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**Mr. Raad was at the Department of Civil Engineering, University of California, Berkeley, when this research was performed.*

Stabilization of Expansive Shale Clay by Moisture-Density Control

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Stabilization of expansive Pierre shale has been a continuing problem in South Dakota for many years. A procedure has been developed to produce a roadbed with very few differentials that uses special undercutting to a depth of 1.83 m (6 ft), replacement of subgrade by selected materials, rigid control of moisture to achieve low density at a high moisture content, and lime treatment in the upper 15.2 cm (6 in) of subgrade. A density of 92 percent of the maximum American Association of State Highway and Transportation Officials T 99 test value and a moisture content of 3 percent above the optimum were set as targets. The high degree of stability of roads constructed by this procedure is shown by the very good roughness-index ratings. The average roughness index, based on a 0 to 5 rating system, is 4.34 for the full length of 209.2 km (130 miles) of surfacing. The projects observed and tested for this study have been in service from 5 to 8 years.

A good road is one that is stable from the subgrade depth to the top of the surfacing. To achieve this type of stability, it is necessary to design each component so as to use the best materials and techniques available. Stability, especially in the subgrade, is dependent on the environment in which the material is located. All of the components of the road structure must be able to resist the deteriorating effects of climatic cycles and traffic loads. A road structure is usually divided into four component parts—subgrade, subbase, base, and bituminous mat or wearing course. In a rigid design, it is composed of a concrete surfacing and a base. The subgrade is the native material below the imported materials.

Subgrade stabilization is usually carried out by the incorporation of some type of stabilizing agent into the soil. However, there are other things that can be done to improve the natural stability of fills and the upper portions of cut sections before incorporating stabilizing additives into the subgrade.

Subgrade soil is by nature differentiated, especially in the upper A horizon. It is highly responsive to moisture and thermal changes. The B horizon is more pervious to water, which moves within this zone in channels of least resistance or in a differential manner, resulting in variable support values. The C horizon is even less permeable than the B horizon and reacts differently than do the two upper horizons. Obviously, for the road surfacing to remain stable, differentials in the subgrade support must be minimized as much as possible. This stabilization can be accomplished by blending and mixing the various soil horizons into a more uniform mass.

This report deals primarily with experience in South Dakota in the design and construction of 209 km (130 miles) of I-90 on highly expansive Pierre shale. However, although these methods of stabilization and construction procedures were applied to the problems of Pierre shale, they could also be applied successfully to other types of expansive soils in other parts of the country. The paper also presents the results of postconstruction testing and the ridability of the surfacing, as of June 1975.

HISTORICAL BACKGROUND

The Materials and Soils Program of the South Dakota Division of Highways has long recognized the problems connected with the design of road surfaces on the highly expansive Pierre shale. Over many years, changes have been made in the methods of design and the construction procedures in an attempt to reduce the expansion of the soils and its resultant effects.

The Missouri River divides South Dakota almost equally into two distinct land forms. The soils of each half have quite different physical characteristics, such as liquid limits, plasticity indexes, volume changes, and bearing values. The upper mantle of soil in the eastern half of the state is composed chiefly of glacial till, silt clays, clay silts, and sand silts. The western half of the state, except in the Black Hills area, is composed of Pierre shale and weathered Pierre shale clay, commonly referred to as gumbo. A small area near the south-central Nebraska border is predominantly sand, and some small areas on the northwest corner have thin sandstone overlays. The mineral composition of the bentonitic shale is largely montmorillonite. The physiographic features associated with this geology are shown in Figure 1.

Pierre shale was laid down west of the Missouri River in South Dakota in the Mesozoic Era (1). The shale structures are the result of several sea intrusions over a period of millions of years and are composed of layers of interbedded, highly plastic, colloidal clays and silts, with some areas containing layers of nearly pure bentonite. This deposition varies in thickness from a meter or two to a hundred or more. The layers are differentiated with respect to degree of weathering, volume changes, and water susceptibility. The upper portion of the deposition is weathered to a fine, homogeneous till-like clay from 0.305 to 4.6 m (1 to 15 ft) in thickness. The next portion is weathered to an open-jointed condition that readily admits surface water when the upper soil and clay till are removed. The inorganic colloidal clays and the bentonite are susceptible to large volume changes when they are subjected to prolonged alternate dry and wet cycles, such as are prevalent in western South Dakota. The expansive pressure of the clay derived from Pierre shale in some areas is nearly 689 kPa (100 lbf/in^2) (Figures 2 and 3).

