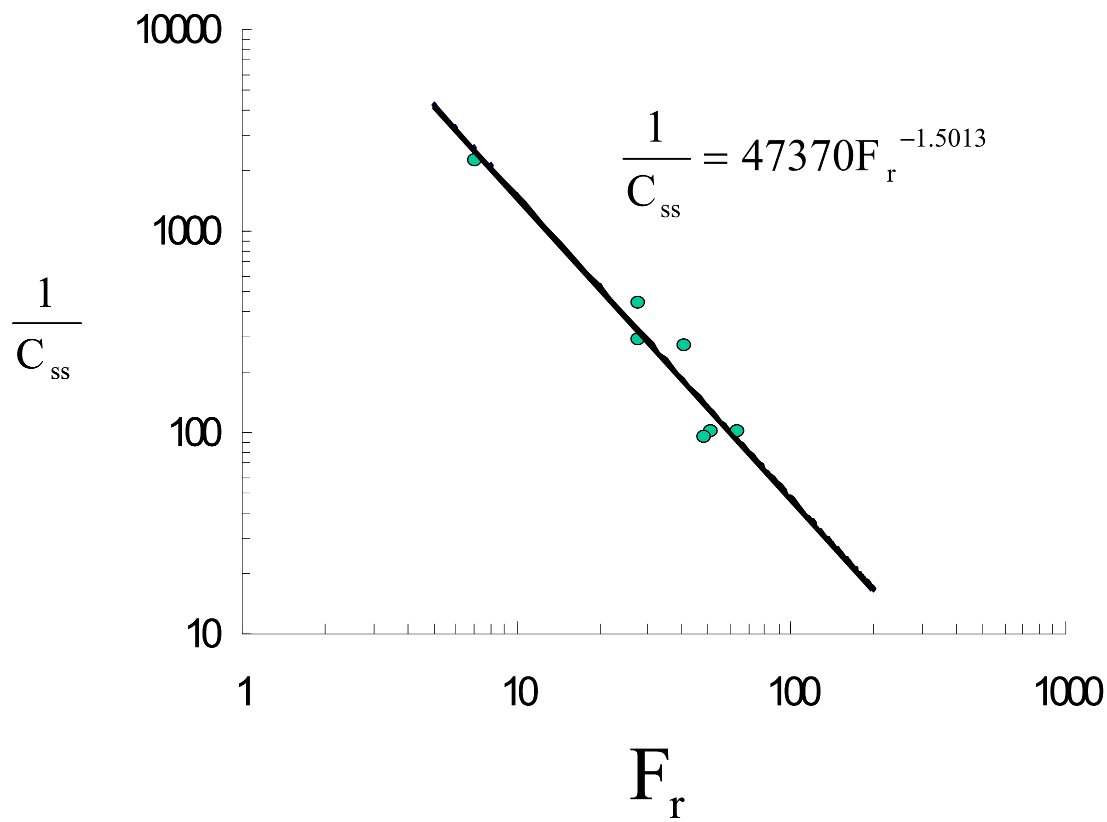


Figure 22. The reciprocal of the index, c_{ss}, and the ratio of the factor of safety to embankment height.

strain (or embankment creep), c_{ss}, may be estimated from the same type of equation used to estimate secondary consolidation, or

$$c_{ss} = \frac{H_{ss}/H_e}{\log_{10}\left(t_{ss} / t_c\right)} \tag{2}$$

where

H_{ss} = settlement (inches) of the approach embankment due to secondary compression and shear strain (estimated projected value at the end of 27.4 years),
H_e = height of approach embankment (inches),
t_c = time (days) of placement of approach pavment (the time between the start of embankment construction and the placement of the approach pavement, and
t_{ss} = time (days at the end of significant secondary compression and shear strain of the approach embankment=10,000 days).

Solving Equation 1 for the term, c_{ss},

$$c_{ss} = 10^{(-1.5013\log_{10}F_{It}+4.7370)}, \tag{3}$$

and substituting the expression of c_{ss} into Equation 4

$$H_{ss} = c_{ss} H_e \log_{10}\left(\frac{t_{ss}}{t_c}\right), \tag{4}$$

and

$$H_{ss} = \left(10^{(1.5013\log_{10}F_r-4.7370)}\right) H_e \log_{10}\left(\frac{t_{ss}}{t_c}\right). \tag{5}$$

Example illustrating the Mitigation of Creep Settlement
To illustrate how Equation 5 may be used to estimate the long term settlement (due to shear strain and secondary compression) of an approach embankment, the example approach embankment shown in Figure 16 may be used for illustration. In this example, two types of shales, New Providence and Kope clayey shales, were selected to illustrate the method of mitigating the magnitude of approach settlement. Slope stability analyses (based on a search analyses) were performed to determine the minimum factor of safety corresponding to each pair of shear strength parameters obtained for the low-energy, standard, and modified compaction energies. Shear strength parameters (corresponding to each compaction energy) used for the two

Table 3. Factors of safety obtained from different slope stability methods using different effective stress parameters for the approach slope in Figure 16.

