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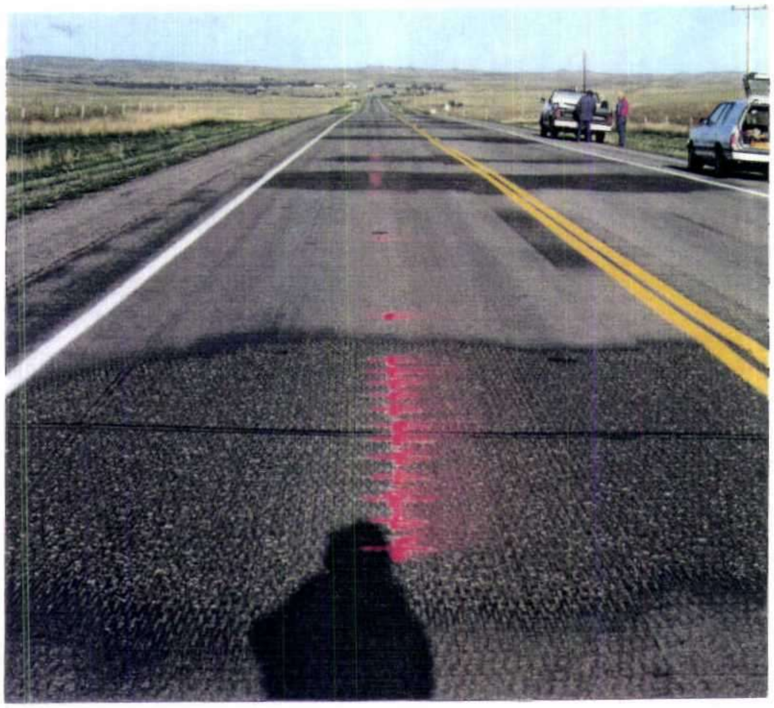

HEAVES AT TRANSVERSE CRACKS IN ASPHALT CONCRETE PAVEMENT

Study SD92-11
Final Report

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16. Abstract

This report presents the results of the investigation for the cause of heaves at transverse cracks on US Highway 212 between LaPlant and the Missouri River. The investigation included a one year field monitoring of surface movements at and near transverse cracks at the above project and at five control projects. During this period, soil and asphalt samples were taken and tested. The soil samples were tested for moisture content to determine any variation with time, and for swell potential. The asphalt cores were tested for AC content. A test pit to a depth of approximately six feet was dug at the location of a transverse crack, and soil samples were taken throughout the pit.

The results of the study indicate that the cause of the heaves is swelling of the subgrade soil from water infiltration through the transverse cracks. This is evidenced by the moisture content distribution around the cracks, which show a decreasing moisture content of both the base and the subgrade material with increasing vertical and horizontal distance from the crack. The problem also appears to have been aggravated by overcompaction and placement of the subgrade at too low moisture contents. Frost also appears to contribute to seasonal heaves.

Based on the results of the study, recommendations are given for remediation and reconstruction. Remediation procedures range from placement of an impermeable asphalt rubber membrane which will minimize reflective cracking and water infiltration through transverse cracks to grinding and overlays. For rebuilding and new construction on other projects, criteria for placement moisture and compaction are proposed. The use of an impermeable membrane between the subgrade and the base material is also suggested.

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CHAPTER I

EXECUTIVE SUMMARY

This report presents the results of the research project SD-92-11, "Heaves at Transverse Cracks in Asphalt Concrete Pavement", performed for the South Dakota Department of Transportation (SDDOT) by the South Dakota School of Mines & Technology. The project was performed during the period May 1, 1992 through June 30, 1993.

The principal objectives of the research project were twofold:

1. Determine the causes of heaves at transverse cracks on US Highway 212 from LaPlant to the Missouri River.
2. To recommend design, construction and maintenance standards, specifications and methods to eliminate or prevent heaves at transverse cracks.

The project was divided into several tasks, including a literature study, a summary of the construction history and procedure, field investigation and sampling, laboratory testing, analysis of observations and measurements, and the development of conclusions and recommendations.

In order to evaluate the cause of the heaves on US Highway 212 between LaPlant and the Missouri River, it was decided to compare this section with five other stretches of pavement (control projects) founded on similar subgrade soils. The field investigation included a 12 month study of lateral and vertical movements at and near 12 transverse cracks on the problem section and 4 transverse cracks on each of the five control projects. Soil samples were taken by crews from the SDDOT at and near the cracks on all highway projects investigated. The soil sampling was performed at three different times to study water content variations with time and season. In addition, the samples were analyzed for plasticity and swell potential. Asphalt cores were also taken, and were measured for temperature related volume changes. In addition, extraction tests for determination of asphalt cement content were performed on the asphalt cores following completion of the temperature volume change study. Toward the end of the project, a test pit was also dug on US Highway 212 between LaPlant and the Missouri River at Mile Reference Marker (MRM) 189.4 to determine the distribution of moisture content beneath a crack.

Following the completion of the field study and the laboratory testing, the data obtained was analyzed. Based on the results of the moisture content distribution in the subgrade soils around the cracks, it appears that water infiltration into the base and subgrade had taken place through the transverse cracks. With the availability of moisture, heaves have been and are being cased by both soil expansion and frost. It was also concluded that commonly accepted compaction procedures for swelling soils were not being followed. It was found that the soils were compacted at an average near 100 percent of the maximum density as obtained by the AASHTO T-99 method of compaction, and that over half the soils were placed at moisture contents dry of optimum. Thus, the subgrade soils were overcompacted on the dry side of optimum, resulting in a subgrade soil structure which is highly susceptible to swelling. Also, the subgrade on US Highway 212 from LaPlant to the Missouri River, was different from those on the control projects in that a significant portion of the subgrade consisted of large, intact shale blocks that were not broken up by the compaction process. Such blocks were also found during the excavation of the test pit. Up to 170 cracks per mile developed the fall and the first winter following construction. Relatively large crack openings developed and with a two year delay before crack sealing, severe heaving at the transverse cracks resulted.

Several remedial measures may provide solutions to the heaves on US Highway 212 from LaPlant to the Missouri River. The longevity of the measures will vary with the measure and the maintenance requirements for the solution chosen. A range of solutions are outlined in the report. Complete reconstruction should also be considered, provided an impermeable membrane is used between the subgrade and the base material. Grinding and overlaying the existing pavement may provide a short term solution, but is likely to require considerable maintenance in the future. Even though the heaves at the crack as caused by subgrade swelling may be tapering off, the moisture redistribution and moisture infiltration at new cracks will continue to cause heaves. The uneven moisture content distribution near the cracks will most likely produce seasonal heaves due to frost, and will be a problem for all measures except complete reconstruction. Remediation procedures that do not include measures to reduce or minimize reflective cracking and water infiltration through transverse cracks are in all likelihood going to require considerable amounts of maintenance and frequent resurfacing.

The primary recommendation is that the section of US Highway 212 between LaPlant and the Missouri River be overlaid using a procedure developed and successfully applied at the Arizona Department of Transportation. The procedure involves the sealing of existing cracks, tacking an impermeable membrane over the pavement to prevent moisture infiltration into the subgrade from the cracks, and placing a new wearing asphalt course above the membrane. It is also recommended that the membrane be placed continuously across the pavement, into the ditches, and up the cut slope opposite the ditch. This is to prevent moisture from infiltrating the swelling soils and being drawn under the pavement by capillary action.

Alternately, the pavement may be milled, rolled, and covered with a temporary bearing surface. This will allow the base and subgrade to "breathe", and the moisture content to equilibrate. A permanent surface may then be placed. The disadvantage with this method will be problems with rough rideability and possibly low bearing capacity until a permanent surface is placed. In the long run, this solution should require a minimum of maintenance, provided consideration is given to measures that will minimize water infiltration through pavement cracks.

In order to minimize heaves at transverse cracks on future projects, it is recommended that the SDDOT review the earthwork placement, moisture conditioning, and compaction criteria presently used. Particular attention should be given to disking, placement water content, and degree of compaction.

As heaves at transverse cracks are primarily caused by water infiltrating the subgrade through the cracks, it is recommended that on new sections, crack sealing be performed following the first winter after paving, and not on a fixed time schedule of two years after paving as is the current practice. Chip sealing is not related to the heave problem and can be performed as is.

In order to minimize crack development in areas where the subgrade consists of expansive clays soils, it is recommended that the SDDOT continue to monitor the performance of different AC material and aggregate mixes and use those that have shown the best performance. It is also recommended that the SDDOT implement a study of the AC material on US Highway 212 from LaPlant to the Missouri River.

CHAPTER II

BACKGROUND

II-1 INTRODUCTION

After completion in 1988, heaves along transverse cracks developed on an approximately 19 mile section of US Highway 212 from LaPlant to the Missouri River. The heaves developed in excess of one inch high, extended from 3 to 10 feet on each side of the transverse cracks, and have resulted in unacceptably rough rideability and reduced safety. At this time, emergency maintenance in the form of milling of the pavement at the heaves and crack sealing has been performed. Although rideability has been improved, the pavement is still rough, and further heaving appears to be taking place. The highway is located in an area where the subgrade soils consist of highly expansive soils and shales. Swelling is common if moisture penetrates or accumulates in expansive soils, and problems relating to swell and heave have been experienced previously in South Dakota and other states, but rarely on a widespread scale as observed on US Highway 212.

Several other stretches of highways in the area are also founded on expansive soils. The reconstruction and repaving of US Highway 212 from LaPlant to Ridgeview and from Ridgeview west 12 miles was completed in 1990. The section from LaPlant to Ridgewiew has very few transverse cracks, and although the subsurface conditions along this route are very similar to those on US Highway 212 from LaPlant to the

Missouri River, no significant heaves at the cracks have yet developed. At the section from Ridgeview west 12.1 miles, numerous cracks have developed, and some heaves are evident, but the heaves are not of the same order of magnitude as those from LaPlant to the Missouri River. Significant heaves along traverse cracks have not developed on the projects on SD Highway 73 from Howes Corner to Faith. The projects from Howes Corner to Faith were completed in the period 1988 to 1990. Along Highway 34 from White Owl to Howes Corner, which was completed in 1991, only four cracks have developed, and no heaves are evident.

II-2 OBJECTIVES

Due to the unusual severity of the heave problem along US Highway 212 from LaPlant to the Missouri River, the SDDOT decided to perform a research project to identify the cause(s) of the heaves on this route. The principal objectives of this research project were twofold:

1. Determine the causes of heaves at transverse cracks on US Highway 212 from LaPlant to the Missouri River.

2. To recommend design, construction and maintenance standards, specifications and methods to eliminate or prevent heaves at transverse cracks.

Although any recommendations for revised construction procedures may not necessarily result in a reduction in initial construction cost, the solutions arrived at may result in lower maintenance cost and longer highway service life. Rideability will also be improved, which is important to the traveling public in terms of safety, comfort, and vehicle maintenance.

II-3 SCOPE

The research project was divided into seven different tasks. The tasks are listed below, and the report is divided into sections that generally follow the scope of the investigation. The tasks to be performed were as follows:

1. Meet with the Technical Panel to review the work plan and scheduling;
2. Review and summarize literature relevant to heaves and transverse cracks in asphalt concrete. This will include contacting other states and agencies who may have experienced similar problems;
3. Perform periodic field measurements for a minimum of 12 months to monitor heave development and crack growth on the projects listed in Table II-1 to determine the extent of the problem;
4. Evaluate design, materials, soil types, construction methods and maintenance practices on the referenced projects;
5. Recommend changes to design, construction, and maintenance standards, specifications and methods;
6. Submit a final report summarizing relevant literature, research methodology, data, findings, and conclusions;
7. Present findings and conclusions to SDDOT Research Review Board at the conclusion of the project.

Specifically, the research was to study the project on US Highway 212 between LaPlant and the Missouri River, and compare it to the performance of five other highway projects (control projects) constructed using similar design and founded on similar subgrade soils. The specifics about the location of the different projects and length of the sections are presented in Table II-1. below:

TABLE II-1. PROJECT SPECIFICS

PROJECT NO.	LENGTH	LOCATION	YEAR SURFACED
F0212(56)187	18.7 miles	LaPlant East to the Missouri River	1989
F0212(40)178	08.7 miles	Ridgeview to LaPlant	1990
F0212(72)166	12.1 miles	Ridgeview West 12.1 miles	1990
F0034(47)96	15.6 miles	White Owl to Howes Corner	1991
F0073(14)146	14.2 miles	Howes Corner North	1989/1990
F0073(12)160	14.1 miles	Faith South	1988

CHAPTER III

LITERATURE REVIEW

III-1 GENERAL

A literature search on swelling soils in highway subgrades, on heaving along transverse cracks (also termed tenting), and the formation of transverse cracks was performed. Information was obtained from both existing literature on the subjects and from interviews with representatives from other departments of transportation. Universities performing research on related topics were also contacted.

Significant amounts of research have been performed on transverse cracking in asphalt pavement and on treatment of swelling soils under highways. The International Conferences on Expansive Soils, publications from the Transportation Research Board, the Geotechnical Journal of the ASCE, and numerous conferences on the subjects contain a multitude of papers on swelling soils, highway roughness resulting from swelling clays, and asphalt pavement cracking. Many of these publications are cited in the following paragraphs. In the United States, Robert L. Lytton at the Texas Transportation Institute (17,18) has researched solutions to highway related heave and has published numerous papers on the subject. The problem is also treated in detail in a publication from the Arizona Department of Transportation (29), and in a follow up Transportation Research Board publication by Forstie et. al. (4). A key summary of experiences with swelling soil in highway subgrades was presented by Snethen et. al. (31).

Based on the nature of the problems experienced on US Highway 212 and their probable causes, the literature search was divided into four parts:

1. Causes of heaves at transverse cracks.
2. Transverse crack formation;
3. Earth work control of swell, and;
4. Post construction remediation.

The amount of available literature, especially on swelling clays and compaction is enormous, and all cannot be referenced in this report. However, key publications are referenced and cited in the bibliography.