A short history of the design and performance of highways built in these expansive soils before the use of deep undercutting and moisture and compaction control is given to enable the reader to better understand the reasons for using drastic undercutting methods. Before 1952, asphaltic mats, 3.8 to 5.1 cm (1.5 to 2 in) thick, and base gravel, 0.10 to 0.15 m (4 to 6 in) thick, were used as a surfacing over the Pierre shale. Several short sections of plain 0.15 and 0.20-m (6 and 8-in) thick portland-cement-concrete pavements were placed on the shale near Pierre, Winner, and Chamberlain. Both of these types of surfacing distorted badly in 3 to 5 years. This distortion was not due to the traffic, but to the expansive soil that warped the surfaces to such a degree that traffic soon finished the breakup.

After observing the poor results of these thinner designs, thicker pavement surfaces were constructed by using 7.6 to 10.2-cm (3 to 4-in) mats over 0.13 to 0.20 m (5 to 8 in) of base course and 0.15 to 0.38 m (6 to 15 in) of subbase. At the same time, procedures were implemented to control the subgrade moisture, undergrade cut sections up to 0.30 m (12 in), scarify an additional 0.15 m (6 in) of soil, and recompact the scarified and undercut soil. The undercut soil was replaced with selected soil where possible. In more recent years, cut sections have been undercut to the toe of the in-slopes. The undercut soil is either returned to the cut and recompacted or replaced with selected soil.

Observations of the roads built by using these methods indicated a certain degree of success in reducing the severity of warping in areas containing highly expansive soil. The majority of these roads have an adequate riding surface, although there is still a certain amount of undesirable roughness. This roughness is found in areas where there is as much as 0.76 m (30 in) of subbase material under the base and mat. The same conditions have also developed where extremely thick bases were used under concrete.

PRELIMINARY FIELD STUDIES AND LABORATORY TESTS

The preliminary soil investigations by the highway division consisted of the development of a continuous soil profile from field measurements of the types of soil along each project. Tests were made on samples of the various soils encountered, and a careful performance study was made of an existing highway, US-16, that parallels the entire length of I-90 (Figure 1). Areas requiring considerable maintenance because of differential swell, usually in cut sections, were investigated by digging a series of closely spaced holes and determining the position and extent of the in-place soil types and the nature of each layer. In-place moisture and density tests of the upper portion of the subgrade soil, which was covered by asphalt concrete and the soil-aggregate base course, were also taken from the existing road. Large differentials in liquid limits and volume changes over very short distances were found not only in a longitudinal direction for 1.5 to 4.6 m (5 to 15 ft), but also in very thin vertical layers of 0.15 to 0.46 m (6 to 18 in). The quantity of available topsoil [usually the top 0.15 m (6 in) of sodded areas] and the availability of weathered soil suitable for topping the upper 0.91 m (3 ft) of the entire subgrade were estimated. This material is usually found in the upper 0.31 to 1.5 m (1 to 5 ft) of the natural terrain.

The degree of variations and the high incidence of change in the physical properties of the interbedded soils required a method by which soils from one zone could be