Type of Compacted Shale Embankment		Compaction Energy Level[1]	Angle of Internal Friction[2], φ' (Degrees)	Cohesion[2], c' (lbs/ft^2)	Hopkins (Method of Slices)	F_{lt} Slepak-Hopkins (Perturbation Model-Free Body))	Bishop
					Factor of Safety[3]		
Bull Fork Approach Embankment	Embankment	Standard	29.8	69	1.24	1.25	1.25
		Assumed increasing compaction energy (hypothetical)	29.8	600	1.57	1.59	1.57
			29.8	1,000	1.74	1.76	1.76
			29.8	2,000	2.14	2.14	2.16
	Foundation	"As Sampled"	31.0	0.0			
Kope Clayey Shale		Low	28.4	27	1.12	1.12	1.12
		Standard	28.0	92	1.19	1.20	1.20
		Modified	27.8	576	1.52	1.52	1.52
New Providence Clayey Shale		Low	27.2	40	1.08	1.09	1.09
		Standard	27.5	335	1.40	1.40	1.40
		Modified	25.5	1013	1.65	1.65	1.64

1. *Low Compaction—8,500 ft-lbf/ ft^3*
   Standard Compaction (AASHTO T-99)—12,400 ft-lbf/ ft^3
   Modified Compaction (AASHTO T-180)—56,000 ft-lbf/ft^3
2. *Effective stress parameters from consolidate-undrained with pore pressure measurements.*
3. *Slope stability analyses performed using methods developed by Hopkins (1991), Slepak-Hopkins (2001), and Bishop (1954).*

shales are shown in Table 1. Factors of safety obtained for the approach embankment built of the materials described in Figure 16 and assuming that the approach embankment had been built of each type of shale are summarized in Table 3. Using shear strength parameters obtained from consolidated-undrained triaxial tests with pore pressure measurements of thin-walled tube samples obtained from the approach embankment, the minimum factor of safety was only 1.25. Assuming that the cohesive component of strength, c', would increase with compactive effort, the factor of safety ranges upward to 2.12 for assumed hypothetical values of cohesion. Factors of safety for the Kope shales ranged from 1.12, based on low-energy shear parameters (Table 1 and 2), to 1.52, based on modified compaction energy. Similarly, for the New Providence shale, the factors of safety increase from 1.09 to 1.65, respectively.

Foundation soils of the approach site were only 12 feet thick, as determined from geotechnical borings. Consolidation analyses estimated that both primary and secondary settlement would be completed by the time the approach pavements were placed. Consequently, any settlement of the approaches would be due to secondary compression and shear strain within the approach embankment. This was confirmed by foundation settlement measurements (See Figure 17). To illustrate the use of Equation 5 and the importance of compactive effort, factors of safety obtained from the slope stability analyses of the bridge approach embankment (Figure 16) may be used in that equation. An estimate of long-term shear strain and creep of the approach embankment may be computed as described below. The following parameters are known –or assumed--(for the case of materials listed in Figure 16):

F_{lt} = minimum long-term factor of safety = 1.25 (See Table 3),

H_e = height of embankment 98 ft,

$$F_r = \frac{H_e}{F_{lt}} = 78.4$$

$t_{ss} \approx$ 800 days after start of construction (estimated value),

$t_c \approx$ 10,000 days (or 27.4 years--it is assumed that after this time period the settlement will be insignificant).

Substituting the values above into equation 5, the estimated long-term approach settlement is

$$H_{ss} = \left(10^{\left(1.5013\log_{10}\left(\frac{98}{1.25}\right)-4.7370\right)}\right)98\text{ft}\left(\frac{12\text{ in}}{\text{ft}}\right)\log_{10}\left(\frac{10{,}000\text{days}}{800\text{days}}\right).$$

H_{ss} = 16.5 in.

This amount of embankment creep would be unacceptable. One way to reduce long-term settlement would be to increase the long-term factor of safety of the embankment. To minimize creep settlement requires an increase in the long-term factor of safety. This could be accomplished in different ways. But the first consideration should be an attempt to compact the approach fill to a larger dry density than the dry density obtained from “standard” compaction (AASHTO T-99). Achieving a larger dry density may be obtained using the impact roller compactor. To explore this possibility, factors of safety obtained from different compactive efforts were examined.

As shown in Table 4, if the existing materials used to construct the embankment could have been compacted to larger dry densities than those obtained from standard compaction, then creep settlement could have been made smaller. Based on the assumption that the cohesive component could be increased by increasing compactive effort, factors of safety increase from 1.25 to values of 1.59, 1.76, and 2.14, respectively, for hypothetical cohesive values of 600, 1,000, and 2000 lb/ft^2, respectively. As shown in Table 4, creep settlements estimated from Equation 5 decrease from 16.5 to values of 11.5, 9.9, and 7.4 inches, respectively. If the factor of safety could be increased to about 3.0, then the settlement could be reduced to about 4.6 inches, as shown in Figure 23. Increasing the factor of safety to this value might be accomplished by decreasing the slope from 2:1 to maybe 3:1 or 3.5:1. Other alternate means might include building the approach embankment fully, or partially, with hydrated lime-soil or Portland cement mixtures to increase shear strength. Increasing the factor of safety beyond about 3.0, as shown in Figure 23, would not reduce the settlement significantly below 4.6 inches. However, design measures could be adopted that would reduce future settlement from an estimated value of 16.5 inches to about

Table 4. Summary of factor of safety and estimated creep settlements.

Type of Compacted Shale Embankment		Compaction Energy Level[1]	Angle of Internal Friction[2], φ' (Degrees)	Cohesion[1], c' (lbs/ft^2)	FS Slepak-Hopkins (Perturbation Model-Free Body))	Creep and shear strain Settlement (inches)
Bull Fork Approach Embankment (As Built)	Embankment	Standard	29.8	69	1.25	16.5
		Modified	29.8	600	1.59	11.5
			29.8	1000	1.76	9.9
			29.8	2000	2.14	7.4
	Foundation	"As Sampled"	31.0	0.0		
Kope Clayey Shale		Low	28.4	27	1.12	19.4
		Standard	28.0	92	1.20	17.5
		Modified	27.8	576	1.52	12.3
New Providence Clayey Shale		Low	27.2	40	1.09	22.0
		Standard	27.5	335	1.40	13.9
		Modified	25.5	1013	1.65	10.9

4.6 inches. The magnitude of 4.6 inches could be tolerated and may require some future maintenance.