III-2 CAUSES OF HEAVES AT TRANSVERSE CRACKS

The offices of the departments of transportation in Arizona (3), Colorado (15), Montana (37), Utah (13), and Wyoming (8) were contacted about their experience with heaves along transverse cracks. All departments had experienced problems with heaves, although possibly not of the severity as seen on US Highway 212. None of the departments had performed any significant research on the causes of the problem, but most felt that heaves along transverse cracks is a result of surface water infiltration through the cracks. Tenting could then take place either as a result of swelling of clay materials in the subgrade or by frost. It was felt that the number one method of prevention is twofold: frequent inspection for transverse crack development, and prompt crack sealing. It was not felt that the problem was related to the type of asphalt or base material used. However, the problem could be aggravated using asphalt types that are brittle and more susceptible to cracking, especially under low temperature and fast falling temperature conditions. Most departments also felt that frost could aggravate the problem, especially when a highly compacted base material with more than 6 to 8 percent fines was used. In addition, overcompaction of the subgrade as well as inadequate base thickness was felt to be contributing factors under certain environmental conditions. The solutions to the problems differed from state to state. Some states require a two foot thick layer of sand under the pavement such that any moisture penetrating the base could be drained off. Other states do not allow the use of a sand layer, but require full depth asphalt in areas with highly expansive subgrade material, while other states, such as Arizona and Wyoming, preferred the use of synthetic membranes to prevent moisture infiltration. Compaction of the subgrade is generally specified wet of optimum. Lime treatment has generally not been successful, while over excavation and moisture conditioning of the subgrade has showed some promise.

III-3 CAUSES OF TRANSVERSE CRACKS

It is generally accepted that transverse cracking in asphalt pavement is caused by several mechanisms (1). Some of the causes are listed below. The order of importance depends on soil and climatological factors, and may vary from area to area:

1. Stresses induced by thermal shrinkage, especially during thermal shock;
2. Stresses in non-asphalt treated base layers;
3. Variations in subgrade moisture and temperature, and;
4. Differential frost-heaving of the subgrade.

Low temperature cracking is generally confined to the northern regions of the United States and most of Canada where winter temperatures drop below about -10°F (-23.3°C). In order to study the problem, the Asphalt Institute formed an ad hoc committee, which produced a report with guidelines for design techniques to minimize low temperature transverse cracking (1). The occurrence, severity, and frequency of temperature cracking was found to depend on several factors, including:

1. Climatic Effects;
2. Subgrade Type;
3. Asphalt Properties;
4. Mix Properties and Mix Design;
5. Pavement Design and Structural Effects;

6. Pavement Age, and;

7. Traffic Effects.

Recent research by the Texas Transportation Institute has shown that transverse crack frequency may be a function of the plasticity of the subgrade material, as the frequency of cracking and the width of cracks generally increases with soil Plasticity Index. This may be true in climates without the extreme temperature conditions in the northern regions, where thermal effects most likely overshadow the effects of swelling and shrinkage of the subgrade. Recent research under cold climate conditions Greenstein et. al., (5) indicate that selection of an asphalt cement with a low susceptibility to cracking can decrease crack frequency substantially. Ambient air temperature was also found to be of importance, as pavement laid at air temperatures between 53°F (12°C) and 59°F (15°C) was associated with accelerated surface cracking. Placement of pavement was therefore recommended stopped as the air temperature dropped below 59°F (15°C). Salter and Al-Sharkarchi (28) also showed that sharp-grained crushed aggregate also performed considerably better than rounded gravel aggregates, and reduction in thermal damage was also noted with the use of copolymers and higher penetration binders. Marks and Huisman, (19), showed that the crack susceptibility is related to the Pen-Vis number (PVN), with a PVN of zero being an excellent AC with low temperature susceptibility. Their research on four stretches of Highway in Iowa showed that the average transverse crack frequency was decreased from 35 feet to 170 feet by going from a high temperature susceptibility to a low temperature susceptibility AC. By increasing the AC content of the pavement by one percent, the transverse crack frequency was further increased to 528 feet. Studies in South Dakota (2) have also shown that transverse cracking is related to asphalt hardness (2).

On the more theoretical side, Raj Dongree et. al. (27) compared critical strain energy release rate J_{ic} (the critical value of the J integral) for the development of a fracture criterion at low temperatures, and found that J_{ic} showed promise as a fracture characterization parameter for asphalt concrete at low temperatures.

III-4 EARTH WORK CONTROL OF SWELL

It is generally accepted that heaves at transverse cracks can be minimized if efforts to control swelling of subgrade are implemented during construction. Design criteria for subgrades constructed from swelling soils and shales are presented in detail in a publication by Snethen et. al (31). Snethen et. al. give a detailed summary of construction practices from states which have problems with swelling clays, and also outlines practices used by the different states to minimize swelling problems in highway construction. Such procedures include:

1. Over excavation of swelling soil and replacement with non-expansive soils;
2. Application of surcharge pressure;
3. Preventing access of water to the soil;
4. Prewetting of the soil;
5. Chemical stabilization;
6. Mechanical stabilization (Compaction and moisture control), and;
7. Heat treatment.

III-4.1 OVER EXCAVATION AND SURCHARGE

Over excavation and replacement is controlled by availability of non-swelling material, and may not be economical and possible, since areas of swelling soils, especially in South Dakota, extend over vast areas. Experience in South Dakota (21) also indicates that over excavation and replacement with non-swelling soils to a depth of up to 18 inches has been unsuccessful. Application of surcharge pressure is also nearly impossible with highway construction, since surcharge pressures generated from the pavement and base material is in most cases insufficient to counterbalance the swelling pressures.

III-4.2 PREVENTING ACCESS OF WATER TO THE SOIL

Among the most promising methods applied is the prevention of water infiltration into the subgrade. This procedure has been applied in several states, and has been particularly successful in Arizona (4), Colorado (15), and Wyoming (8), but has met with mixed results in South Dakota (9,20,22). It was concluded that the lack of success with one South Dakota project may lay with the difference in procedure as compared to those used in some other states. The applications in Arizona, Colorado, and Wyoming included placement of membranes under the entire pavement, extending into the ditch and up the slope outside the ditch, thus preventing water standing in the ditch from migrating under the pavement section. The application in South Dakota also included placement of a membrane under the pavement, but the membrane was placed vertically to a depth of four feet outside of the shoulder, thus forming a cut off wall just outside the shoulder. Water standing in ditches and water beyond the membrane could then infiltrate the soil and migrate under the cutoff wall and cause swelling. In Arizona, the same procedure has been successful when the membrane was extended vertically to a depth of eight feet (3).

III-4.3 PREWETTING OF THE SOIL

Prewetting of the soil includes ponding of water, and in some instances drilling of holes at close spaced centers and filling the holes with water. This method appears practical only in areas where the soil has a high permeability, and where water is readily available, and has proven successful in some areas. Since neither high permeability soils nor water are readily available in South Dakota, the method does not seem applicable here.

III-4.4 CHEMICAL STABILIZATION

Chemical stabilization has been tried with lime, fly-ash, Portland Cement, and other chemical admixtures. Of these, stabilization using lime has been the most successful, but the method has resulted in mixed results. Injection of lime through drill holes was tried in South Dakota (33), and did improve serviceability, but the migration of lime away from the drill holes was quite limited. The mixing of lime with the soil as the lifts are placed, appears to be the most effective technique to reduce swell. The method does have limitations, in that it requires even application, thorough mixing, and thus requires extensive quality control. The addition of lime to clay also increases the frost susceptibility.

III-4.5 MECHANICAL STABILIZATION.

Control of placement water content and degree of compaction has also been an effective method in reducing subgrade swell. The basic research on compaction and its influence on the properties of soils was thoroughly researched in the 1950's and 1960's, and the subject is now well understood (6,30). Gibbs and Holtz (6) have shown that swell potential is reduced at moisture contents above optimum, and by lowering compaction densities, and that overcompaction at molding water contents below optimum can lead to serious swelling problems. Also, kneading type compaction gives a clay mineral structure that is less susceptible to swelling. However, the combination of high molding water content and compaction procedures appear to be the most important factor. It has been demonstrated that reducing the compaction requirements and increasing the moisture content has been effective in reducing or retarding swelling.

The South Dakota Department of Transportation states that a 3 to 6 foot zone of the subgrade is to be constructed from soils with higher moisture content and lower density than the underlying embankment (32). The upper zone of the subgrade should be constructed at 92 percent of the maximum dry density as obtained by the AASHTO T-99 method of compaction, and the placement moisture content should lie between optimum and three percent above optimum.

III-4.6 HEAT TREATMENT

Heat treatment has not been tried extensively in the United States, and does not appear to be economical at this time.

III-5 POST CONSTRUCTION REMEDIATION

Experience with post construction remediation of swelling problems in Arizona has been treated in detail by Forstie et. al. (4) and Scofield (29). Forstie conducted several tests and conducted a survey of procedures, and summarized their effectiveness. In all, 14 procedures were outlined. A summary of the procedures and the results are presented in the table below. The significance of the study as related to the current project is that the swelling clays encountered by Forstie had Liquid Limits and Plasticity Indexes on the order of 65 and 38 percent, respectively, which is comparable to those encountered on the sections studied in this report. The results of the studies are summarized in Table III-1.

TABLE III-1. POST REMEDIAL MEASURES BY ARIZONA DEPARTMENT OF TRANSPORTATION

PROCEDURE	SITES	RESULTS
Subdrains (French Drains)	2	Negligible
Undercut and Recompact	2	Negligible
Undercut and Replace	3	Negligible
Widen Cut Ditches	2	Negligible
Plastic Sheets on Shoulders	1	Negligible
Recompact with Lime or KCl	2	Negligible
Postholes with Lime, KCl, or Ion-Tech Chemicals	3	Negligible
Ponding with Lime, NaCl or KCl	7	Minor
Electro-osmosis with Lime, KCl	7	Minor
Paved cut Ditches	5	Significant
Cat-Blown Asphalt Membrane	2	Significant
Full Depth Asphalt Concrete	2	Significant
Asphalt-Rubber Membrane	4	Significant

It should be noted that with the impermeable membranes, existing, deteriorated sections were first leveled with asphalt, then overlaid with the membrane, and finally paved with two inches of asphalt overlay. Control sections were also established where no membrane was used. The results showed that transverse crack frequency was significantly reduced, and that service life, based on initial observations, roughness and rideability, may have been tripled from that of the originally constructed pavement. Furthermore, based on the Arizona Department of Transportation's experience with overlays and using membrane treatment, the following conclusions were drawn (4):

1. Treatment with asphalt-rubber membranes will prevent reflective cracking;

2. Shoulder and ditch membrane treatments (asphaltic rubber, paved shoulders and ditches or any other suitable water-proofing membrane layer) can and will improve the long term ride performance of a highway placed over an expansive soil.

3. With both ride and cracking being reduced in severity, maintenance cost would be reduced.

The membrane implementation is now in effect on an operational basis by the Arizona Department of Transportation for roads on expansive subgrades.

CHAPTER IV

CONSTRUCTION HISTORY

IV-1 GENERAL

Most of the information provided, about the construction history and procedures, was obtained from files maintained in the SDDOT's Belle Fourche and Pierre area offices. The information, about soil conditions and quality control testing, was taken from soil boring logs and field test records. Information regarding construction procedures, specific events, and dates is based on review of the design documents, field diaries and interviews with the field crew members (7, 16).

The pavement design for the sections studied are similar, however, the thickness of the design components of the pavements varied from location to location. A summary of asphalt, base, and subbase thickness is presented in Table IV-1 below.

TABLE IV-1.
PAVEMENT COMPONENT DESIGN THICKNESS

LOCATION	ASPHALT COURSE			BASE	SUBBASE
	WEARING	LEVELING	BASE		
US212E	2.5		4	6	10
US212M	2.5		4	8	8
US212W	1.5	1	4	8	8
SD73N	1.5	1	3	6	8
SD73S	2	1	4.5	5	6
SD34	1.5	1	4	6	8

All dimensions are in inches

The base material on all projects is very similar, and consists of natural gravely sand with less than 5 percent passing the Number 200 Sieve. The gravel and sand particles classify as sub-rounded. The Plasticity Index of the fines is less than 6 percent. The soil classification in accordance with the AASHTO classification system is A-1-a.

IV-2 US HIGHWAY 212 LAPLANT TO THE MISSOURI RIVER

The earthwork for the highway was completed during the summer of 1988, and paving was completed in the late summer of 1989. The earthwork was performed by two different contractors, Schweigert Construction Company, Inc., and Knight Construction & Heavy Constructors, Inc..

The soil investigation performed along the route, indicated that the subgrade material consisted of high plasticity clay, classified as A-7-6 material by the AASHTO soil classification system. The average Liquid Limit and Plasticity Index as calculated from the laboratory tests performed for the investigation, was 76 and 46 percent respectively.

The earthwork consisted of shoulder to shoulder undercutting to depths between 3 and 6 feet, and the subgrade fill consisted of fresh cut shale and subgrade salvage material from the old highway. Based on conversations with SDDOT inspectors present at the site (16), the subgrade salvage material was not separated from material obtained in the undercutting process. The subgrade fill also contained large blocks of shale, which were mixed with the old material. No disking of the material was performed, and breakdown of the shale blocks was not complete. The base material consisted of a lower two inch layer of salvage material from the old base, with upper portion of the base consisting of virgin gravel.

The results of the density and moisture test on the subgrade performed during construction are shown in Figures IV-1 through IV-4, and are separated by contractor. No significant difference between the results from the different contractors is evident. The results indicate the moisture content varied from 4 percent dry of optimum to 7 percent wet of optimum. Percent compaction was nearly identical for the contractors, with an average degree of compaction of 99.7 percent. The range in compaction was from 95 to 107 percent.