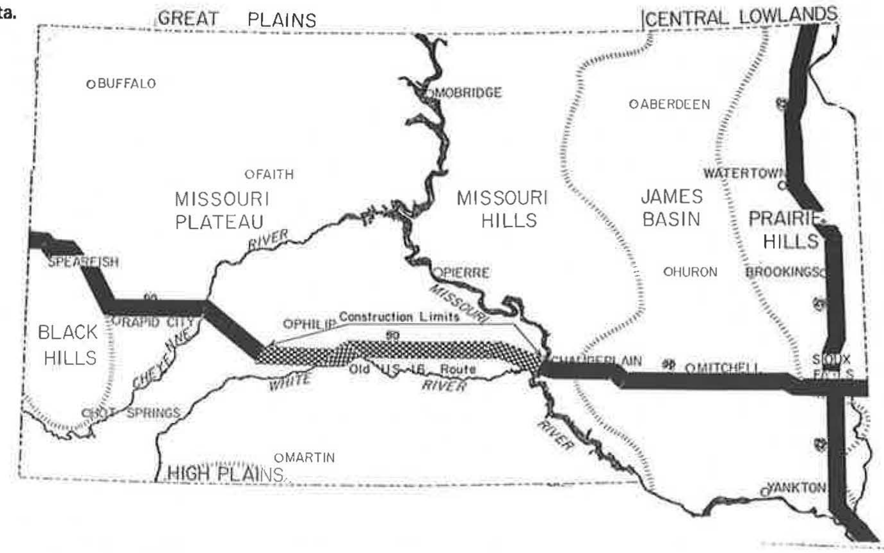


Figure 1. Physiographic divisions of South Dakota.

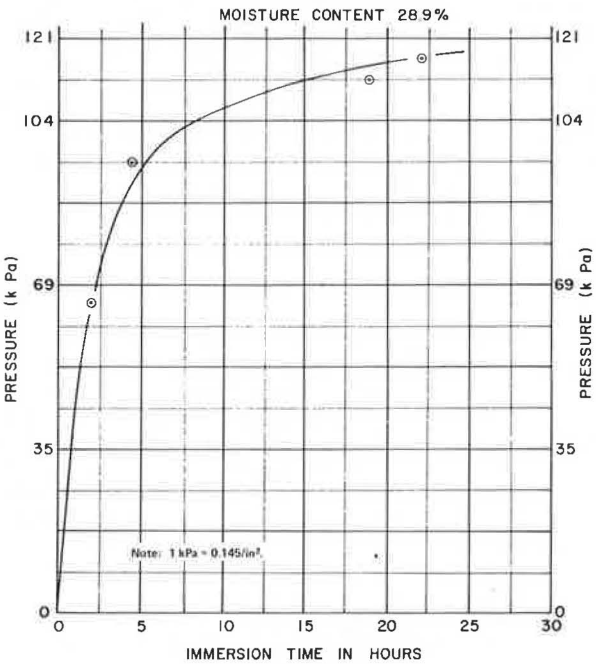


Figure 2. Expansion pressures of South Dakota soil (standard AASHTO compaction T-99).

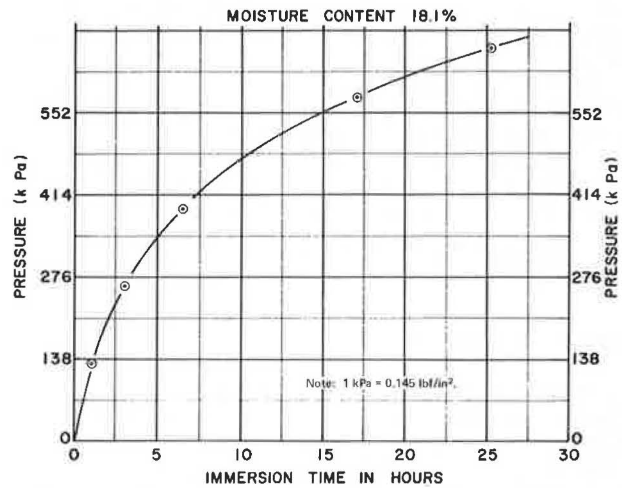


Figure 3. Expansion pressures of South Dakota soil (modified AASHTO compaction T-180).

salvaged or stockpiled for use in the upper portion of the roadbed. In certain areas, the soils in the upper 1.2 or 1.5 m (4 or 5 ft), exclusive of sod, were naturally fairly well blended. A series of moisture, density, and swell tests were run to determine the moisture and density combination that would produce the least amount of swell and still allow construction traffic. The weathered shale in the upper layer of soil had a lower swell potential than did the lower layer of jointed shale. The least amount of swell occurs when the weathered soil is compacted at approximately 3 percent above optimum, at 92 percent of the AASHTO T 99 test. The test data for the two samples described below and given in Figures 4 and 5 show that the swell potential of the weathered shale is less than half that of the jointed shale (1 kg/m^3 = 0.062 lb/ft^3). These tests were one of the important factors in the decision to construct rigid as well as flexible pavements on the Pierre shale.

