Construction Methods To Control Expansive Soils in South Dakota

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A review is presented of the stabilization and construction methods used by the South Dakota Department of Transportation during the past 17 years. The nature of the soils involved is discussed and a short history of the problems encountered is given along with the techniques of grading and stabilization used to control the expansive properties of the Pierre shale. The 1967 performance of the roadway surface is compared with that in 1984. The conclusion is that rideability will be better over a longer period of time at lower maintenance costs on highways placed on expansive soils when the soils have been undercut, reworked, and replaced at controlled moisture and densities than on highways placed on these soils using normal methods. Stabilizing the upper portion of the replaced soil with lime preserves the high moisture content used in placing the soils and provides added support for the prevention of rutting by the equipment used in the placement of the surfacing courses.

Swelling soils are a construction problem in many parts of the United States as well as in many areas of the world. Nearly the whole western half of South Dakota and small portions of the eastern half are composed of highly expansive Pierre shale. The liquid limit of this soil varies from 65 to 120 with plastic indices of 40 to 85. This condition is aggravated by perched water tables and associated frost problems that exist, in some areas, because of the fractured structure of the soils.

The Missouri River divides the state of South Dakota into two distinct landforms. The western half, with the exception of the Black Hills area, is composed mainly of Pierre shale. There is a small region of sand dunes near the Nebraska border and a thin Fox Hill sandstone cover in the northwest corner. The White River formation overlies a portion in the southwest central portion of the state (Figure 1).

The Pierre shale is composed of interbedded layers of highly plastic colloidal clays and silts, with some areas containing nearly pure bentonite. These deposits vary in thickness from several feet to several hundred feet. The layers are differentiated with respect to the degree of weathering, volume change, and water susceptibility. The upper portion of the deposit is weathered to a fine homogeneous till-like clay, varying in depth from 1 to 15 ft. The next portion is weathered to an open jointed condition that admits water when the upper clay mantle is removed and is susceptible to large volume changes when subjected to alternate dry and wet cycles, such as those prevalent in western South Dakota. The expansive pressure of the shale varies from 15 to 100 psi, depending on the moisture and density at which it is placed (Figures 2 and 3).

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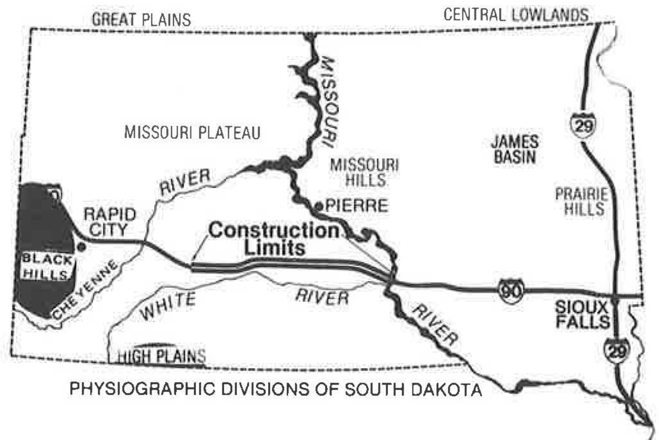

FIGURE 1 Physiographic divisions of South Dakota.

OBJECTIVE

The purpose of this paper is to briefly review the design and construction methods used by the South Dakota Division of Highways in building a 131-mi portion of I-90 through the Pierre shale and to present data on how well this length of Interstate has served in regard to rideability, deflection, and maintenance during the past 13 to 17 years.

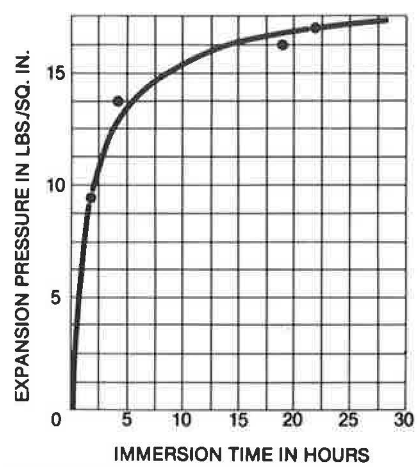

FIGURE 2 Expansion pressures of South Dakota soil: standard AASHTO compaction (T-99); moisture content, 28.9 percent.