The project was paved using 50 percent virgin asphalt (85/100 penetration grade) and 50 percent salvage asphalt. The AC was supplied through Little America and Conoco. Based on interviews with the site inspectors and information obtained from the construction diaries, the contractor had difficulties with low air voids. In order to bring the air voids up, the AC content of the mix was reduced by .3

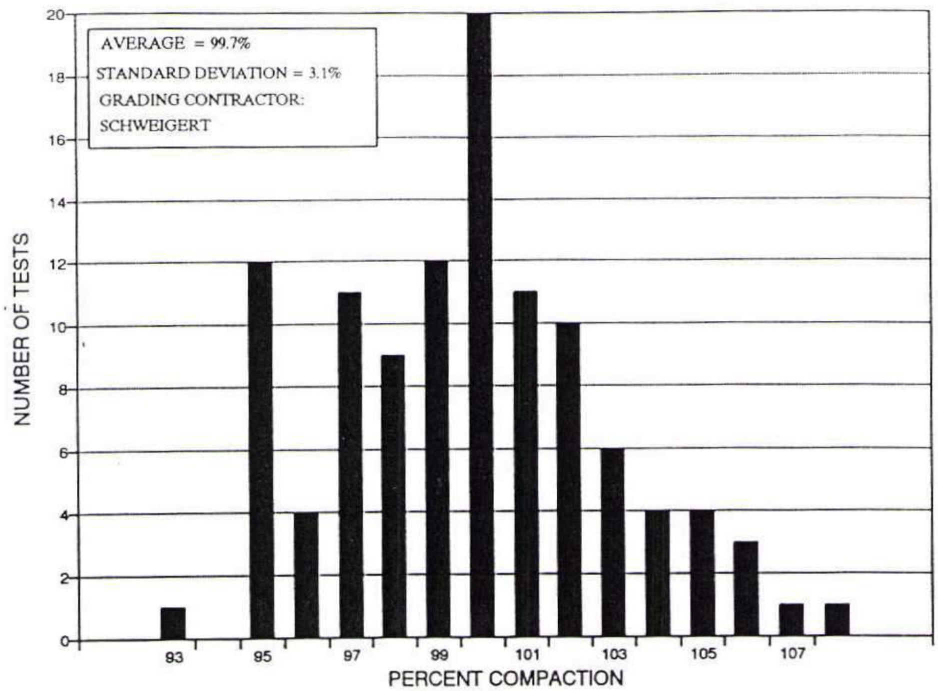


FIGURE IV-1. US212 LAPLANT TO RIVER DENSITY DISTRIBUTION

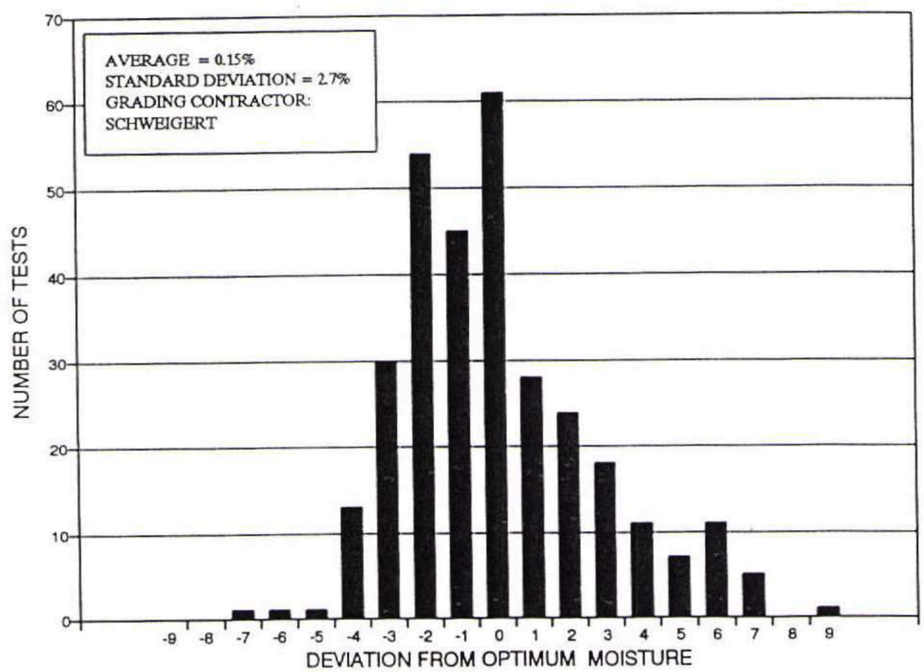


FIGURE IV-2. US212 LAPLANT TO RIVER MOISTURE DISTRIBUTION

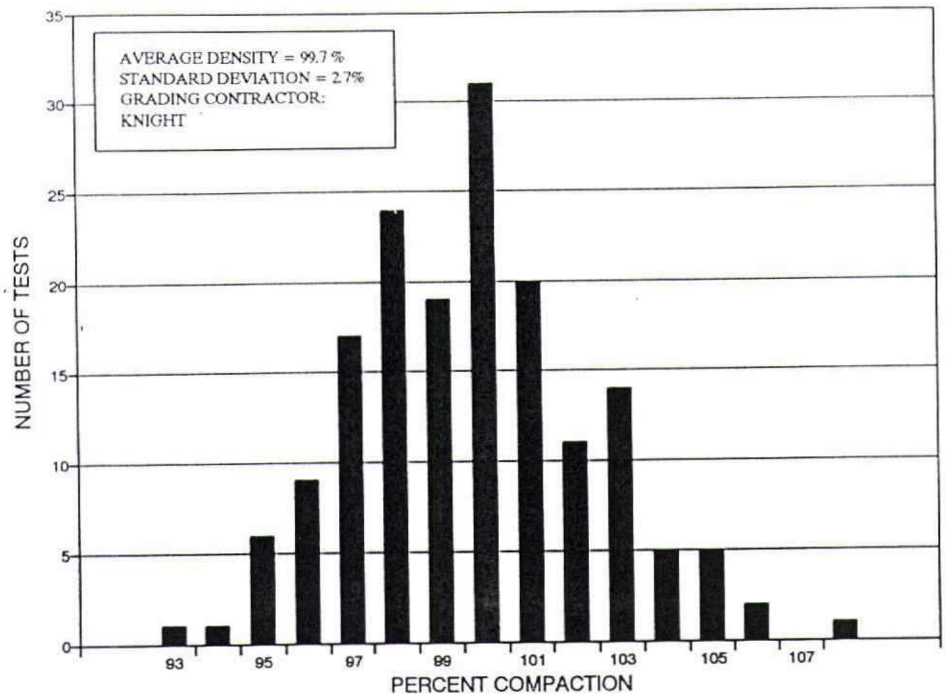


FIGURE IV-3. US212 LAPLANT TO RIVER DENSITY DISTRIBUTION

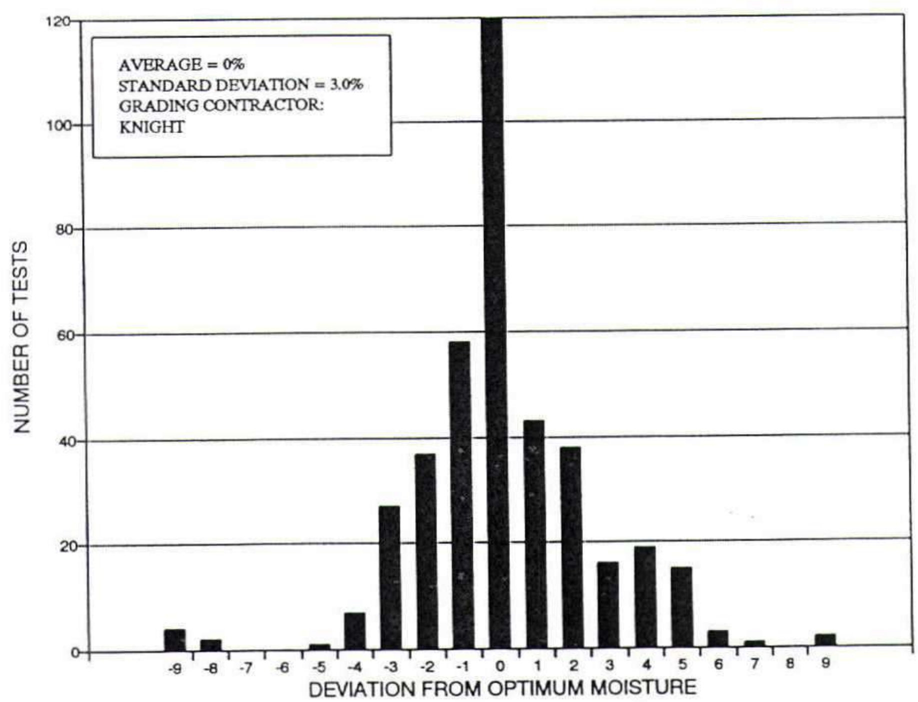


FIGURE IV-4. US212 LAPLANT TO RIVER MOISTURE DISTRIBUTION

percent. In accordance with information obtained, tearing of the mat was observed in June of 1989. In addition, severe cracking developed during the winter of 1989-90, however no crack or chip sealing was performed until 1991.

IV-3 US HIGHWAY 212 RIDGEVIEW TO LAPLANT

The earthwork was started in late 1989, and completed during the summer of 1990. Paving was completed during the early fall of 1990. The earthwork contractor was A.G.E. Corporation.

The laboratory tests performed on samples obtained during the subsurface investigation along the route indicate that the subgrade soils primarily consisted of high plasticity clays with Liquid Limits ranging from 52 to 91 percent and Plasticity Indexes in the range 32 to 55 percent. The average Liquid Limit was 71 percent while the average Plasticity Index was 46 percent. Most of the subgrade material classified as A-7-6 in accordance the AASHTO classification system.

The subgrade was undercut from toe to toe to a depth of 4 feet. The old subgrade consisted of wet clay, while the virgin material was highly weathered shale (hard clay) which was free of shale fragments. The old subgrade backfill was separated from the virgin material, and the materials were not combined during placement of the new subgrade. No disking of the fill was performed on the project.

The results of the density and moisture tests performed during construction are presented in Figures IV-5 and IV-6. The average deviation from optimum moisture content was 0.2 percent, which indicates that slightly more than half the samples were placed dry of optimum. The standard deviation was 2.0 percent based on the analysis of the deviation values for moisture content from optimum. The analysis of the density tests gave an average degree of compaction of 98.9, with compaction for most samples ranging from 95 to 106 percent.

The project was paved using both virgin and recycled asphalt. The AC was 85/100 penetration grade for the base course and AC-5 for the wearing course, and was supplied by the Little America Sinclair refinery in Wyoming and by Koch. Only a handful of cracks developed over the winter of 1990-91, and crack and chip sealing was performed in 1991.

IV-4 US HIGHWAY 212 RIDGEVIEW WEST 12.1 MILES

The earthwork on the portion of US Highway 212 from Ridgeview west spanned from July 1989 through October of 1989, and the pavement was laid in the summer of 1990. The earthwork contractor on this project was Schweigert Construction.

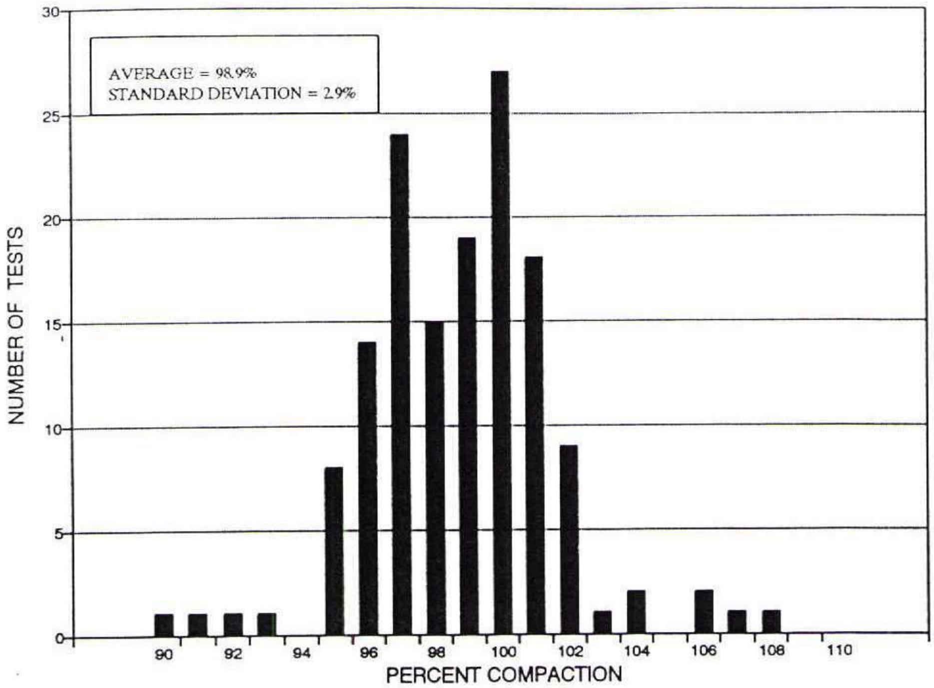


FIGURE IV-5. US212 RIDGEVIEW - LAPLANT DENSITY DISTRIBUTION

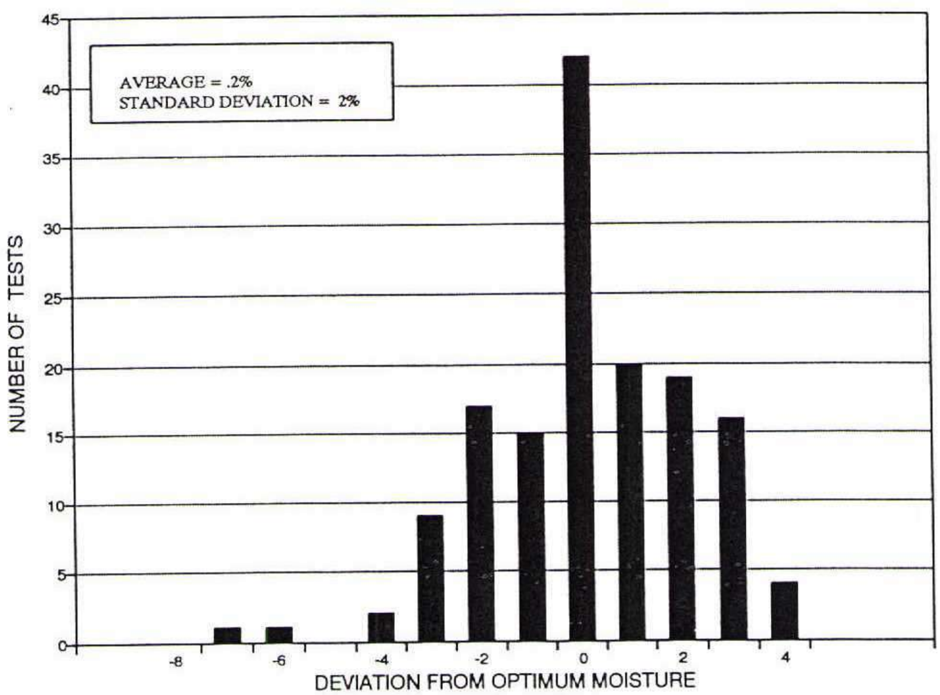


FIGURE IV-6. US212 RIDGEVIEW - LAPLANT MOISTURE DISTRIBUTION

The subgrade earthwork consisted of shoulder to shoulder undercutting to a depth of 3 to 4 feet. In reference to interviews with the field crews (16), the old railroad embankment running parallel with the route was used for fill on the eastern five miles of the project. The fill consisted of "good weathered shale", which was free of large and small shale fragments. No effort was made to separate excavated virgin soil and old fill, hence the subgrade fill was placed as a mixture of old and new fill. No disking was performed on the fill following placement of the lifts.