Property	Weathered	Jointed
Location	Fort Pierre area	Fort Pierre area
Optimum moisture, %	23.5	30.9
Max density, kg/m^3	1538	1309
Liquid limit, %	49.8	111.0
Plastic index, %	27.4	80.3

PRECONSTRUCTION PLANNING

Before the grading plans were prepared, there was considerable discussion about the types of pavement that could economically be placed and maintained over the expansive soils west of the Missouri River in South Dakota. The final decision was to use a 0.21-m (8-in) thickness of continuously reinforced concrete pavement on 15.2 cm (6 in) of lime-stabilized base course on the 209-km (130-mile) section between Cactus Flat and Chamberlain, except for an 18-km (11-mile) section westward from the Missouri River Bridge at Chamberlain and a 24-km (15-mile) section westward from Vivian. These sections of roadway are constructed of deep-strength asphalt because it was believed that the maintenance problems would be less in these highly bentonitic-soil areas in the event that the deep undercutting and moisture and density controls were not adequate to contain the heaving.

Before the specifications for these projects were written, it was recommended that, in addition to the normal undercutting procedures, there be an additional 0.61 (2 ft) of undercutting and that the placement of soil use the energy-input method, which controls the number of roller passes per layer of soil placed. Special field-control crews to maintain strict supervision of the selected-soil placement were also recommended.

Main-line undercutting for the top 0.91 m (3 ft) of earth subgrade was designated for the full roadbed width, shoulder slope to shoulder slope. The backfill material for the top 0.91 m (3 ft), including the adjacent fill areas, is composed of selected subgrade topping. The undercut area below the top 0.91 m (3 ft), to a depth of 1.83 m (6 ft), is confined to the area between the subgrade shoulder lines. This lower 0.91 m (3 ft) forms a trench 15.9 m (52 ft) wide, shoulder to shoulder, and 0.91 m (3 ft) deep (Figure 6).

Four soil inspectors who had experience in grade construction were chosen from various districts of the department. These inspectors were given a course of instruction in the central laboratory dealing specifically with construction items as they related to soil selection, moisture and density control, soil identification, special tests, and interpretation of the specifications and special provisions.

These men worked directly under the resident engineer and acted in an advisory capacity about the grading operations as they related to the selection of the topping soil and the identification and disposal of highly plastic soils. They were responsible for the determination of the undercutting that was needed in addition to that shown on the plan and the deletion of undercut where it was deemed unnecessary. It was their responsibility to document all such changes.

SUBGRADE CONSTRUCTION

The specifications required that the upper 0.91 (3 ft) of subgrade, in both cuts and fills, be constructed of weathered soil selected for that purpose. The lower 0.91 to 1.83-m (3 to 6-ft) zone of the entire subgrade was constructed of normal soil, using higher moistures and lower minimum densities than for the underlaying embankment. One inspector was assigned to each working area of a project. There was usually more than one contractor on a project, and individual inspectors were always assigned to each contractor.

The inspectors kept daily diaries and recorded all of the activities connected with the grading work. The specifications required that the soil be broken down or pulverized so that approximately 50 percent would pass a 6.35-cm (2.5-in) sieve to achieve an adequate and uniform moisture content and to prepare each individual lift for compaction. The specifications also required that a minimum of one density test be taken for each 0.8 km (0.5 mile) of road per lane for each zone or layer.

The inspectors were directly in charge of these operations and enforced the requirements. Many liquid-limit tests were run to determine the areas of least expansive soil for use as the selected subgrade topping and for backfilling. The inspectors worked several kilometers ahead of the grading operations and supervised the location of topping material and determined the quantity available. They also supervised the placement of the selected soil in the upper 0.91 m (3 ft) of the subgrade, both in cuts and fills. In areas where extremely high bentonitic materials were encountered, the soil inspectors identified the unsuitable soil, recommended its removal, and supervised additional undercutting and backfilling. Although it was preferred to meet the target value limits, test values were acceptable if within the minimum specification requirements.