Estimated creep settlement based on the assumptions that the approach embankment was built of Kope or New Providence clayey shales are summarized in Table 3. In each hypothetical case, the impact roller compactor could be effective in reducing creep settlement. In the case of the Kope shales, creep settlement could be reduced from an estimated value of 19.4 inches to 12.3 inches. In the case of the New Providence shales, creep settlement could be reduced from 22 inches to 10.9 inches. In each case, additional design measures would probably be needed to increase the factor safety to reduce the creep settlement. Another potential approach might include delaying constructing the approach pavements. As shown in Figures 20 and 21, creep settlement (arithmetic scale) as a function of the logarithm of time is linear. Hence, creep settlement decreases rapidly with the passage of each log cycle, that is, a short delay in paving may cause most of the creep settlement to occur before paving the approaches, provided the factor of safety of the approach embankment is sufficiently large.

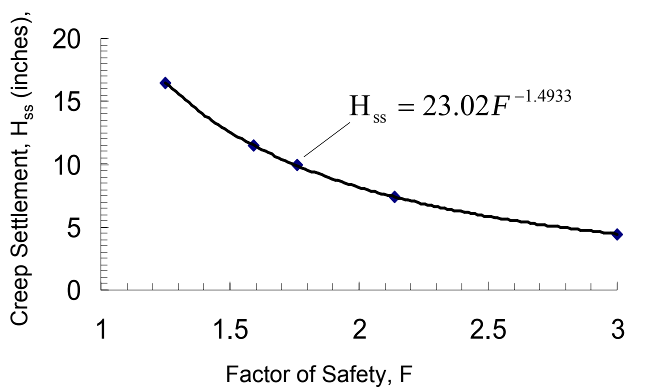


Figure 23. Factor of safety of the eastern bridge approach embankment (I 64 bridges across Bullfork Creek in Rowan County, Kentucky) as a function of creep settlement.

Improvement of Highway Subgrades Using Impact Roller Compaction

Mechanical Compaction

Typical requirements in Kentucky, and many agencies, specify that highway pavement subgrades be compacted to 95 percent of standard compaction (AASHTO T 99) and ± 2 percent of optimum moisture content. The importance of compaction of the soil subgrade during early construction may be analyzed using the Perturbation limit equilibrium model (Slepak and Hopkins, 1995; Hopkins et al. 2005). In the early construction example (before paving) shown in Figure 24, it is assumed that the subgrade was constructed using New Providence clayey shales. Bearing capacity analysis using the low energy-compaction effective stress parameters (Table 1) yields a factor of safety of only 0.77, or failure under the tire contact stress (assumed 80 lb/ft^2) exerted by dual-wheel tires. Using effective stress parameters associated with standard compaction, the factor of safety was only 1.07 (Figure 25). In both cases, large settlement and rutting of the subgrade would occur under construction traffic. During construction this could impede construction and may pose a problem in constructing the pavement.

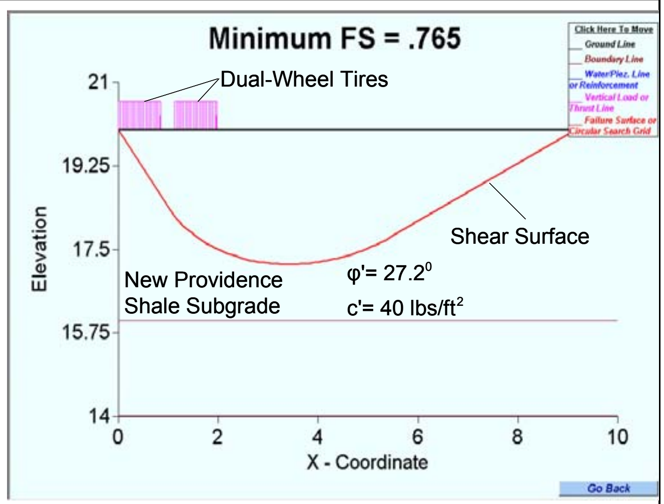


Figure 24. Bearing capacity analysis of a subgrade constructed with New Providence clayey shales using low-energy effective stress parameters.

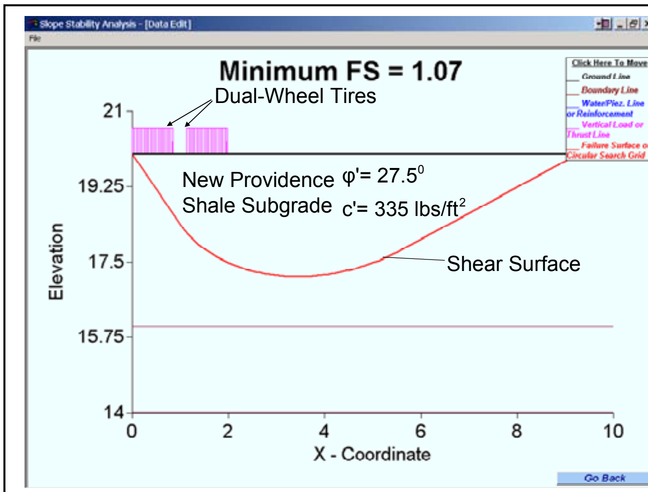


Figure 25. Bearing capacity analysis of a subgrade constructed with New Providence clayey shales using standard effective stress parameters.