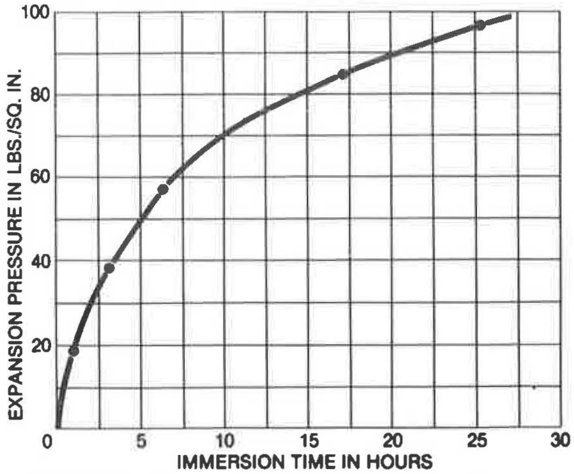

FIGURE 3 Expression pressures of South Dakota soil: modified AASHTO compaction (T-180); moisture content, 18.1 percent.

HISTORY

Before 1952 asphaltic mats of 1½ to 2-in. thickness in conjunction with 4 to 8 in. of standard base course were used as surfacing materials over the Pierre shale. Several short sections of 6 to 8 in. of concrete surfacing had been placed on the shale in and near the cities of Pierre, Winner, and Chamberlain. Both of these surfacing types distorted badly within 3 to 5 years after placement. The most dramatic failure of all of these highways was a 9-mi section east of Rapid City on I-90. It was built in 1961 with normal methods and became extremely rough in the summer of 1962. This dramatic failure provided the impetus for the Materials and Soils Section of the Division of Highways to begin a serious investigation aimed at finding methods to curtail the extreme warping of highway surfaces that occurs when they are constructed in expansive soil *(1–3)*.

A comprehensive research program involving a test project of 8.7 mi and containing several types of stabilizing agents and different types and thicknesses of surfacing materials was started in 1963. This test project was studied for a period of 4 years, concluding in 1968. A continuation study was also conducted on this project, which concluded in 1975.

Analysis of some of the early test results indicated that stabilization of the upper surface of the shale subgrade did produce, to some degree, a smoother riding surface in the same time frame as highways previously constructed without stabilization. However, because of the deep-seated nature of the moisture problems associated with the Pierre shale, a considerable amount of roughness developed in many sections of the test project.

Observations of projects built before the test project indicated that where undercutting of 6 to 12 in. was performed, an improvement of rideability was obtained. With the knowledge gained from the research project and a review of construction techniques and performance of existing highways in the Pierre shale, comprehensive laboratory tests were started in 1964. These tests included special moisture and density tests and volume change tests and freeze-thaw data on shale with and without stabilizing agents. On the basis of data obtained, a decision was made that in addition to using lime as a stabilization agent, an extreme depth of undercutting in conjunction with rigid moisture and density control would be used on the remaining portion of I-90 from Mileposts 131 to 262, which is located in the Pierre shale. The biggest factor in determining the extreme depth of undercutting was laboratory test results revealing that the swell potential of the upper mantle of Pierre shale, as well as the jointed weathered shale, could be drastically reduced when the density was held to 92 percent of AASHTO T-99 and the moisture was held at 3 percent above the optimum moisture content. The surfacing from Mileposts 131 to 198 and from 213 to 251 is composed of 8 in. of continuous reinforced concrete on a 3-in. lime-stabilized base over a 6-in. lime-stabilized subgrade. Mileposts 198 to 213 and Mileposts 251 to 262 are composed of 9 in. of asphalt concrete on a 6-in. lime-stabilized subgrade.