To meet the density limits of the specifications, the energy-input method was used to control the work. It was expected that four uniform coverages of the roller would be needed to meet the stipulated density requirements. However, a minimum of two coverages were required, with additional coverages as determined by the engineer and contractor, based on the results of random density tests. Frequent moisture and pulverization tests were taken to ensure that the moisture was adequately dispersed (4).

STUDIES

Costs

On the average, a total of 198 000 m^3 (259 000 yd^3) of excavation and selected-soil topping were handled for each kilometer of grade built at a cost of approximately \$123 000/km (\$197 000/mile). The total cost of grading the 209 km (130 miles) of projects was \$25 716 306.

These costs were taken from the abstract bid items. The undercutting and borrow items were included in the unclassified excavation. The majority of the projects listed in this report were graded during 1967 to 1970. The bid prices on unclassified excavation, at that time, ranged from \$0.21 to 0.28/m^3 (\$0.16 to 0.21/yd^3). The selected-soil topping was bid as low as \$0.145/m^3 (\$0.11/yd^3). Figure 7 shows the surfacing costs for 13 continuous-reinforced-concrete surfacing projects and for 3 full-depth asphalt-concrete surfacing projects. These costs include shaping, the lime treatment of the base, and materials for shoulders.

Test Data

The average moisture and density tests indicate that the moistures and densities were reasonably close to the goals set. However, an in-depth investigation of the subgrade on one project showed that the results were not quite as close as the construction tests indicated. A review of 89 tests taken from the upper 1.83-m (6-ft) zone of this project shows an average density of 99.5 percent of the AASHTO T 99 test and a moisture content of about 2 percent above optimum. There is no doubt that the moisture content results are due to the loss of a certain amount of moisture during the compaction process and that the increase in density was due to construction equipment. A total of 41 281 985 m^3 (53 991 610 yds^3) of soil was processed and compacted on these projects. Adherence to the specifications effectively produces, as nearly as possible, a uniformly blended subgrade with a uniform moisture content. A resumé of the moisture and density tests and the volume of soil excavated is given in Table 1.

Ridability

Roughness-index measurements have been made on these projects since they were constructed. These data are shown in Figure 8 and indicate that the special moisture and density controls used appear to have retarded the adverse effects of the expansive soil through 1975. The earliest project constructed, which is now 8 years old, is located from Cactus Flat eastward. The most recently constructed project, which is adjacent to the Missouri River, was completed in 1973.

At present, some sharp bumps have appeared in the pavement because of fault lines in the shale. These occurred where it was not feasible to cut enough to eliminate the fault blocks. The bumps are especially noticeable just west of the Missouri River and in areas near Kadoka, Stamford, and Murdo.

The average roughness index for the full length of the 16 projects tested is 4.34. This is a very good rating, considering that several of the projects are 8 years old. The project built in the Missouri River trench has the poorest index rating (4.15). It is also one of the newer projects, having been built in 1972.

In general, the ridability is extremely good when it is considered that in the past it was not possible to maintain

Figure 4. Special swell tests on weathered shale.

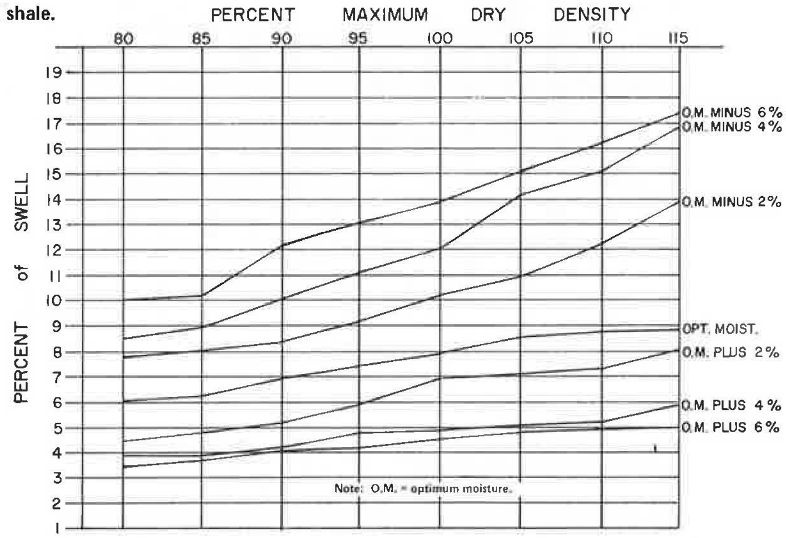


Figure 5. Special swell tests on jointed shale.