If modified effective stress parameters ($\phi'=25.5^0$;$c'=1013$ lbs/ft^2) are used, then a factor of safety 1.56 is obtained. The larger compaction energy provides a vast improvement in the stability of the subgrade during construction. Using the impact roller compactor could vastly improve the performances of subgrades during and after construction.

Bearing capacity analyses were also performed for the case where the complete flexible pavement had been built on the subgrade constructed of New Providence shale. In this example, the flexible pavement was assumed to be 6 inches thick while the aggregate base was assumed to be 12 inches thick. Based on the low energy parameters, the factor of safety was only 1.11. Stability was improved some using the standard parameters. The factor of safety increased to 1.41. Using the modified parameters, the factor increased significantly to 1.93, as shown in Figure 26.

If pore pressures are introduced into the clayey subgrade, the factors of safety decrease. This condition, of course, can occur after a period of time when the subgrade is exposed to flowing water in the aggregate base, as well as subsurface water. Pore water pressures may be introduced into the clayey shale layer by using the pore pressure ratio, r_u, or

$$r_u = \frac{u}{\gamma h}, \tag{2}$$

where u = pore water pressure,
γ = unit weight of water, and
h = depth of the point in the soil mass below the ground surface.

In this case, the pore water pressure is assumed constant throughout the clayey shale subgrade. Based on the standard parameters and introducing a pore pressure ratio of 0.5, the factor of safety decreases from 1.40 to 1.05, or near failure. Based on the modified parameters and using the pore pressure ratio of 0.5 in the clayey subgrade, the factor of safety decreases from 1.93 to 1.60. The factor of safety is still in a range of good stability. The use of impact compactors has the potential to vastly improve the bearing capacity of the flexible pavement when it is used to obtain dry densities and moisture contents near those obtained from modified compaction. Impact compactors generally disturb the top portion of a subgrade to a depth of about 4 to 8 inches and a smooth drum roller may be needed for finishing the surface of the subgrade.

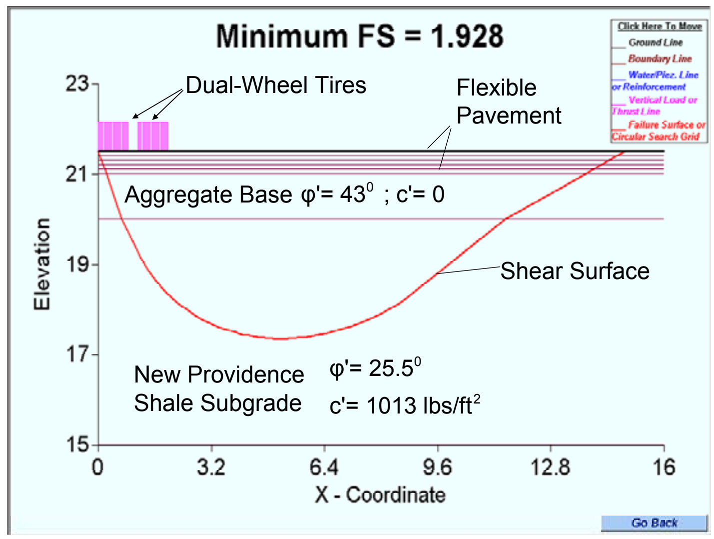


Figure 26. Bearing capacity analysis of a subgrade constructed with New Providence clayey shales using modified compaction effective stress parameters.

Chemical Stabilization and Mechanical Compaction

Using mechanical impact compaction, as illustrated above, can potentially increase the bearing strength and stability of subgrades. However, in the long-term, the top of the clayey subgrade may weaken when water flows outward, as well as downward, in the aggregate base. As water seeps downward, the top of clayey subgrades absorb water and swells. As the clayey subgrade swells, it loses strength (and cohesive strength). The weight of the flexible pavement and aggregate base are usually not sufficient to prevent the swelling process. As shown in Figure 27 (percentile test value as a function of moisture content), in situ moisture contents measured at the tops of soil subgrades in Kentucky are larger than moisture contents measured at some depth (from thin-walled tube samples) below the tops of the soil subgrades (Hopkins et al. 2002: Hopkins et al, 2006)).

Based on extensive studies and numerous tests, the in situ CBR value of clayey subgrades where only mechanical (standard) compaction has been used was only about 1.8 at the 85th percentile test value, as shown in Figure 28. Although impact compaction could improve the bearing strength of the untreated clayey subgrade, the top of the subgrade may still weaken when subjected to water. Tests would have to be performed to evaluate the swelling potential of clayey specimens compacted to dry densities and moisture contents approaching modified compaction. As shown in Figure 28,

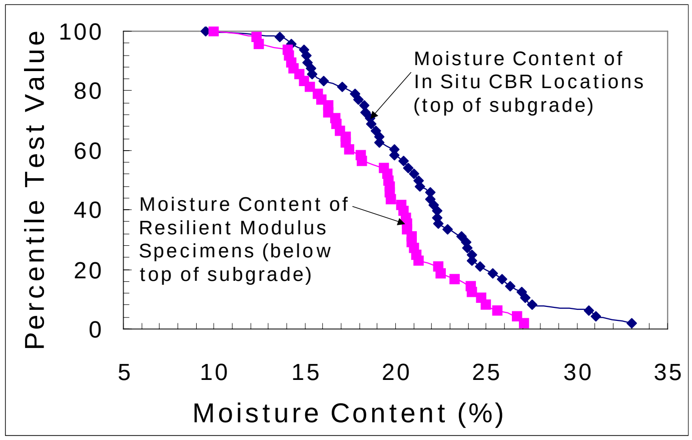


Figure 27. Comparison of moisture contents of specimens obtained from in situ CBR locations and resilient modulus specimens obtained below the top of the untreated soil subgrades.