GRADING TECHNIQUES

Special soil inspection teams who had considerable experience in grading construction were chosen from various districts of the department. These teams were given a special course of instruction in the central laboratory that dealt specifically with construction as it related to soil selection, special moisture-density control work, soil identification, interpretation of specifications, and special provisions. These men worked directly under the resident engineer and acted in an advisory capacity to him regarding the grading operation insofar as it related to selection of soil and the identification and disposal of highly plastic soil. They were responsible for determination of undercutting that was needed in addition to the plan quantities. They were required to keep daily diaries of all phases of work within their responsibilities.

Mainline undercutting for the top 3 ft of earth subgrade was designated full roadbed width, shoulder slope to shoulder slope. The backfill material, including the upper 3 ft of the adjacent fill areas, is composed of selected subgrade topping. The undercut area below the top 3 ft to a depth of 6 ft is confined between the subgrade shoulder lines. This lower 3 ft forms a trench 52 ft wide, shoulder to shoulder (Figure 4).

In order to meet the density limits of the specifications, the energy-input method was used to control the work. It was anticipated that four uniform coverages of a sheepsfoot roller over the disked surface would meet the stipulated density requirements. Frequent moisture and pulverization tests were taken to ensure that proper moisture was adequately dispersed to obtain the required density.

Review of Special Provisions

1. Selected subgrade topping for the upper 3 ft of all roadway backfill and embankments was sufficiently processed to readily receive water with 90 percent (exclusive of rock) passing the 1-in. sieve in accordance with Test SD 212.

2. Sources of selected topping were confined to horizons and areas designated on the plans or by the soil crews and engineer. The upper 6 in. of sod was excluded.

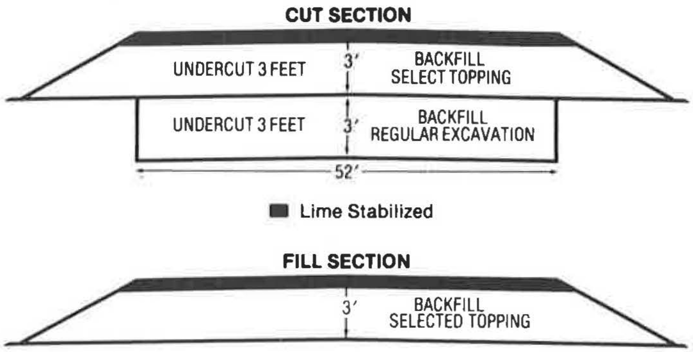

FIGURE 4 Typical grading sections: South Dakota I-90, Mileposts 131 to 262.

3. The 6-ft depth of undercutting was reduced to 3 ft or increased as necessary by the engineer.

4. Undercut or other suitable material that did not contain bentonite was used to construct the undercut area below the 3-ft selected topping zone.

Selected topping was measured and paid for on plans showing embankment volume and shrinkage. Payment was made for additional work, selection, blending, extra haul, and interim stockpiling.

Review of Requirements for Upper 6 ft of Embankment

1. The selected topping was placed in loose lifts generally not exceeding 8 in. and pulverized by disking to the satisfaction of the engineer, as determined by Test SD 113.

2. Compaction was obtained by use of controlled passes of a sheepsfoot roller. Four coverages of the sheepsfoot were normally used, with a minimum of not less than two coverages.

3. Density tests were made to guide the energy-input method; the following tests were required:

- Density minimum, 92 percent AASHTO T-99;
- Density target, 95 percent AASHTO T-99; and
- Moisture target, 3 percent above optimum,

Requirements for Embankments Below 6-ft Zone

1. Loose lifts did not exceed 12 in.

2. Each lift was leveled and pulverized by disking and properly watered and mixed.

3. Density tests to guide the energy-input method were as follows:

- Density minimum, 95 percent AASHTO T-99;
- Berm adjacent to structures, 100 percent AASHTO T-99; and
- Moisture target, 3 percent above optimum.