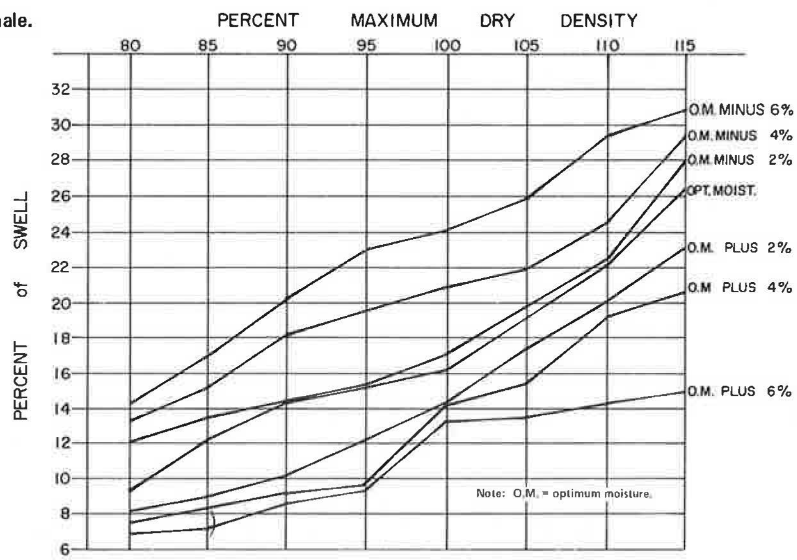
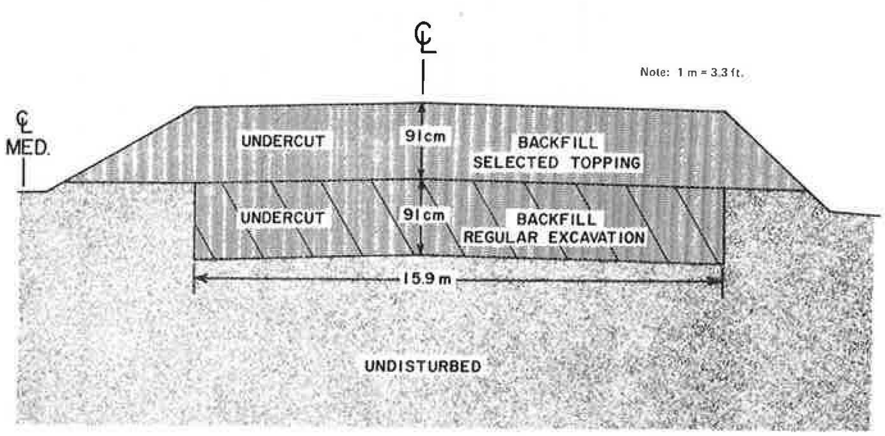


Figure 6. Typical grading section.