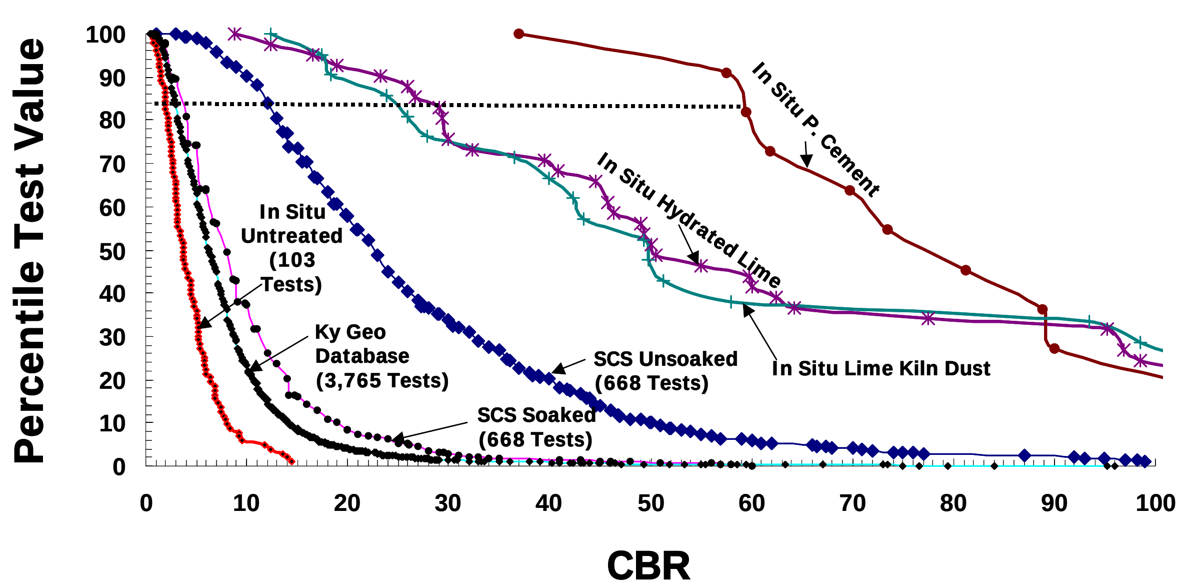


Figure 28 Comparisons of the percentile test values of in situ values of CBR for untreated and chemically treated subgrades and laboratory values of CBR.

mixing chemical additives with the clayey subgrade significantly improves the bearing strengths of clayey subgrades. At the 85th percentile, the in situ CBR of subgrades mixed with lime kiln dust, hydrated lime, and Portland cement were 24, 27, and 59, respectively. The subgrades represented in the figure ranged in age from about 8 to 15 years. The stabilized subgrades were compacted using conventional compactors. Impact compaction has the potential of increasing the CBR strength of chemically treated subgrades. Also, the untreated subgrade located below the treated subgrade would vastly improve the bearing strength of the entire media. Again, impact compactors generally disturb the top portion of a subgrade to a depth of about 4 to 8 inches and a smooth drum roller may be needed for finishing.

Full Depth Reclamation

Full depth reclamation is a pavement rehabilitation technique. The full flexible pavement section and a predetermined amount of the underlying material are uniformly crushed, pulverized, or blended, as shown in Figure 29, to establish a stabilized base course. The strength of the blended material may be improved further by adding stabilizing chemical or other additives. Chemical additives may include hydrated lime, lime kiln dust, and Portland cement. Fly ash may also be added. Another additive that has been used is asphalt.

Figure 29. Full-Depth Reclamation equipment used to crush, grind, and pulverize flexible pavement

Full Depth Reclamation may be used to depths exceeding 12 inches, but typically it is performed to depths of 6 to 9 inches. According to the literature by one manufacturer, conventional compaction equipment used for breakdown rolling range from vibratory pad-foot rollers (52,000 lbs centrifugal force) to pneumatic rollers (25 tons) relative to depth and characteristics of the pulverized layer. A pneumatic roller, or a heavy smooth drum vibratory compactor, is generally used to seat any loose aggregates. Final rolling may be performed using a 12-14 ton range single or tandem steel drum (static) roller.

Shallow depths may limit the maximum benefit that might be derived from the use of this technique and this may limit the use of this method to city streets and low volume roads. Because of the numerous clayey subgrades in Kentucky, and elsewhere, the shallow reclamation depths may pose a problem. Based on recent research in Kentucky (Hopkins 2005), a weak soft zone of soil oftentimes forms at the top of the subgrade. Frequently, in situ CBR values measured at the tops of subgrades range from about 1 to 5. Typically, the insitu CBR at the tops of clayey subgrades at the 85th percentile test value is about 1.8. Although the reclaimed material may be formed to provide a good strong base, or subbase, the improved layer may end up resting on a much weaker subgrade. Hence, there may be a need to extend the depth of reclamation to 12 inches or more to avoid the soft-zone situation and to increase the use of the technique to higher classes of roads. Compacting reclaimed materials at depths greater than 12 inches may pose a compaction problem using conventional equipment. Consequently, the use impact roller compaction equipment has the potential of solving this problem since it has been reported that depths up to 24 inches may be compacted to 90-95 percent of maximum dry density obtained from AASHTO T-180. Test trials and research needs to be performed to prove this very important point. It is envisioned that the mixing process would extend to sufficient depths (greater than 12 inches) that would include a portion of the clayey subgrade. During this mixing process chemical additives, such as hydrated lime, Portland cement, or lime kiln dust, would be added to the pulverized material during the grinding process. The impact compactor would be used to compact the material to meet standard or modified compaction specifications.