Review of Original Laboratory Test Data

A series of density tests was run to determine what effect higher compactive efforts would have on the shale in regard to cubic weight and moisture and what effect they would have on shale that had been stabilized with lime. The T-180 method produced a density of 96.2 pcf at an optimum of 23.5 percent for the raw shale. A density of 91 pcf at an optimum of 26.5 percent was obtained for the lime-stabilized soil. The T-99 method produced densities of 77.2 pcf at an optimum moisture of 33.6 percent for the raw soil and 70.0 pcf at an optimum of 35 percent for the lime-stabilized shale (Figure 5).

To find out how different degrees of density and moisture affected swell potential, a series of swell tests was performed on soils taken from the upper mantle of Pierre shale and from the underlying jointed shale. It was found that when this soil was subjected to lower water content and higher density, the swell potential was two to three times higher than when the soil was subjected to lower density and higher water content. The same series of tests applied to the jointed underlying shale showed that the swell potential was two to four times higher for the high-density–low-moisture condition than for the lower

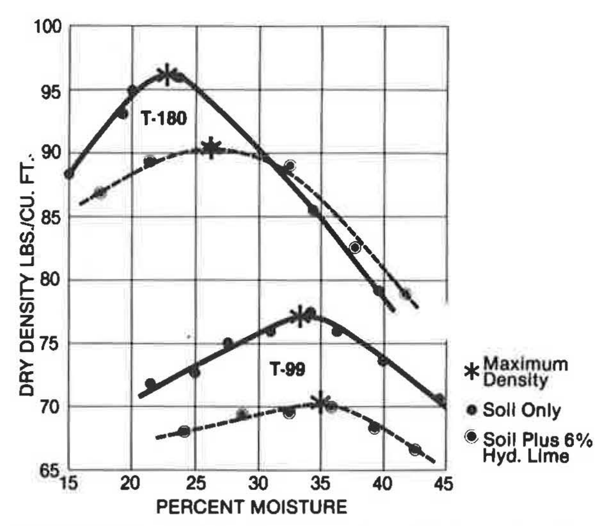

FIGURE 5 Effect of compactive effort on density of South Dakota shale.

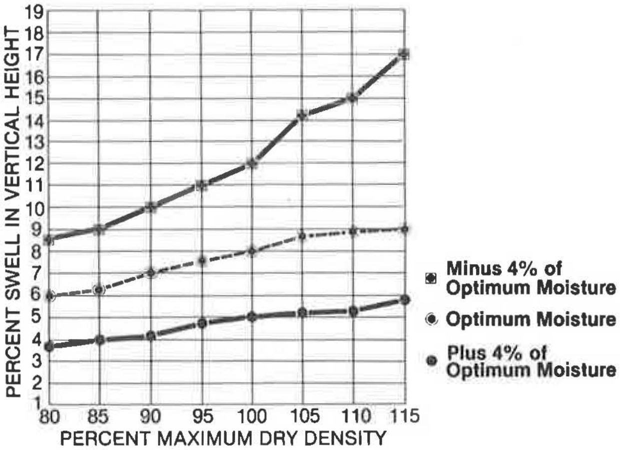

FIGURE 6 Special swell tests on weathered shale.

density and higher moisture (Figures 6 and 7). As noted earlier, it was on the basis of these tests that the decision was made to undercut the parent shale and replace it with selected topping from the upper mantle of Pierre shale.

EARLY PERFORMANCE

A review of the follow-up field tests, in which 21,851 moisture-density tests were taken, shows that the material was placed

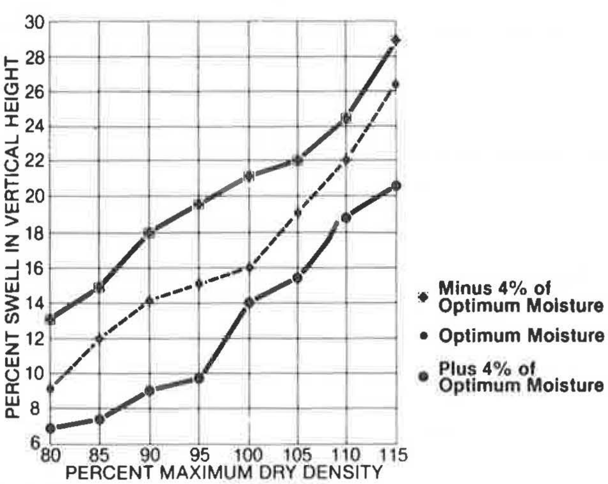

FIGURE 7 Special swell tests on jointed shale.