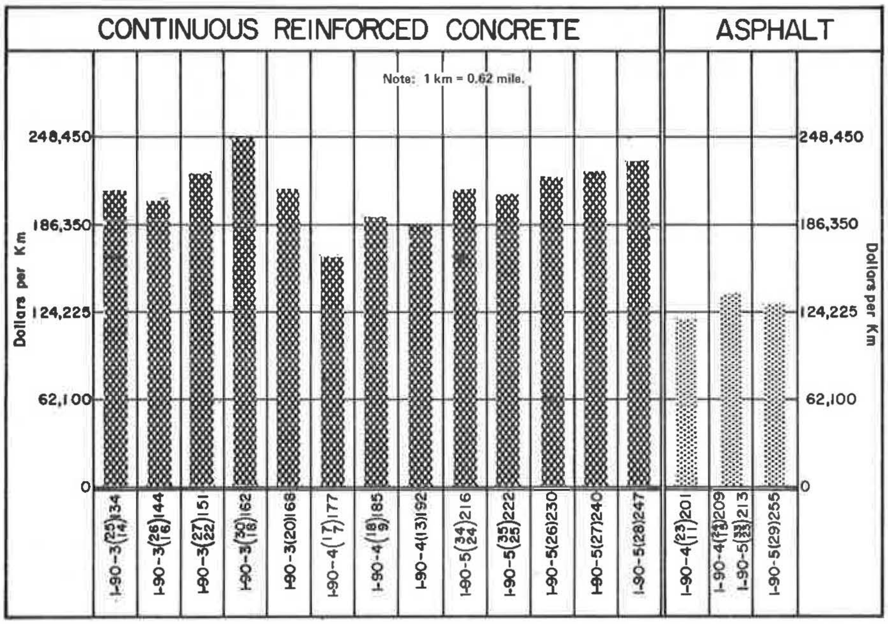


Figure 7. Cost study.

Table 1. Construction test data.

Project	County	Length (km)	No. of Tests	Percentage of T-99 Test Density: Upper Half of Zone	Percentage of T-99 Test Density: Lower Half of Zone	Percentage Moisture Above Optimum: Upper Half of Zone	Percentage Moisture Above Optimum: Lower Half of Zone	Volume Excavated (m^3)
I-90-3(14)134	Jackson	16.0	2 750	96	97	2.0	1.8	2 218 378
I-90-3(16)144	Jackson	11.8	1 337	96	98	2.0	1.5	1 368 539
I-90-3(22)151	Jackson	15.8	1 338	97	95	1.4	1.8	2 426 788
I-90-3(18)162	Jackson	10.3	946	98	98	1.5	1.0	1 407 278
I-90-3(20)168	Jackson	14.2	494	100	100	0.6	0.7	2 827 064
I-90-4(7)177	Jones	12.9	972	99	98	2.2	1.8	2 330 844
I-90-4(9)185	Jones	11.4	968	99	97	2.0	1.7	2 689 159
I-90-4(13)192	Jones	13.7	1 706	99	98	2.5	2.5	3 045 705
I-90-4(11)201	Jones	12.8	1 032	97	99	2.7	2.8	2 449 058
I-90-4(19)209	Jones	6.5	877	101	100	2.9	2.8	2 005 524
I-90-5(23)213	Lyman	4.8	651	101	100	2.9	2.8	1 487 406
I-90-5(24)216	Lyman	9.9	1 268	100	98	3.5	2.8	2 573 478
I-90-5(25)222	Lyman	11.9	877	98	98	2.3	2.4	2 487 287
I-90-5(26)230	Lyman	15.5	1 068	97	100	3.2	2.8	2 711 396
I-90-5(27)240	Lyman	11.7	699	97	99	2.6	2.9	1 988 870
I-90-5(28)247	Lyman	12.9	770	97	99	1.5	1.7	2 274 840
I-90-5(29)255	Lyman	17.4	2 600	99	98	0.0	1.0	5 011 967
Total		209.5	21 851	–	–	–	–	41 303 581
Avg		–	–	98	98	2.1	2.0	–

Note: 1 km = 0.62 mile; 1 m^3 = 35.3 ft^3.