SUMMARY AND CONCLUSIONS

Impact roller compaction has been used to improve embankment and highway subgrades in South Africa, Australia, Europe, and China and other areas of the world. In September of 2003, the International Technology Scanning Program, sponsored by the Federal Highway Administration (FHWA), the American Association of State Highway and Transportation Officials (AASHTO), and the National Cooperative Highway Research Program of the Transportation Research Board, identified this technology as one of several foreign technologies and innovations that could significantly benefit U.S. transportation systems. The technology was high-lighted at the Fifth international Conference on the Bearing Capacity of Roads and Airfields in 1998 (Pinnard). To date, usage, however, of these types of non-circular compactors is at an infant stage in the United States. The capability of this type of roller to compact soils to a high percentage of maximum dry density obtained from modified compaction (AASHTO T-180) could provide many benefits as shown by this study. The stability of such roadway structures as embankments, bridge approach embankments, pavement subgrades could be vastly improved. Application of the compaction technique could potentially increase the usage of the full-depth flexible pavement reclamation. To apply this technique will require developing a compaction experience base with local Kentucky soils and rocks.

RECOMMENDATIONS

Based on the review of impact roller compaction, the following recommendations are made:

1. To maximize benefits obtained from impact roller compaction, field trials at selected individual construction sites should be conducted using different types of Kentucky soils and rocks and compacted with impact roller compactors to determine compaction efficiency and develop compaction specifications. It is essential to develop local experience with Kentucky soils and rocks in developing a new set of compaction experience and specifications.

2. Specifications pertaining to the use of impact compactors have ranged from simple to complex (Avalle, 2004). Based on one experience source, earthworks specifications may take the form of “method specifications’ or “performance specifications.” Method specifications specify the construction methods to be used while performance specifications specify that the “requirements to be met by test in the finished product.” Accordingly, various hybrid specifications have been used for impact roller projects. Each situation must be assesses to determine the most appropriate method to use. For

example, in some cases a detailed trial program (test pads for materials used on a particular project) may be performed in advance of the earthworks project to provide data for analyzing and assessing the effects of impact rolling. Consequently, both a method specification may be formulated based on the assessment and yet some testing (performance specification) may be performed to judge the final results of the impact roller. It is recommended that test pads be constructed at selected sites in Kentucky in order to build an experience base for formulating compaction specifications for various types of Kentucky soils and rocks using impact roller compactor.

3. The goal of the field trials should be to compact soil and rock layers to 95 to 100 percent of maximum dry density and ± 2 percent obtained from modified compaction (AASHTO T 180).

4. To determine lift thickness and the depth that soil can be compacted to achieve 95 percent of maximum dry density and ± 2 percent obtained from modified compaction (AASHTO T 180) of the various Kentucky soils, test pads should be constructed at selected specific sites and compacted with impact compactors. Field and laboratory testing will be required to determine those factors. The number of passes required by a selected type of impact compactor to achieve 95 percent of maximum dry density and ± 2 percent obtained from modified compaction (AASHTO T 180) must be determined at specific site. Once an experience base has been developed, test pads could eventually not be required.

5. To determine the efficiency of impact compactors to break down mixtures of hard rocks and soft clayey shales (or other types of soft rocks), test pads should be constructed at selected specific sites. The number of passes of the impact roller required to obtaining a desirable breakdown of the mixtures and depth of breakage must be determined from field trials and testing. A goal of the test pad testing is break down the mixtures to a degree to achieve 95 percent of maximum dry density and ± 2 percent obtained from modified compaction (AASHTO T 180). Once an experience base has been developed, test pads could eventually not be required.

6. In a similar manner to shale mixtures, test pads should be constructed at selected specific sites to determine the efficiency of impact compactors to break down durable or nondurable shales. The number of passes of the impact roller required to obtaining a desirable breakdown of the mixtures and depth of breakage must be determined from field trials and testing. A goal of the test pad testing is break down the mixtures to a degree to achieve 95 percent of maximum dry density and ± 2 percent obtained from modified compaction (AASHTO T 180). Once an experience base has been developed, test pads could eventually not be required.

7. When impact compactors may be used to improve compaction at bridge approach embankments, long-term settlements of the approach pavement should be monitored over a period of several years to determine if improved compaction using impact compactors mitigates creep settlement. The monitoring period of 3 to 6 years may be sufficient to make this determination provided creep settlement-logarithm of time curve is linear. In

this case, the total creep settlement may be determined by projecting the linear relationship to 10,000 days (27.4 years).