relatively close to the target density and moisture contents. The lower 6-ft zone of all fills was placed at 98 percent of T-99 at 2 percent above optimum moisture. The upper 6-ft zone of all undercut sections and the upper 3-ft zone of all fill sections were placed at 98 percent of T-99 at 2.1 percent above the optimum moisture content (Figure 8).

The South Dakota Division of Highways has run a comprehensive roughness and deflection program on its highways since 1970. The roughometer data are based on an index scale of 0 to 5; 5 is very good and anything below 2.5 is poor. Deflection testing is based on millimeters of deflection on a scale of 1 to 5; 1 is very good and anything over 3 is very poor.

• Total Miles Constructed—130.323
• Number of Tests Taken—21,851
• Average Density in Upper 6′ Zone—98% of T-99
• Average Moisture in Upper 6′ Zone— + 2.1% of Opt.
• Average Density in Lower 6′ Zone—98% of T-99
• Average Moisture in Lower 6′ Zone— + 2.0% of Opt.
• Number of Cubic Yards of Excavation—53,991,610
• One Test Taken for Approx. Each 2,500 Cu.Yds. of Material

FIGURE 8 Moisture-density construction data, South Dakota I-90, Mileposts 131 to 262.

The roughness index in 1975 for the asphalt surfacing was 4.22. Deep-strength asphalt has a tendency to crack at fairly close intervals, which results in faulting. Some roughness due to faulting rather than subgrade differentials resulted, which reduced the index to 3.80 by 1980. The joints were milled off in 1980 and the material was relaid, resulting in an index of 4.31 in 1984. A total of 26 mi of surfacing is composed of asphalt on this portion of the Interstate. The remaining 105 mi is composed of concrete. The roughness index for the concrete was 4.35 in 1975. In 1984 it was 4.09. It is evident that the differentials associated with surfacings built in the Pierre shale have been controlled quite well, because much of this mileage was built in 1967, 1968, and 1969 (Figure 9).

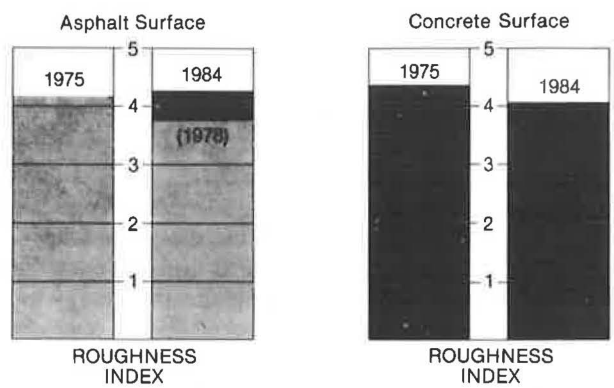

FIGURE 9 Roughometer data, South Dakota I-90, Mileposts 131 to 262.

A review of the deflection data shows that both the asphalt and concrete have slightly less deflection in 1984 than in 1975. It is possible that the overall subgrade condition had a higher moisture content in 1975 or that the lime-stabilized base or subgrade or both have higher strengths now than in 1975 (Figure 10).

The cost factor at the time that this section of highway was built is not really significant because inflation did not really begin until 1973. However, it is reasonable to believe that, all things being equal in regard to grading operations, the added cost of undercutting and lime stabilization would be in the same relative degree to the regular grading costs now as they were at the time that this grade was constructed.