The subsurface soil investigation along the route show that the soil consists of weathered shale and clay of high plasticity with most soils classifying as A-7-6 in accordance with the AASHTO classification system. The laboratory test performed by the SDDOT showed Liquid Limits from the high 40's to the mid 80's, with an average of 60 percent. The Plasticity Index varied from the mid 20's to the mid 40's. The average Plasticity Index was 36.

The results of the density and moisture tests performed during construction are presented in Figures IV-7 and IV-8. The average deviation in moisture content from optimum was .4 percent, which indicates that more than half the samples were placed wet of optimum. The standard deviation from optimum moisture content was 2.0 percent. No samples were more than two percent dry of optimum. The analysis of the

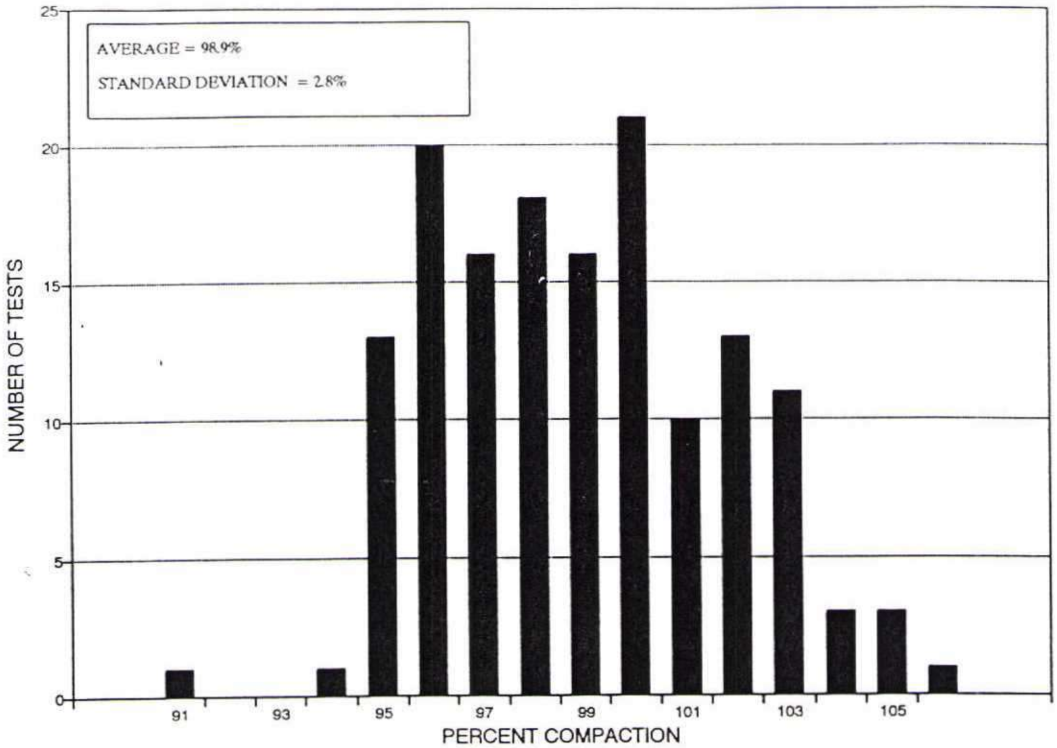


FIGURE IV-7. US212 RIDGEVIEW WEST DENSITY DISTRIBUTION

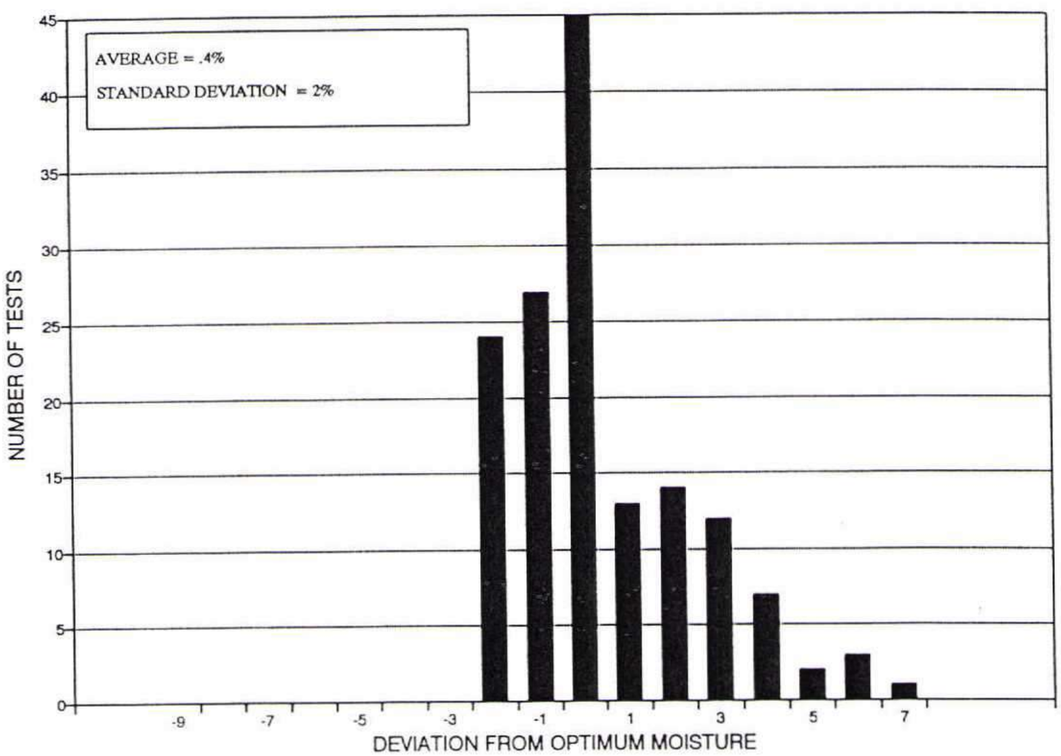


FIGURE IV-8. US212 RIDGEVIEW WEST MOISTURE DISTRIBUTION

density tests gave an average degree of compaction of 98.9, with compaction for most samples ranging from 95 to 105 percent. As can be seen from Figure IV-7, the density test results are almost evenly distributed between 95 and 103 percent compaction.

Penetration grade 85/100 AC was used for the pavement, and was supplied by the Little America Sinclair refinery in Wyoming, and by Koch of Minnesota. In accordance with the construction diaries, the contractor had problems with obtaining satisfactory asphalt densities. A moderate amount of cracking took place over the winter 1990/91. Crack and chip sealing was performed in 1991.

IV-5 SD HIGHWAY 73 FAITH SOUTH

Earthwork operations on the section of Highway 73 from Faith to 14.1 miles south commenced in June of 1987 and was completed in early November the same year. The pavement was laid in 1988. The grading contractor was Zandstra Construction.

The results of the subsurface investigation showed that the soils along the route consisted of low to high plasticity clay and weathered shale. A review of the Atterberg Limits tests indicate that the vast majority of the soils were A-7-6 material. The Liquid Limits ranged from 26 to 82 with an average of 52, while the Plasticity Index ranged from 7 to 59. The average Plasticity Index was 31.

The specifications on subgrade earthwork called for toe to toe undercutting to a depth of 3 feet, with deeper cuts where soft material was encountered.

The analysis of the field density tests show that most of the subgrade was generally compacted at between 94 and 107 percent of the maximum dry density as obtained from the AASHTO T-99 compaction test. The average degree of compaction was 99.1 percent, with a standard deviation of 4 percent. The in place moisture content tests show tests, where soft material was encountered, ranging mostly from 9 percent dry of optimum to 8 percent wet of optimum. Most tests were from 3 percent dry of optimum to 4 percent wet of optimum. The average in place deviation from optimum moisture content was 0.2 percent dry of optimum, while the standard deviation from optimum moisture was 2.8 percent. Hence more than half the subgrade was compacted dry of optimum. The density and moisture histograms are shown in Figures IV-9 and IV-10, respectively.

On this project, 120/150 penetration grade AC was used. The AC was supplied by CENEX in Laurel, Montana, and by EXXON in Billings, Montana. Problems with crack formation during compaction was also experienced on this project. However,

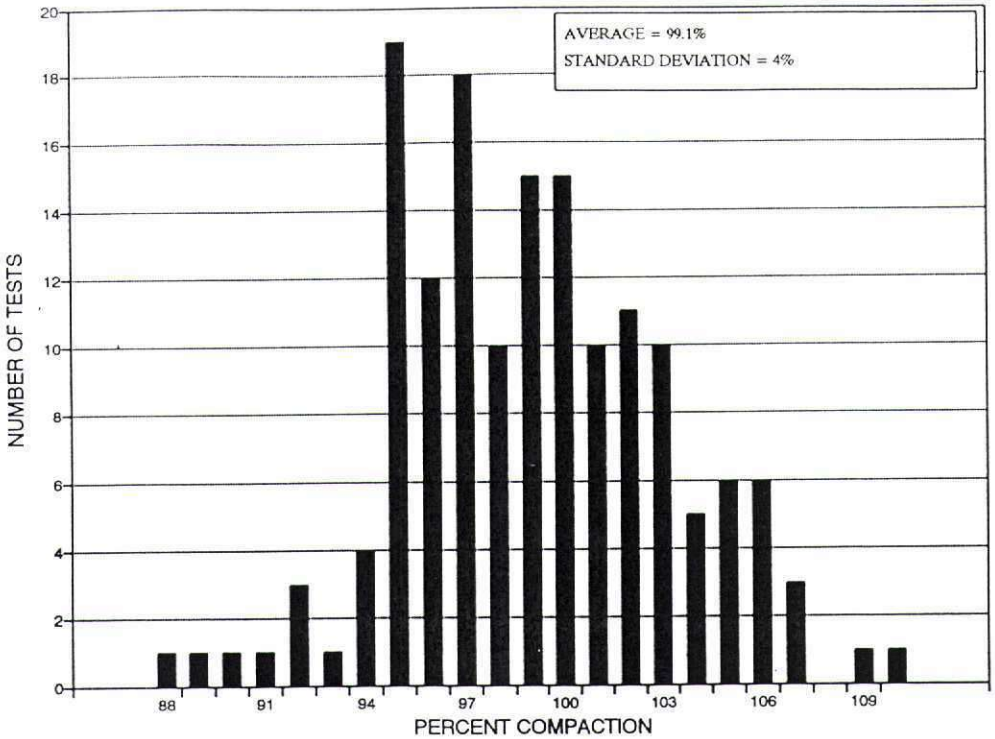


FIGURE IV-9. SD73 FAITH SOUTH DENSITY DISTRIBUTION

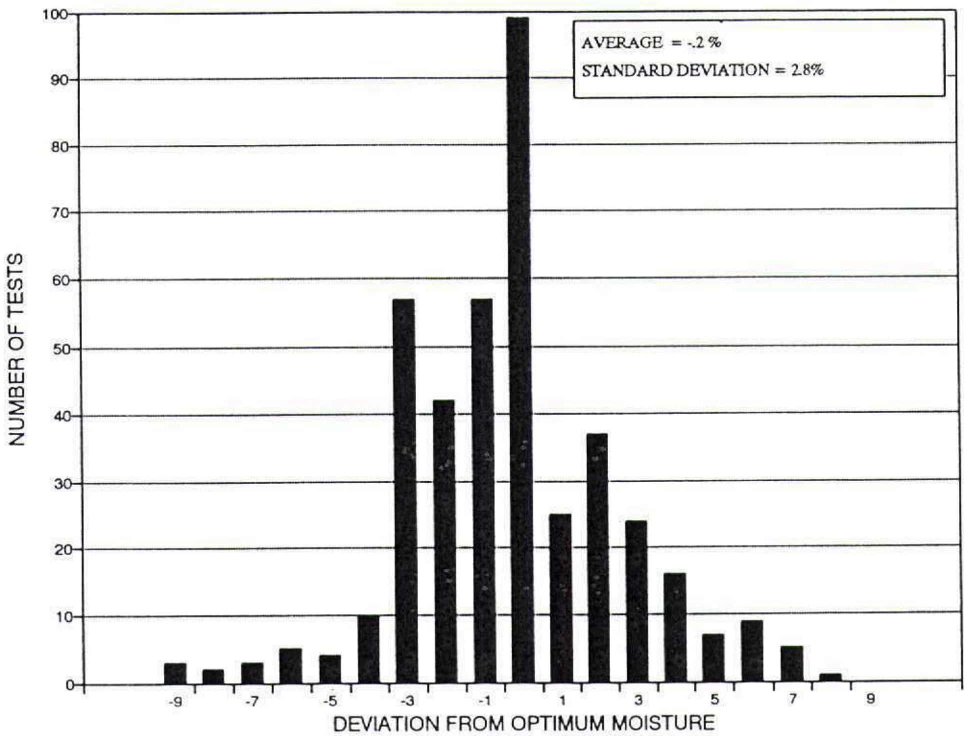


FIGURE IV-10. SD73 FAITH SOUTH MOISTURE DISTRIBUTION

the problem was managed by modifying placement temperature. Crack sealing on this project was performed during 1991, and chip sealing was performed during 1992.

IV-6 SD HIGHWAY 73 HOWES CORNER NORTH

The earthwork on Highway 73 from Howes Corner to 14.1 miles north was begun in late summer of 1988 and completed in mid November of 1989. The contractor was A.G.E. Corporation.

The subsurface investigation performed for the project showed that the soils along the route consisted of clays and weathered shales with Liquid Limits in the range from the low 40's to the high 70's, with the exception of one value of 90 percent. The average value of the Liquid Limits was 59. The Plasticity Indexes ranged from 18 percent to 68, with an average value of 36. In accordance with the AASHTO classification system, most soils fall in the A-7-6 range, and a few samples in the A-6 range.

The earthwork contract called for toe to toe undercutting to a depth of 3 feet, with deeper undercutting where required. The subgrade material was all salvage embankment material and material obtained from the undercutting operation. All shale subgrade material was highly weathered and contained no intact shale fragments.

A summary and analysis of the field density tests show that most of the subgrade was generally compacted at between 94 and 107 percent of the maximum dry density as obtained from the AASHTO T-99 compaction test. The average degree of compaction was 99.9 percent, with a standard deviation of 3.4 percent. The in place moisture content tests show tests ranging mostly from 4 percent dry of optimum to 2 percent wet of optimum, with an average value at 1.1 percent dry of optimum, and a standard deviation of 2 percent. Based on the test summary, it appears that most of the subgrade was compacted dry of optimum. Histograms for the density and moisture tests are shown in Figures IV-11 and IV-12.

The paving began in late 1989 and was completed in 1990. In the beginning, 120/150 penetration grade AC was used, however, due to tenderness and instability problems during compaction, it was decided to change to 85/100 penetration grade AC. The AC was supplied by CENEX of Laurel, Montana. Chip sealing was performed in 1992, however, it was preceded by crack sealing 1991.