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APPENDIX

Kentucky Standard Specifications for Road and Bridge Construction, Edition of 2004 *(Sections on standard compaction of soils and rocks with suggested potential changes in the standard compaction specifications when impact roller compactors are used).*

SECTION 206 ¾ EMBANKMENT

206.01 DESCRIPTION. Form embankments with materials from sources specified in the Plans or from other approved sources.

206.02 MATERIALS AND EQUIPMENT. Use water conforming to Section 803.

206.03 CONSTRUCTION.

[1]**206.03.01 Embankment Foundations**. Remove sod from all embankment areas to a depth of approximately 3 inches. The Engineer will not require the removal of sod when constructing embankments over marshy areas. Remove unsuitable material, including frozen material, encountered in embankment areas before placing any embankment material thereon. When the height of the embankment, at subgrade elevation, is to be greater than 3 feet above existing concrete pavement, either break the pavement until no fragments have a dimension greater than 3 feet or remove the pavement. When the height of the embankment, at subgrade elevation, is to be 3 feet or less above existing concrete pavement, remove the pavement. When placing embankment above existing asphalt pavement, break up to destroy all cleavage planes or remove as the Engineer directs. Cut benches with horizontal and vertical faces into the original ground of embankment foundations as required. When practical, benches should be into rock. Compact the horizontal face. Provide subsurface drainage as specified in the Plans or as the Engineer directs.

206.03.02 Embankment. Excavate special ditches and channel changes before constructing adjacent embankment areas. Complete all embankment for any roadway, including ramps, frontage roads within the tolerances specified in Subsection 204.03.10.
Use only acceptable materials from sources permitted in the Contract. Do not place frozen material, stumps, logs, roots, sod, or other perishable materials in any embankment. Do not place any stone or masonry fragment greater than 4 inches in any dimension within one foot of the finished subgrade elevation, unless rock roadbed is specified as provided in Subsection 204.03.10. The Department may allow concrete rubble, without protruding reinforcement, to be placed in embankment provided that no fragment is larger than one foot in any dimension or is placed within 2 feet of the subgrade. When crossing marshy or otherwise unstable areas, the Department may allow the first lift to exceed one-foot loose depth. Use rock or granular material in the first lift, when available, and construct by placing material behind the leading edge of the layer and blading into place to avoid unnecessary disturbance to the original ground. Drain, clean out, and fill ponds lying within the staked construction limits. Construct the upper one foot of the embankment with selected material placed in lifts not exceeding one foot loose thickness and compacted according to Subsection 206.03.03. When rock roadbed is specified, construct the upper 2 feet of the embankment according to **Subsection 204.03.09 B).**

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[1] Consideration might be given to using impact roller compactors to compact original ground whenever the slope permits to increase densities of the foundations soils, improve bearing strengths, and decrease settlements that may occur under embankment loadings. This especially may be useful at bridge approach embankment foundations. Depth of influence of impact roller compactors is greater than convential compactors and may extend downward about 3 to 6 feet (or more). The depth of influence depends on the type of materials, moisture content, and groundwaster conditions (Avalle, 2004).

[2]**A)** Embankments of Earth, Friable Sandstone, Weathered Rock, Waste Crushed Aggregate, Bank Gravel, Creek Gravel, or Similar Materials. Construct in lifts not exceeding one foot in thickness, loose depth, to the full width of the cross section, and compact the material. Shape the upper surface of the embankment to provide complete drainage of surface water at all times. Do not form ruts.

[3]**B)** Embankments Principally of Unweathered Limestone, Durable Shale (SDI equal to or greater than 95 according to KM 64-513), or Durable Sandstone.

206—2 Construct in lifts not exceeding 3 feet. Ensure that the maximum dimensions of boulders or large rocks placed in the embankment do not exceed 3 feet vertically and 4.5 feet horizontally. Place rocks having any dimension greater than 2 feet at least 2 feet below subgrade elevation. Do not dump rock into final position. Distribute the rock to minimize voids, pockets, and bridging. The Engineer will not require rolling in the construction of rock embankment. Do not construct the rock embankment to an elevation higher than one foot below subgrade elevation.

[4]**C)** Embankment of Rock/Shale/Soil Combination. Construct in lifts not exceeding one foot in thickness; however, when the thickness of the rock exceeds one foot, the Department may allow the thickness of the embankment lifts to increase, as necessary, due to the nature of the material, up to 2 feet. Apply a sufficient amount of water to induce slaking when mixtures contain 50 percent or more non-durable shale. Do not dump the mixture into final position. Distribute the mixture in a manner that minimizes voids, pockets, and bridging.

[5]**D)** Embankments Principally of Non-Durable Shale (SDI less than 95 according to KM 64-513). Remove or break down rock fragments or limestone slabs having thickness greater than 4 inches or having any dimension greater than 1 1/2 feet before incorporating them into the lift. Construct in loose lifts not exceeding 8 inches in thickness. Apply water to accelerate slaking. Uniformly incorporate the water throughout the lift using a multiple gang disk with a minimum disk

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[2] Test pads should be built at selected sites to determine the depth of influence obtained by impact compactorstablh. The depth of influence achieved by the impact compactor most likely will be greater than 1 foot. Test pads would establish the increased depths and establish permissible loose lift thickness.

[3] The use of test pads at selected sites would aid in determining the degree of break down of these types of materials with impact compactors, whether or not the larger rocks could be broken down into much smaller pieces, and the approximate depth of influence. The goal here, it is suggested, is to break all rocks down to pieces of 6 inches, or less. The test pad experiments would examine gradation (as function of depth) of the matrix and determine the number of passes of the impact compactor as a function of test pad settlement.

[4] See Note 3. Test pad experiments may show that the lift thickness may be greater than one foot using impact compactors for these materials.