A review of the surfacing, grading, undercutting, and lime

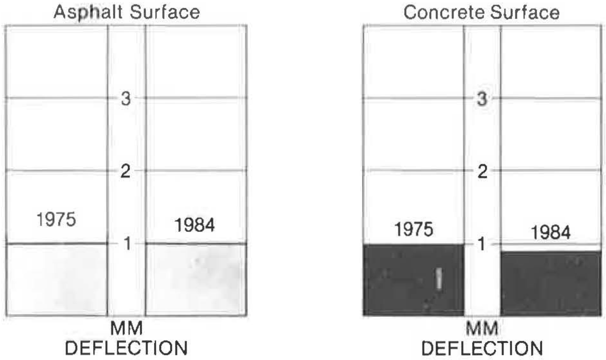

FIGURE 10 Deflection data, South Dakota I-90, Mileposts 131 to 262.

stabilization costs in Figure 11 shows that the asphalt surfacing cost about 35 percent less than the concrete, but the inflation in asphalt costs has now made these two types of surfacing about equal in cost. Grading costs in South Dakota have always been quite low in comparison with those in other areas of the country. it can be seen that the additional cost of undercutting

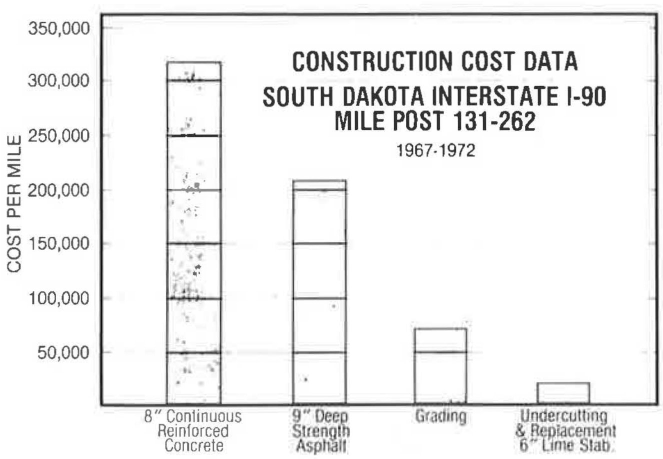

FIGURE 11 Construction cost data, South Dakota I-90, Mileposts 131 to 262, 1967–1972.

and lime stabilization increased the grading costs by about 38 percent. However, the increase was only about 5 percent of the overall surfacing and grading cost for the concrete sections and about a 9 percent increase in the overall cost for the asphalt sections.

CURRENT PERFORMANCE

Reference to the current performance in regard to rideability and deflection in 1984 is shown in Figures 9 and 10. In Figure 12 maintenance costs are compared for both asphalt and concrete surfaces that were placed on the selected lime-stabilized areas. These costs are compared with those for other similarly constructed surfaces on areas where normal grading techniques were used. It is shown that on asphalt surfaces the maintenance costs per year are approximately four times less where undercutting, lime stabilization, and moisture-density control were

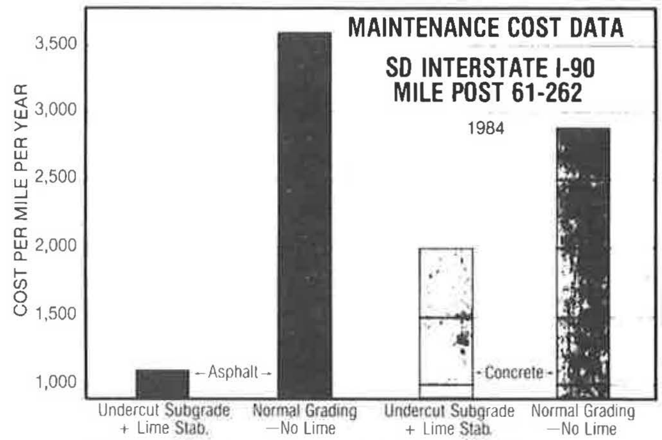

FIGURE 12 Maintenance cost data, South Dakota I-90, Mileposts 61 to 262, 1984.

used compared with costs on asphalt surfaces where these methods were not used. The maintenance costs for concrete surfaces for these same undercut and stabilized conditions are about 30 percent less than for the concrete placed on normal graded subgrades. This is a significant savings considering that most of the mileage of the highways compared is 15 years old and older.