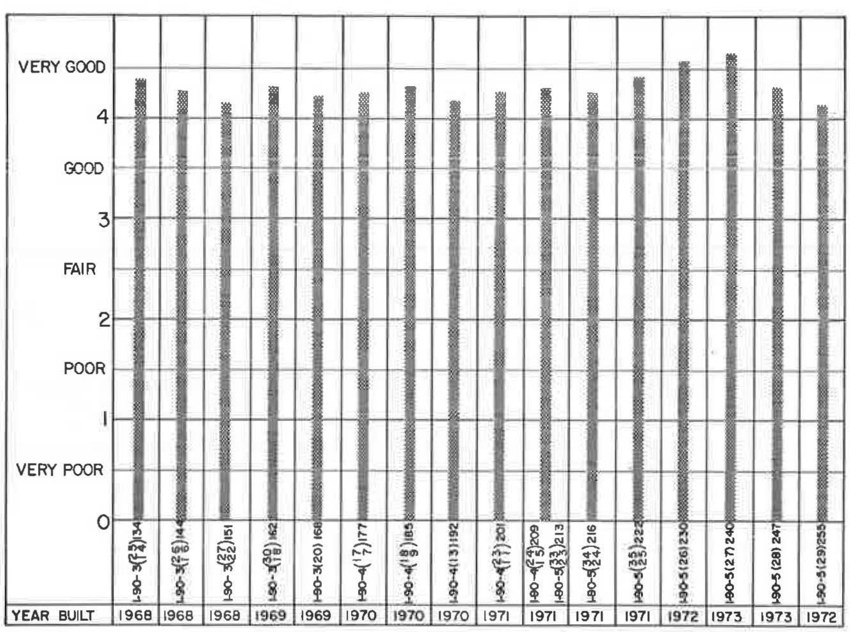


Figure 8. Roughometer analysis (1975).

anything approaching this rating beyond 3 or 4 years.

CONCLUSIONS

It appears that much of the objectionable differentials in the Pierre shale that caused road-surface roughness in the past have been alleviated.

1. It is possible to achieve relatively good density and moisture control by designating the number of roller passes and making frequent moisture tests.
2. The control of special undercutting and soil selection can be managed by the use of specially trained crews assigned to the work.
3. Extreme warping and heaving of the subgrade over narrow fault lines appears to be reduced by deep undercutting of expansive shale and replacement of it with soil having a lower liquid limit.
4. At this time, the ridability of the surfacing is extremely good and better than on similar highways constructed through Pierre shale without deep undercutting and rigid control of compaction efforts and moisture application.
5. The use of lime treatment in the upper 0.15 m (6 in) of the subgrade and a lime-treated base is effective in preserving the moisture content of the subgrade over an extended period of time, which contributes to improved ridability.
6. At the present time, the maintenance costs for the highways listed in this report are very low; however, long-range maintenance costs are time dependent.

Although the methods discussed in this report will not be completely effective in correcting the differentials usually encountered in Pierre shale, the indications, after 5 to 8 years, are that the ridability will generally be much better over a longer period of time at lower maintenance costs.

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Factors Affecting Unconfined Compressive Strength of Salt-Lime-Treated Clay

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Statistical procedures were used to analyze the effects of salt content, lime content, curing time, curing temperature, molding-water content, and soil type on the unconfined compressive strength of compacted specimens. A modified central-composite, second-order, rotatable experiment design and analysis of variance techniques were used to determine the significant main effects, curvilinear effects, and linear interactions at an alpha level of 1 percent. The following effects and interactions were significant at an alpha level of 1 percent: (a) main effects—salt content, lime content, curing temperature, curing time, molding-water content, and soil type and (b) interactions—lime content and salt content, lime content and curing temperature, lime content and molding-water content, salt content and curing temperature, curing temperature and molding-water content, curing temperature and soil type, curing time and soil type, and lime content and salt content and curing temperature. An engineering interpretation of each is given. A coded multiple regression model was developed to the estimate unconfined compressive strength in terms of the statistically significant main effects and interactions.

A combination stabilization strategy—the use of salt in conjunction with the lime treatment of clay soil—has been studied to determine whether small amounts of sodium chloride can be used to accelerate the stabilization phase of the lime-soil treatment process. Previous research by Mateos and Davidson (1) indicated that small amounts of sodium chloride are beneficial to the lime-fly ash treatment of Ottawa sand. The relationship between immersed compressive strength and sodium chloride content was parabolic, with maximum strengths at 7, 28, and 120 d curing times at a sodium chloride content of 1 percent. They also reported that 1 percent sodium chloride in conjunction with either calcitic hydrated lime or dolomitic monohydrate lime and fly ash produced higher compressive strengths than those observed for lime-fly ash-treated dune sand and friable silt without sodium chloride. A 6.1°C (43°F) curing temperature was shown to be detrimental to lime-fly ash-sodium chloride treatment of dune sand.

Thornburn and Mura (2), in a review of the literature of salt stabilization, have summarized several studies that used various inorganic salts with lime and lime-fly ash stabilization. This review did not note any specific reference to the effects of sodium chloride on the lime treatment of clay, although other compounds, such as sodium carbonate, have been used successfully to in-