[5]Numerous types of nondurable shales are prevalent in Kentucky. Mixtures of hard rock and soft shales have posed the most difficult compaction problems in Kentucky and have led to enormous maintenance problems. Test experiments could aid in establishing the degree of break down of these materials, depth of effective breakage, and and effective densities.

diameter of 2 feet or other suitable equipment the Engineer approves. [6]Compact with 30-ton static tamping foot rollers in conjunction with vibratory tamping foot rollers that produce a minimum compactive effort of 27 tons and direct hauling equipment over the full width of the lift to aid in compaction. When questions arise regarding the durability of shale, use KM 64-514 to estimate the durability of the material in the field. When questions arise regarding the durability of shale, use KM 64-514 to estimate the durability of the material in the field.

[7]**206.03.03 Compaction.** Compact the embankment foundations and embankment to a density of at least 95 percent of maximum density as determined according to KM 64-511. The Engineer will check density according to KM 64-412. During compaction, maintain the moisture content of embankment or subgrade material within ± 2 percent of the optimum moisture content as determined according to KM 64-511. Compact each lift as required before depositing material for the next lift. Provide equipment that will satisfy the density requirements at all times. Run the hauling equipment, as much as possible, along the full width of the cross section.

206.03.04 Embankment Adjacent to Structures. Construct according to Subsection 603.03.04 for backfill.

206.03.05 Embankment-in-Place. When the Contract designates original material as unsuitable for the embankment foundation, the Department will designate areas of Special Excavation and/or treatment and will give instructions about the removal and disposal of unsuitable foundation material in the Plans. When a bid item of special excavation has not been included in the Contract and the original ground is specified in the Plans as suitable to serve as the embankment foundation but the Engineer subsequently determines the material is unsuitable to remain in its original position, excavate and dispose of the unsuitable foundation material as directed. Incorporate the excavated material into embankments when manipulations such as spreading thin layers or drying the material make it acceptable for use as embankment-in place. When excavated material cannot be used in embankments, waste the material.

206.04 MEASUREMENT. The Department will measure excavation of benches as Roadway Excavation or Embankment-in-Place, as applicable. The Department will measure the removal of unsuitable materials from embankment 206—3 areas as Roadway Excavation or Special Excavation. The Department will consider removing sod 3 inches or less in depth; removing and/or scarifying of existing pavements in embankment areas; and the addition of water to aid compaction incidental to the earthwork bid items. The Department will measure the quantity of unanticipated waste resulting from landslides or authorized slope changes in place before excavation. The Department will include the quantity of unanticipated waste under Embankment-in-Place. The Department will measure a second presplitting for payment according to Subsection 204.04.04.

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6 The impact compactor may be effective in replacing the 30-ton static tamping foot rollers. Test pad experiments may show that the vibratory roller may not be essential. In some cases, a smooth wheel vibratory roller may needed to smooth the upper surface of a subgrade.

7 It is recommended that this section read (for impact compactors): **“** Compact the embankment foundations and embankment to a density of at least 95 percent of maximum density as determined according from AASHTO T 180. The Engineer will check density according to KM 64-412. During compaction, maintain the moisture content of embankment or subgrade material within ± 2 percent of the optimum moisture content as determined according AASHTO T 180”.

206.04.01 Embankment-in-Place. The Department will measure the quantity in cubic yards as the design quantity shown within the neat lines of the cross sections on the Plans, increased or decreased by authorized adjustments according to Subsection 204.04.02.
Regardless of whether the excavated material is used as Embankment-in-Place or is wasted, the Department will measure and pay for the volume of the unsuitable foundation material that is excavated as Embankment-in-Place. When the Engineer directs that the excavated material be wasted, then the Department will measure the material used to replace the wasted material as the same as the excavated volume, and will pay for the material as Embankment-in-Place. When the excavated material is used in embankment, the Department will make no separate payment for the material necessary to replace the excavated material. For embankment material obtained outside the right-of-way limits, conform to Section 205. The Department will not measure excavation included in the original Plans that is wasted for payment and will consider it incidental to Embankment-in-Place. The Department will not measure overhaul of material for payment and will consider it incidental to Embankment-in-Place.

When payment is made for Embankment-in-Place, the Department will make payment for all embankment constructed on the project, including roadway embankment, refill in cuts, embankment placed in embankment benches, and the volume of trench above the pipe for bedding. The Department will not measure materials from authorized
Roadway and Drainage Excavation for payment and will consider them incidental to the
construction of Embankment-in-Place. The Department will include under authorized Roadway and Drainage Excavation, mainline excavation, embankment benches, special ditches, channel changes, tail ditches, surface ditches, interceptor ditches, entrances, and undercuts in rock cuts. The Department will not measure borrow excavation used to construct the embankment for payment and will consider it incidental to the construction of Embankment-in-Place. The Department may make adjustments to embankment-in-place projects when there is actually unanticipated waste on the project. Waste generated by the project phasing will not be considered for adjustment. The Department will make an adjustment for the actual costs incurred by the Contractor.

206.04.02 Special Excavation. The Department will measure the quantity in cubic yards as the design quantity shown within the neat lines of the cross sections on the Plans, increased or decreased by authorized adjustments as specified in Subsections 204.04.01 and 204.04.02. The Department will not measure overhaul of material and will consider it incidental to Special Excavation.
206.05 PAYMENT. The Department will make payment for the completed and accepted quantities under the following:
Code Pay Item Pay Unit
2230 Embankment-in-Place Cubic Yard
2204 Special Excavation Cubic Yard
2200 Roadway Excavation See Section 204.05
206—4
The Department will consider payment as full compensation for all work required
under this section.
207—1

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