IV-7 SD HIGHWAY 34 WHITE OWL TO HOWES CORNER

The earthwork portion of Highway 34 from White Owl to Howes Corner was completed in 1990, and paving was completed in 1991. The earthwork contractor was A.G.E. Corporation. The design documents called for a 3 foot toe to toe undercut.

The subsurface investigation performed by the SDDOT prior to construction showed that the subgrade material would be

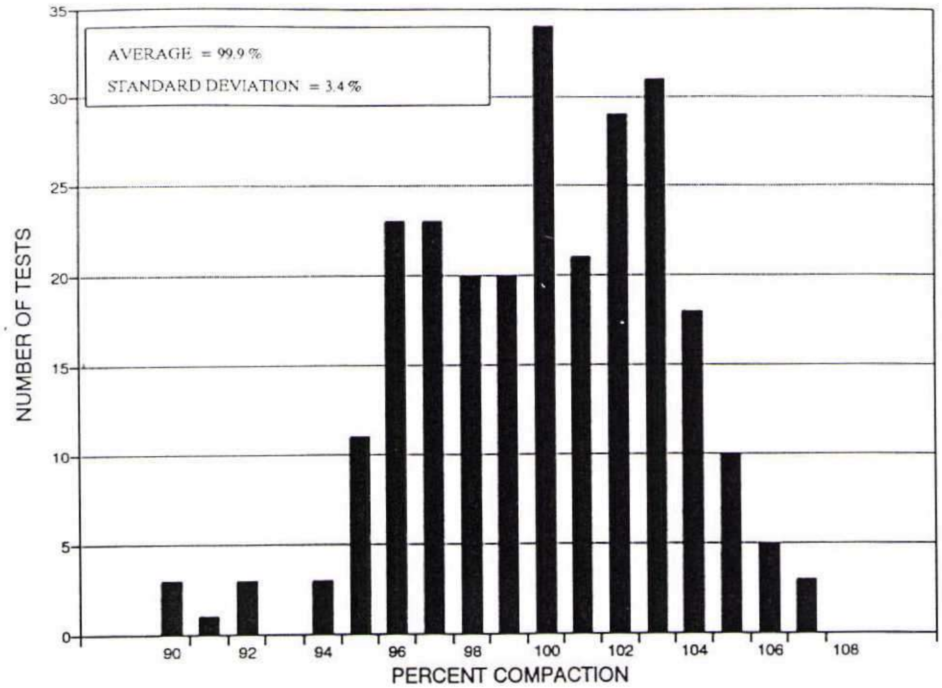


FIGURE IV-11. SD73 HOWES NORTH DENSITY DISTRIBUTION

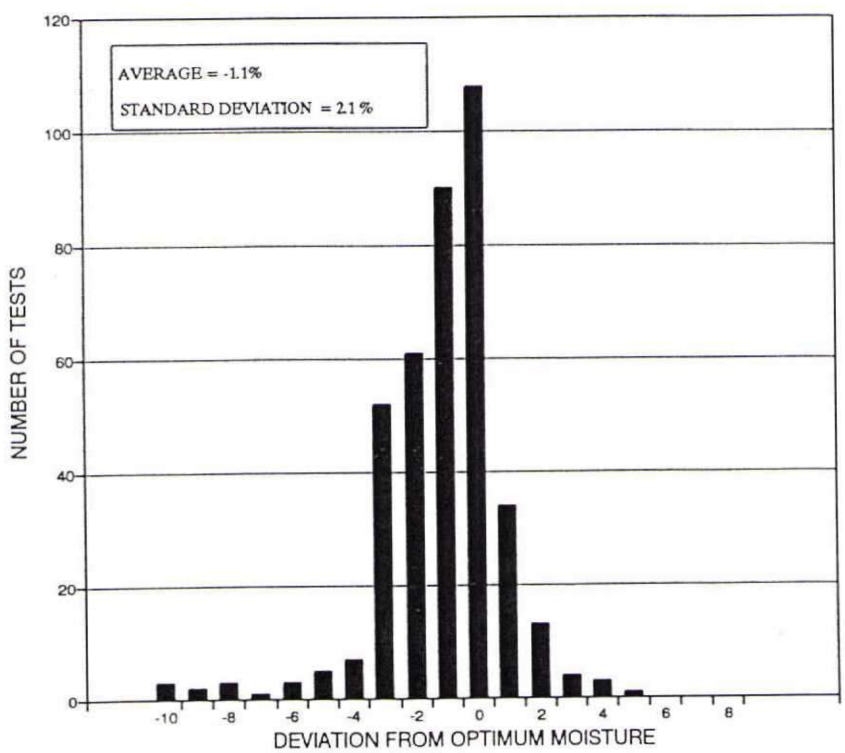


FIGURE IV-12. SD73 HOWES NORTH MOISTURE DISTRIBUTION

taken from clay and weathered shale ranging from low to high plasticity. The results of the Atterberg Limit tests showed soils with Liquid Limits ranging from the middle 20's to the low 70's. The average Liquid Limit was 52 percent. The Plasticity Indexes ranged from as low as 3 to as high as 50, with an average value of 31. In accordance with the AASHTO Classification System, the soils in the subgrade were generally A-7-6, however, a section of A-2-4 soil was encountered in the cut west of MRM 108.

The subgrade generally consisted of highly weathered shale and clay containing no shale fragments. The results of the compaction tests on the subgrade were compiled and analyzed statistically. The results show (Figure IV-13 and IV-14) that the average compaction achieved was over 100 percent of the maximum dry density as achieved by AASHTO T-99, and only four tests were below 95 percent. The standard deviation was 3.5 percent. The majority of the moisture content tests (Figure IV-14) ranged from 7 percent dry of optimum to 4 percent wet of optimum. Average deviation from optimum moisture content was 1.0 percent, and standard deviation from optimum was 3 percent.

Salvage asphalt was not used in the pavement, but was placed as part of the base course and in the shoulders. The asphalt cement used was 85/100 penetration grade, and was supplied from Little America Sinclair refinery in Wyoming. No chip

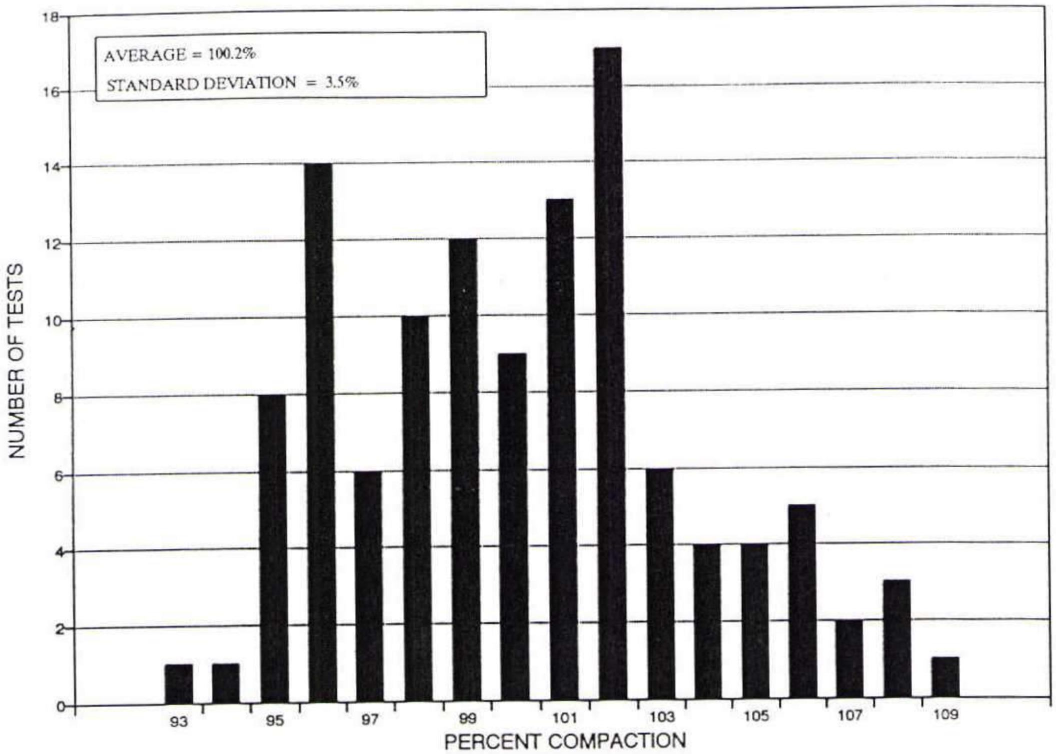


FIGURE IV-13. SD34 WHITE OWL TO HOWES DENSITY DISTRIBUTION

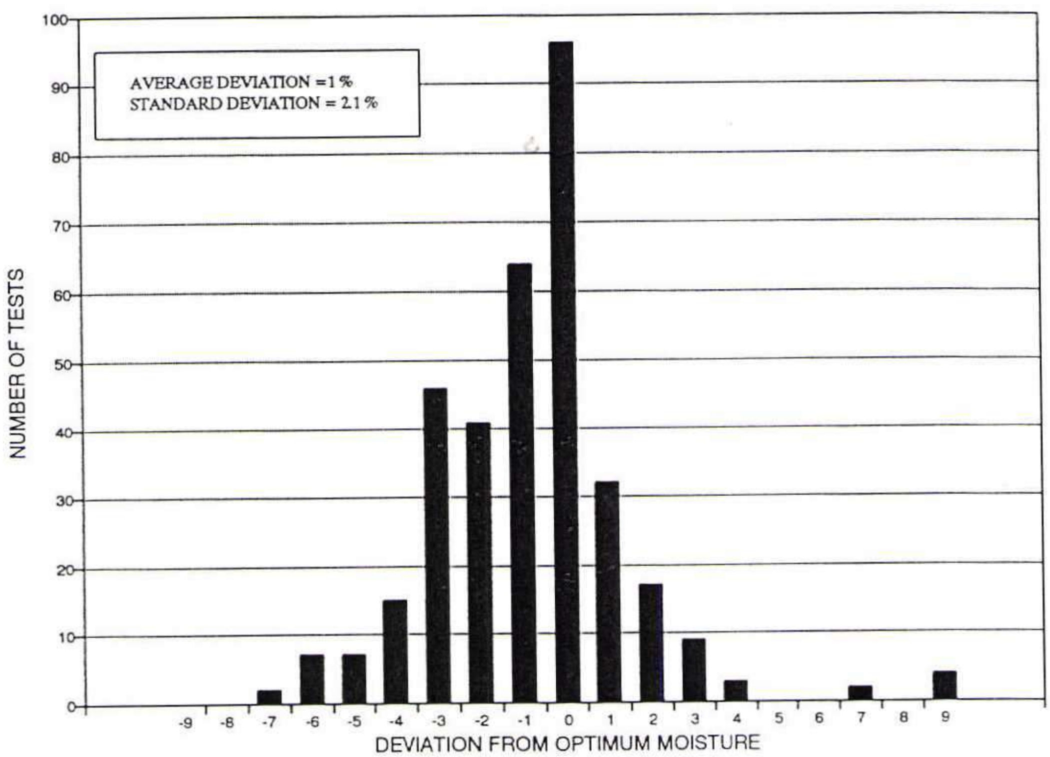


FIGURE IV-14. SD34 WHITE OWL TO HOWES MOISTURE DISTRIBUTION

sealing or crack sealing has yet been performed. The crack survey performed on the pavement in the fall of 1992 showed no cracks over the entire 15.6 miles. The survey was repeated in the spring of 1993, and showed that a total of four cracks had developed.

CHAPTER V

FIELD INVESTIGATION

V-1 INTRODUCTION

The field investigation consisted of performing measurements of horizontal and vertical movements, taking soil and asphalt samples, and digging a test pit on US212 between LaPlant and the Missouri river for the period between May 21, 1992, and April 20, 1993. The subsurface exploration was performed by crews from the SDDOT in Pierre, and traffic control was provided by SDDOT crews from Eagle Butte and Faith. A windshield survey of crack frequency was also performed. The project locations selected for investigation are listed in Table II-1. The specifics about the location of the control sections established for each project and the number of cracks surveyed per control section are presented in Table V-1.

The control sections from LaPlant to the Missouri River extend over three transverse cracks and are from 70 to 120 feet long. At the other project locations, the control sections extend over two cracks and have a minimum length of 50 feet.

The field investigation consisted of surveying lateral and vertical movements at and adjacent to the cracks, soil sampling at and adjacent to the cracks, asphalt coring, and digging of a test pit at the section MRM 189.4 on US Highway

TABLE V-1
PROJECT SPECIFICS

PROJECT LOCATION	MILEAGE REFERENCE MARKER	CUT OR FILL	NUMBER OF CRACKS
US212			
LaPlant to the Missouri River	204.2	CUT	3
	198.4	FILL	3
	189.7	CUT	3
	188.0	FILL	3
US212			
Ridgeview to LaPlant	184.2	FILL	1
	182.9	CUT	2
US212			
Ridgeview 12.1 Miles West	172.0	CUT	2
	167.3	FILL	2
SD73			
Faith 14.1 Miles South	170.0	CUT	2
	168.2	FILL	2
SD73			
Howes Corner 14.2 Miles North	160.1	CUT	2
	150.0	FILL	2
SD34			
Howes Corner to White Owl	109.4	CUT	0
	108.6	FILL	0

212 from LaPlant to the Missouri River. The details of the field investigation are discussed in the following paragraphs.

V-2 SURVEYING

Benchmarks consisting of PK(survey) nails were driven into the pavement between the cracks for reference. Observation points consisting of PK(survey) nails were driven into the pavement between the benchmarks. The observation points were placed at approximately one foot intervals up to a distance of five feet from the cracks, with a sixth nail placed at a distance of ten feet on each side of the crack. At the project from Howes Corner to White Owl, however, no cracks could be found, hence masonry nails were installed at 10 foot spacing at two locations for the measurement of vertical movements only.

V-2.1 LATERAL MEASUREMENTS

The horizontal distance between the observation points was taped to measure any lateral pavement movement. An increase in the distance between two observation points will be termed an expansion, while a decrease in the distance between two observation points will be termed a shrinkage. Baseline lateral measurements were taken on May 21, 1992. The changes in lateral movement versus time, elapsed time from the baseline measurement, were graphed for each crack observed. A total of thirteen observation intervals were measured per crack, however the graphs only depict the movement of the three observation intervals showing the greatest lateral movement. These three observation intervals are located immediately west of the crack, over the crack, and immediately east of the crack. The accuracy of the lateral measurements is considered to be plus or minus one millimeter.