CONCLUSIONS

The following conclusions are based on the review of the data presented.

1. Much of the objectionable differentials in the Pierre shale that have caused considerable road surface roughness in the past have been alleviated.
2. Trained crews can control undercutting and special moisture-density requirements needed to reduce the expansion of heavy plastic soils.
3. Special undercutting with special moisture-density control and replacement of soils can reduce the effects of swelling soils on highway rideability.
4. The long-term maintenance costs are less for surfacings placed on subgrades that have been undercut, reworked, or replaced at special moisture-density requirements than for surfaces placed on normal grades.

The methods discussed in this paper will not be 100 percent effective in correcting all of the differentials associated with expansive soils when highways are built in these areas. However, data given in this paper show conclusively that rideability will be better over a longer period of time at lower maintenance costs on highways placed on expansive subgrades that have been undercut and replaced using special moisture and density control methods and stabilized with lime than on highways where these procedures have not been used.

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Genesis and Distribution of Colluvium in Buffalo Creek Area, Marion County, West Virginia

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Two types of colluvium are present on the Buffalo Creek landscape, an area typical of that part of the Appalachian Plateaus underlain by rocks of the Dunkard Group. Thick deposits of entrenched, diamicton, debris-flow-generated colluvium occur as long fingers in coves (zero-order drainage basins). These deposits range up to 15 m thick but most are 5 to 8 m thick. The deposits are the product of early Wisconsinan slope processes that produced colluvium at a greater rate than it could be removed by streams. Colluvial fingers are currently well drained because they are deeply dissected by gullies; natural slopes on colluvial fingers are relatively stable. The second type of colluvium is generated as modern slope failures shear bedrock and transport material downslope. Colluvium generated by this process collects up to about 2 m thickness on bedrock benches until failure conditions are reached and the material is transferred to the next lower bedrock-controlled bench. Colluvium converges on hummocky areas at heads of gullies and is eventually delivered to streams by debris flow or fluvial erosion of failure toes. Different types of modern natural slope failures occur on different surficial geologic units, indicating that surficial geologic setting has a strong influence on failure mechanisms and relative stability of natural slopes.

The Buffalo Creek area in Marion County, West Virginia (Figure 1), is typical of much of the Appalachian Plateaus underlain by rocks of the Pennsylvanian-Permian Dunkard Group. In the Buffalo Creek area there are two distinct types of colluvium: *(a)* thick, cove-filling diamicton deposits that are relicts of the influences of a Pleistocene climate and *(b)* thin colluvium that is being generated and transported under present-day conditions. ("Diamicton" is used here to denote deposits with bimodal particle size distributions, in particular those with cobble- to gravel-size clasts mixed with a silty and clayey matrix.) The origin and occurrence of these two types of colluvial deposits and their influence on relative slope stability are discussed.

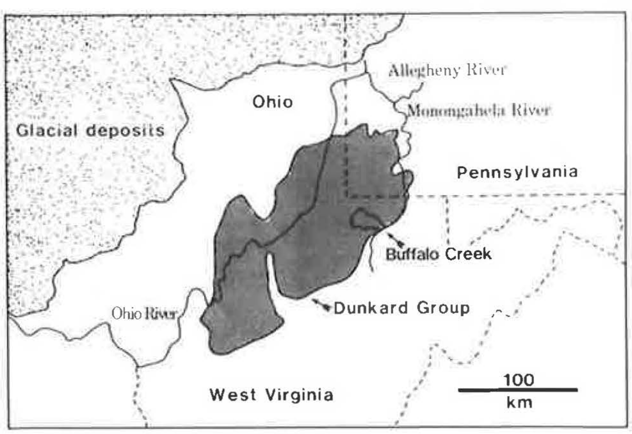

FIGURE 1 Upper Buffalo Creek study area showing outcrop area of Pennsylvania-Permian Dunkard Group and Pleistocene glacial border.

GEOLOGY OF BUFFALO CREEK AREA

The Buffalo Creek area is underlain by gently deformed sandstones, mudstones, and shales of the Pennsylvanian-Permian

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