V-2.1.1 US212 LaPlant To The Missouri River

The changes in lateral movement versus time for control sections at MRM 204.2, 198.4, 189.7, and 188 are shown in figures V-1 through V-12. The maximum lateral movements were detected on March 2, 1993, 285 days after the baseline measurements were taken. The lateral movements varied from a 6 millimeter shrinkage to a 14 millimeter expansion. The maximum lateral deformation was measured across the crack and was 11 millimeters in expansion. The average deformation between nails not spanning the cracks was two millimeters in shrinkage.

V-2.1.2 US212 Ridgeview To LaPlant

As shown on Figure V-13 through V-15, a maximum lateral movement of 12 millimeters across the crack was observed March 2, 1993, 285 days after the baseline measurements were taken. Over the period of the investigation, the lateral movements varied from a 2 millimeter shrinkage to a 13 millimeter expansion at MRM 182.9, with the largest movement occurring across the crack. The average maximum expansion was 12 millimeters.

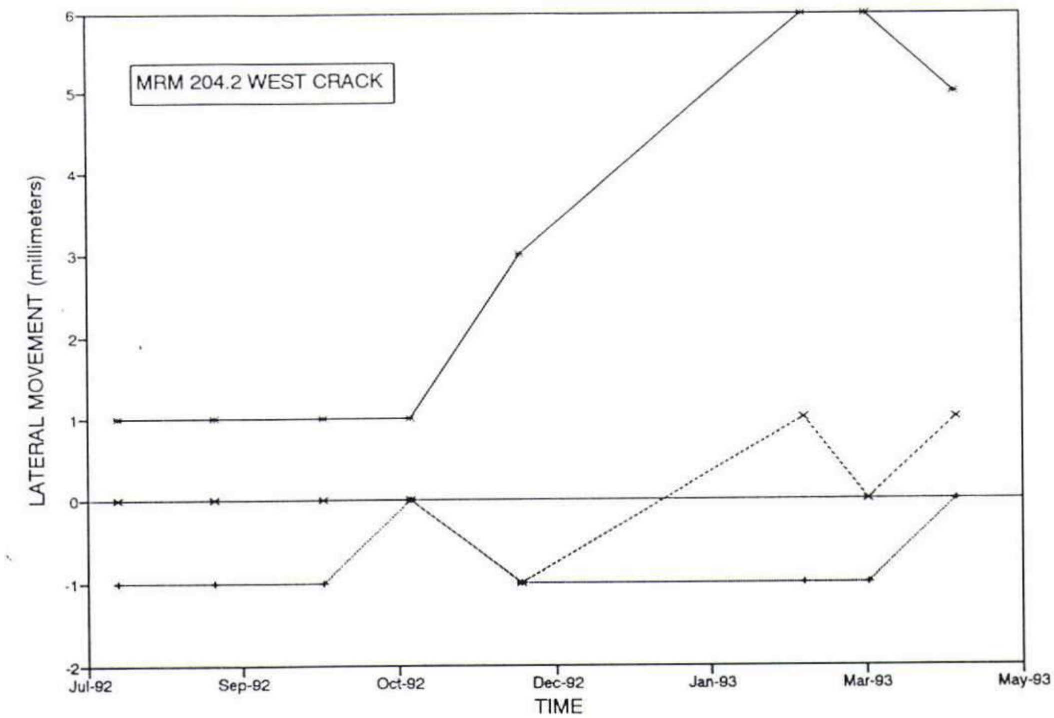


FIGURE V-1. US212 LAPLANT TO RIVER LATERAL MOVEMENT

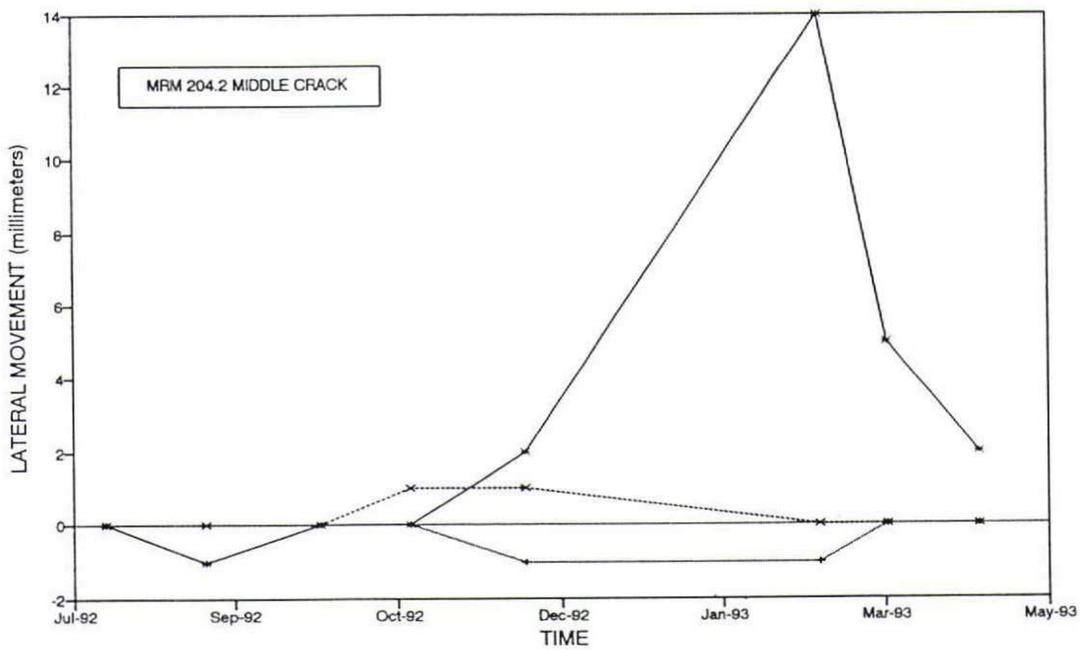


FIGURE V-2. US212 LAPLANT TO RIVER LATERAL MOVEMENT

West 1 pt — Over Crack — East 1 pt

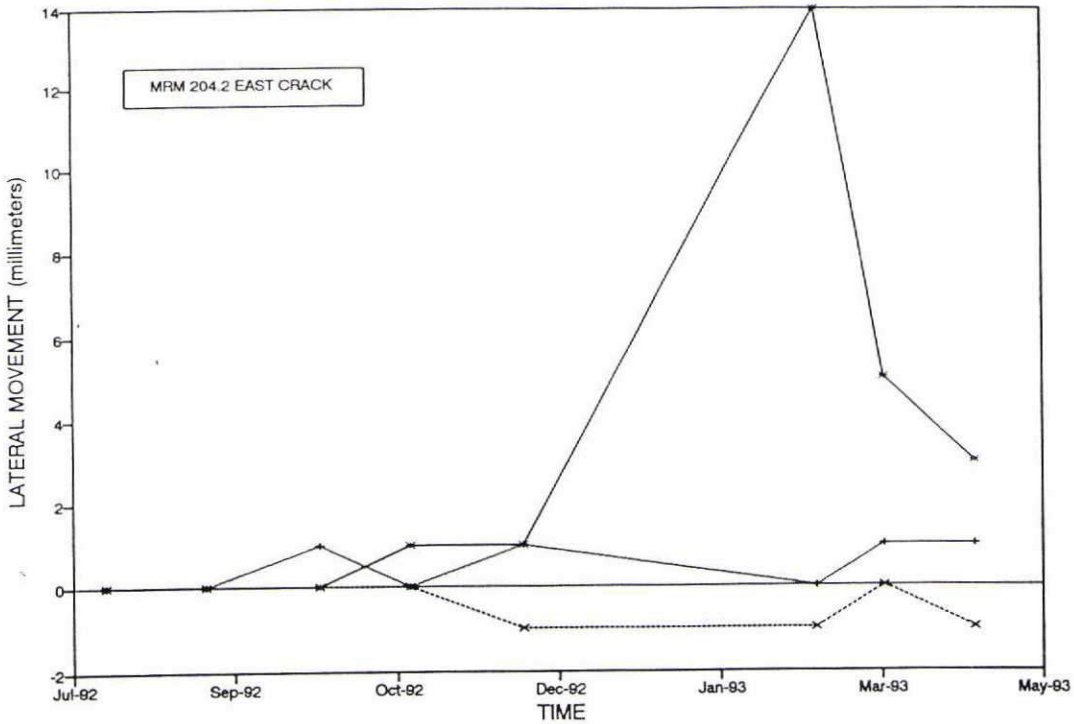


FIGURE V-3. US212 LAPLANT TO RIVER LATERAL MOVEMENT

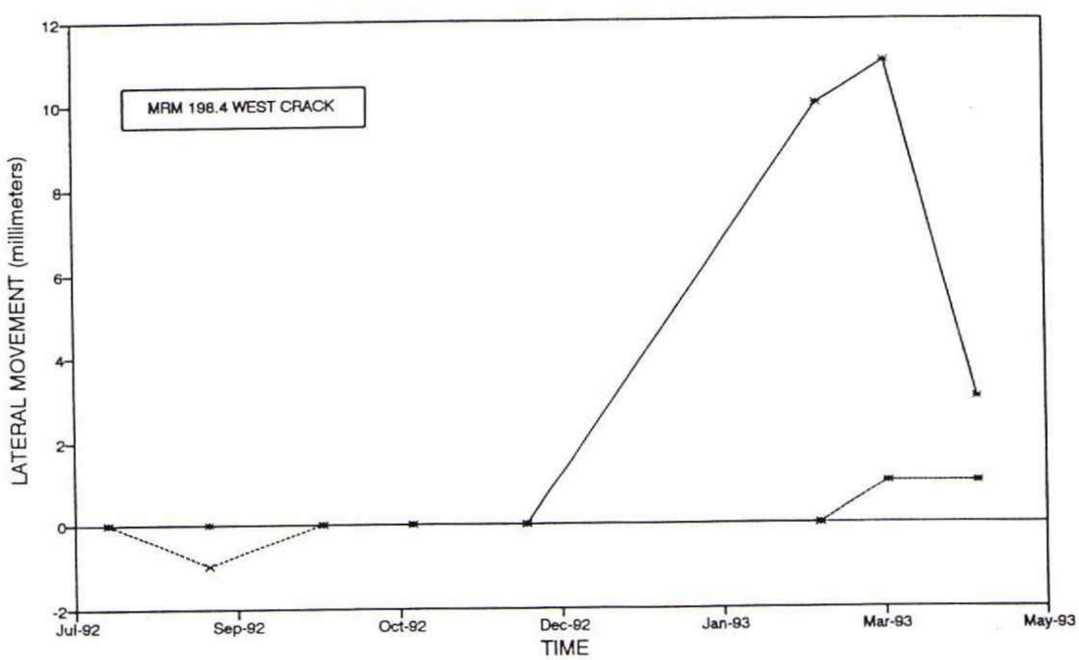


FIGURE V-4. US212 LAPLANT TO RIVER LATERAL MOVEMENT

West 1 pt — Over Crack — East 1 pt

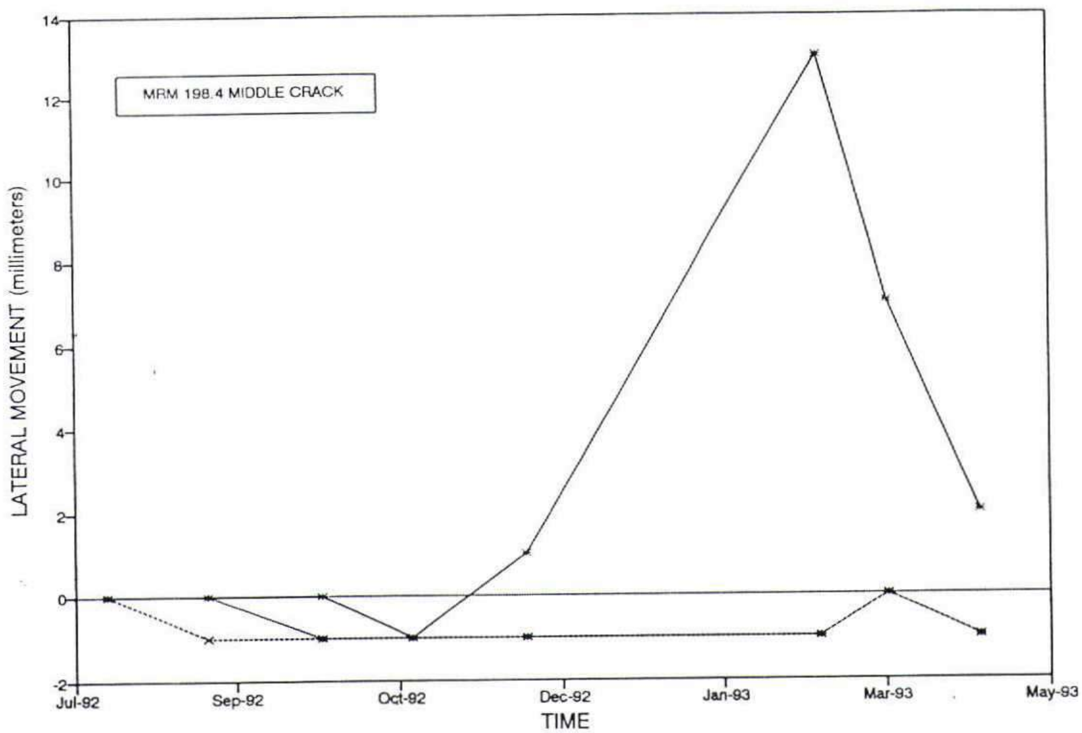


FIGURE V-5. US212 LAPLANT TO RIVER LATERAL MOVEMENT

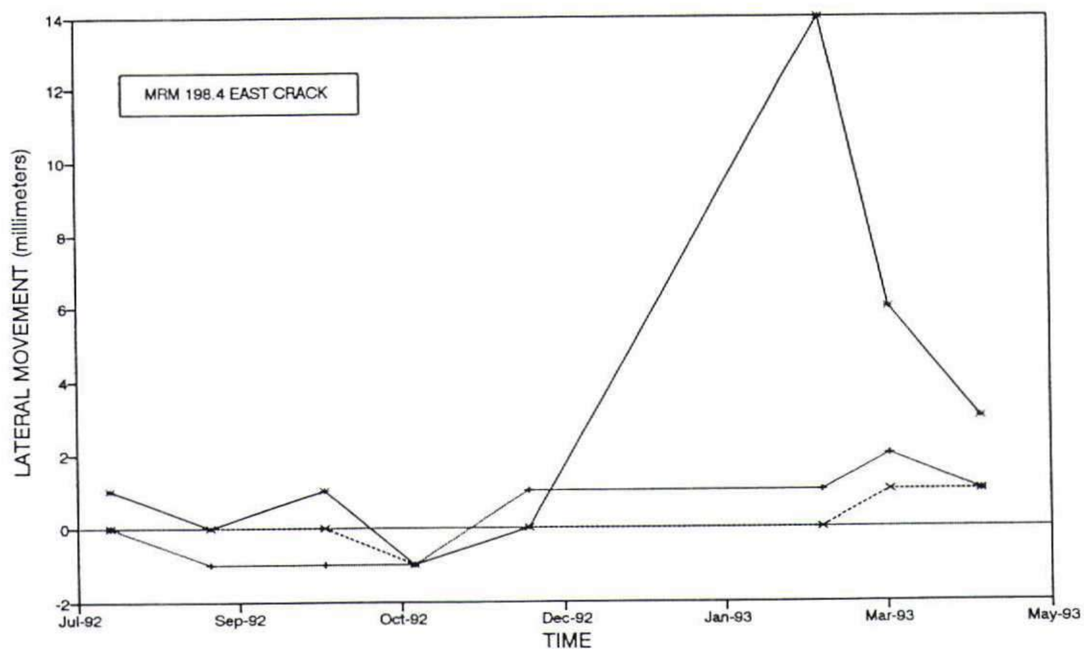


FIGURE V-6. US212 LAPLANT TO RIVER LATERAL MOVEMENT

West 1 pt — Over Crack — East 1 pt

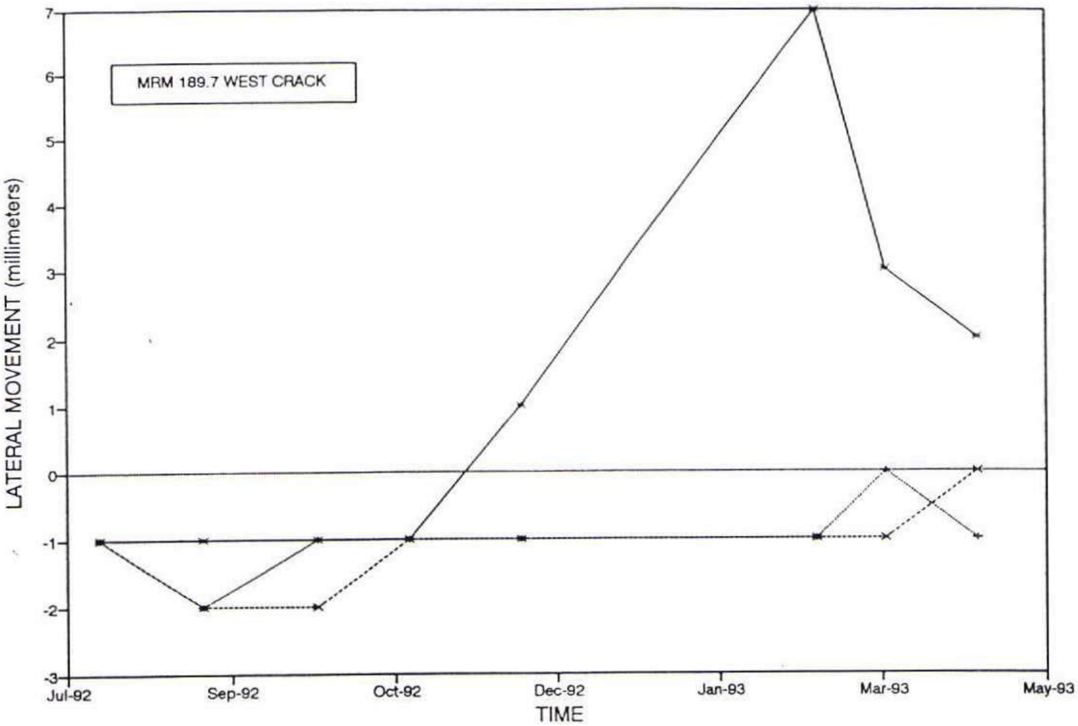


FIGURE V-7. US212 LAPLANT TO RIVER LATERAL MOVEMENT

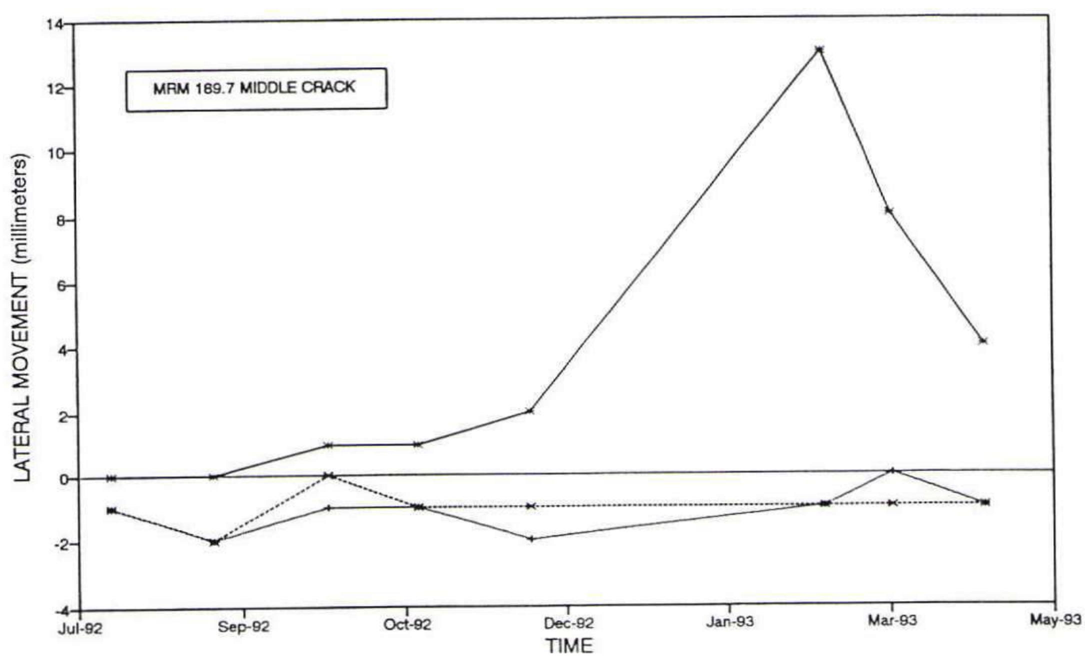


FIGURE V-8. US212 LAPLANT TO RIVER LATERAL MOVEMENT

West 1 pt — Over Crack — East 1 pt

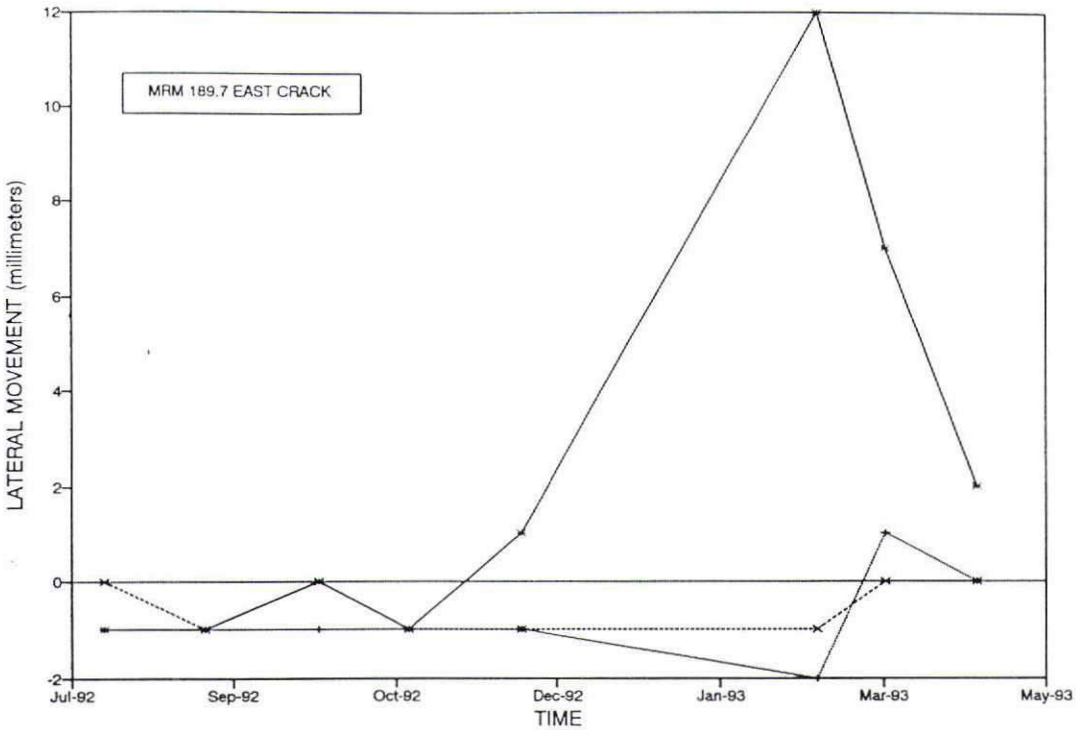


FIGURE V-9. US212 LAPLANT TO RIVER LATERAL MOVEMENT

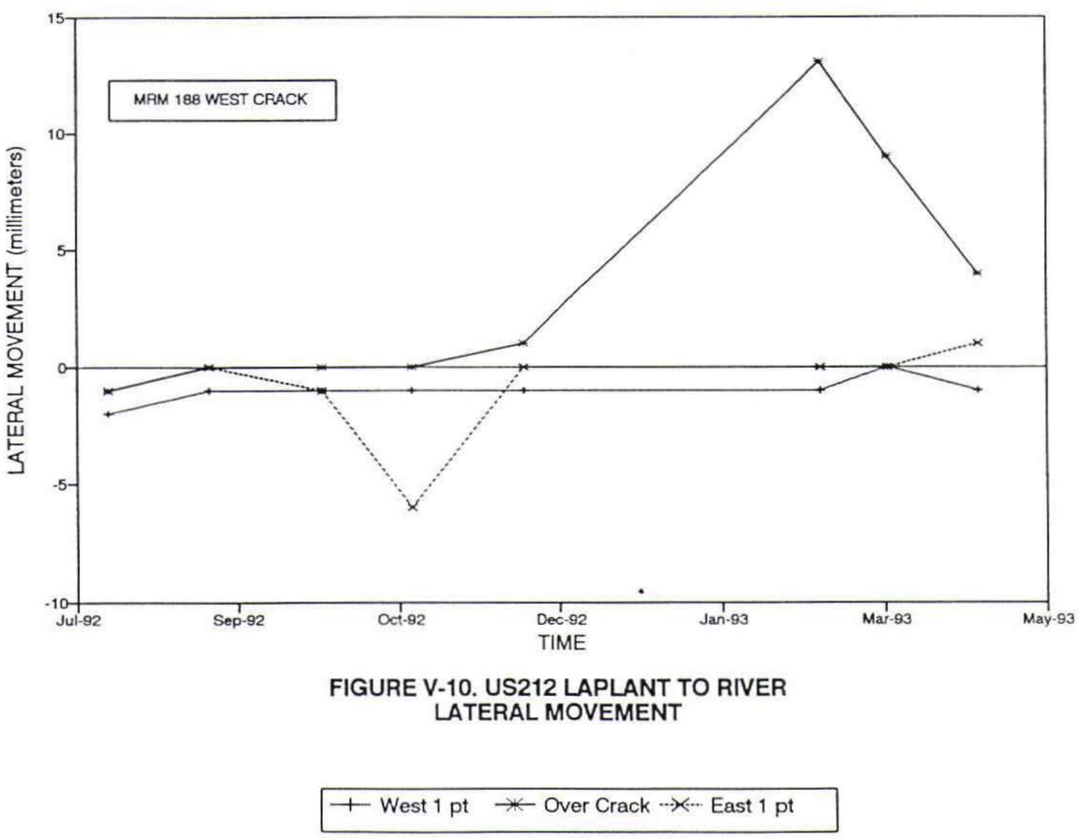


FIGURE V-10. US212 LAPLANT TO RIVER LATERAL MOVEMENT

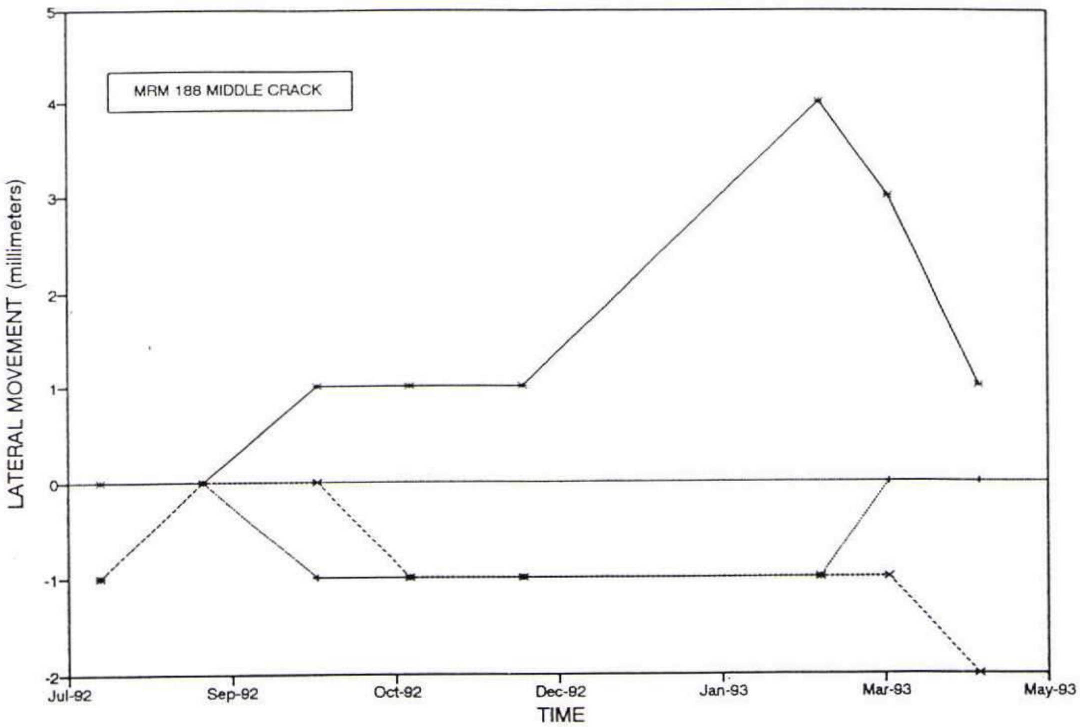


FIGURE V-11. US212 LAPLANT TO RIVER LATERAL MOVEMENT

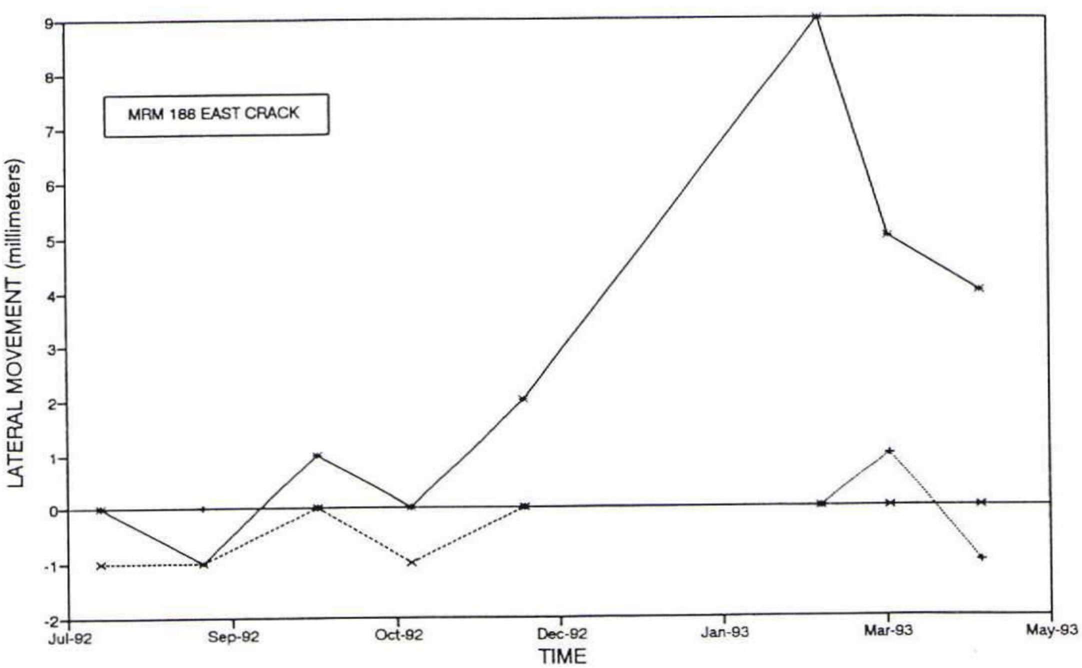
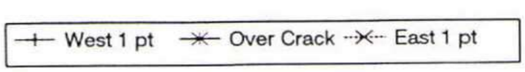


FIGURE V-12. US212 LAPLANT TO RIVER LATERAL MOVEMENT



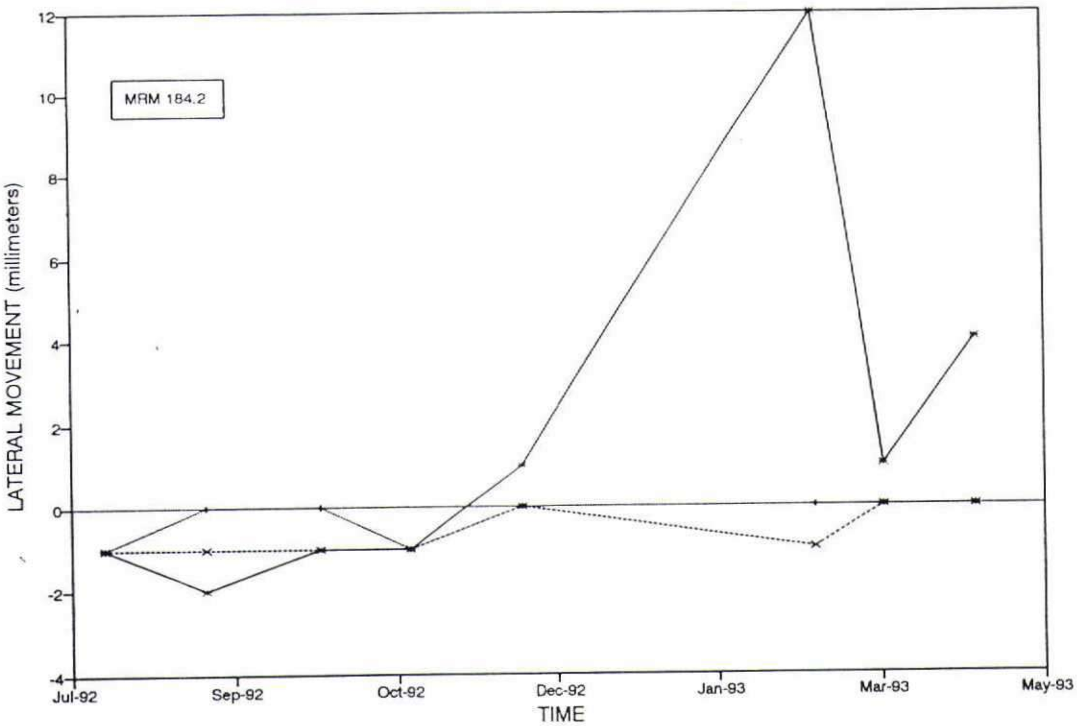


FIGURE V-13. US212 RIDGEVIEW TO LAPLANT LATERAL MOVEMENT

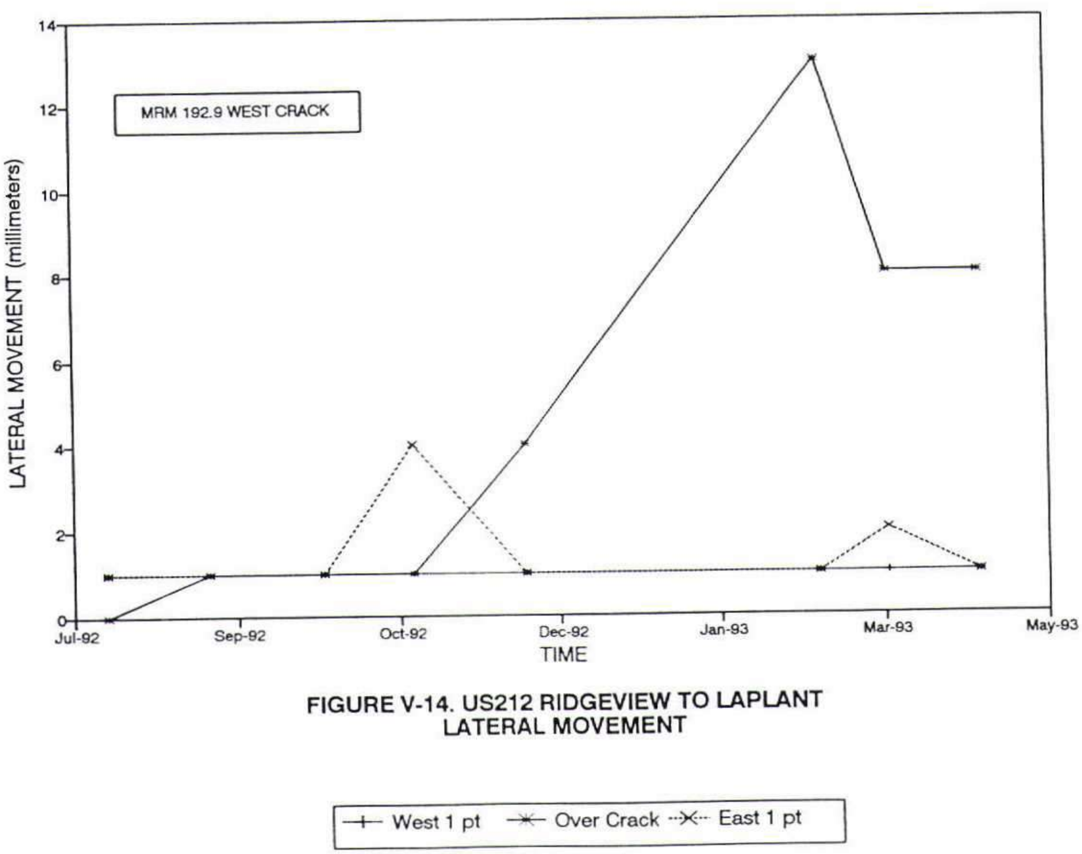


FIGURE V-14. US212 RIDGEVIEW TO LAPLANT LATERAL MOVEMENT

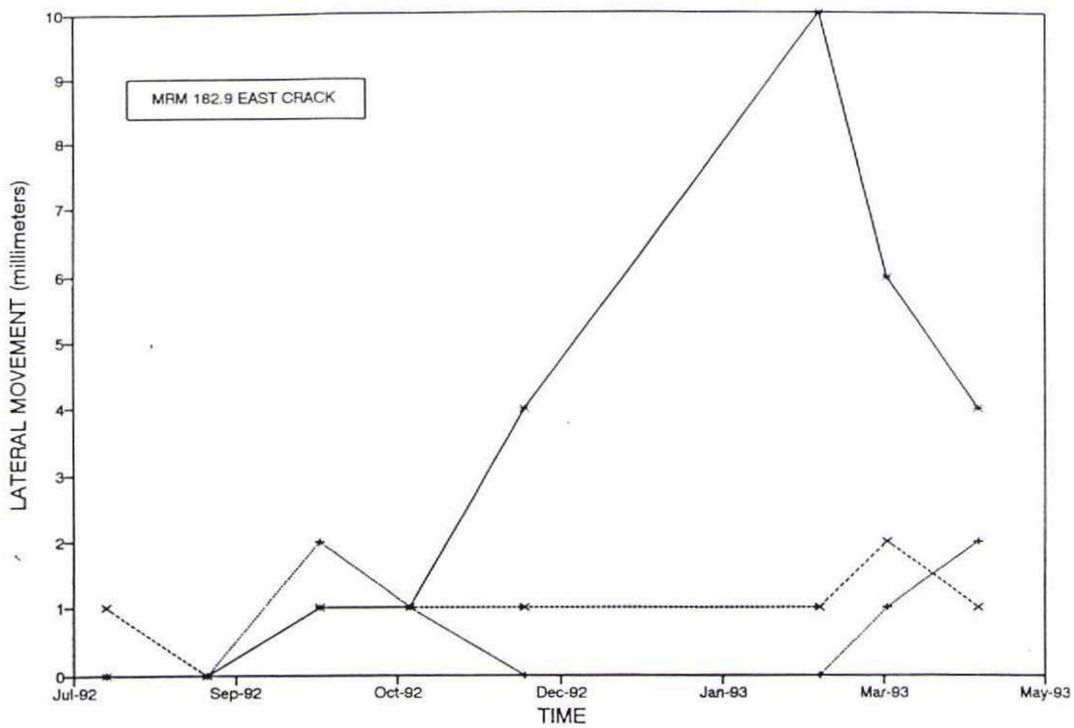


FIGURE V-15. US212 RIDGEVIEW TO LAPLANT LATERAL MOVEMENT

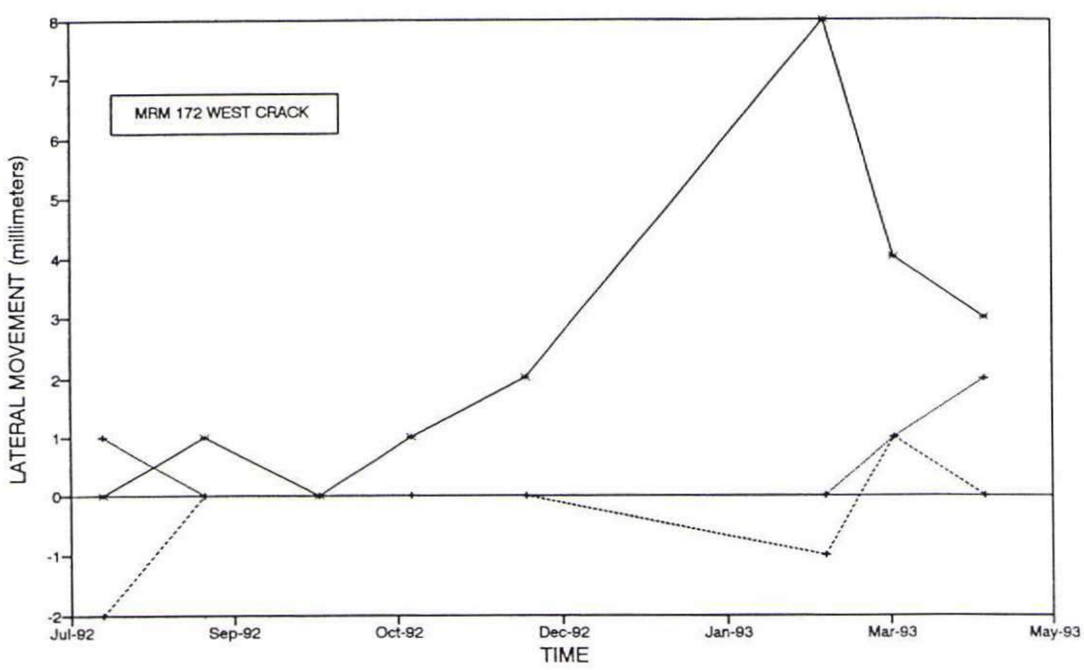


FIGURE V-16. US212 RIDGEVIEW WEST LATERAL MOVEMENT

West 1 pt | Over Crack | East 1 pt

V-2.1.3 US212 Ridgeview West 12.1 Miles

The maximum lateral movement was recorded on March 2, 1993, 285 days after the baseline measurements were taken. The lateral movements varied from a 2 millimeter shrinkage to a 10 millimeter expansion across the crack. The average minimum fluctuation was two millimeter in shrinkage, while the average maximum expansion was nine millimeters. The graphs representing the changes in lateral movement versus time for the control sections at MRM 172 and MRM 167.3 are shown in figures V-16 through V-19.

V-2.1.4 Highway 73 Faith South

The lateral movement versus time for the control sections at MRM 170 and MRM 168.2 are shown in figures V-20 - V-23. The maximum lateral movement was detected on March 2, 1993, 285 days after the baseline measurements were taken.

The lateral movements varied from a two millimeter shrinkage to a 14 millimeter expansion across the crack. The average minimum fluctuation was a two millimeter shrinkage, while the average maximum fluctuation was an 11 millimeter expansion.

V-2.1.5 Highway 73 Howes Corner North

The maximum lateral movement at the section between Howes Corner and 14.2 miles north was recorded on March 2, 1993, 285 days after the baseline measurements were taken. The lateral movements for control sections at MRM 160.1 and MRM

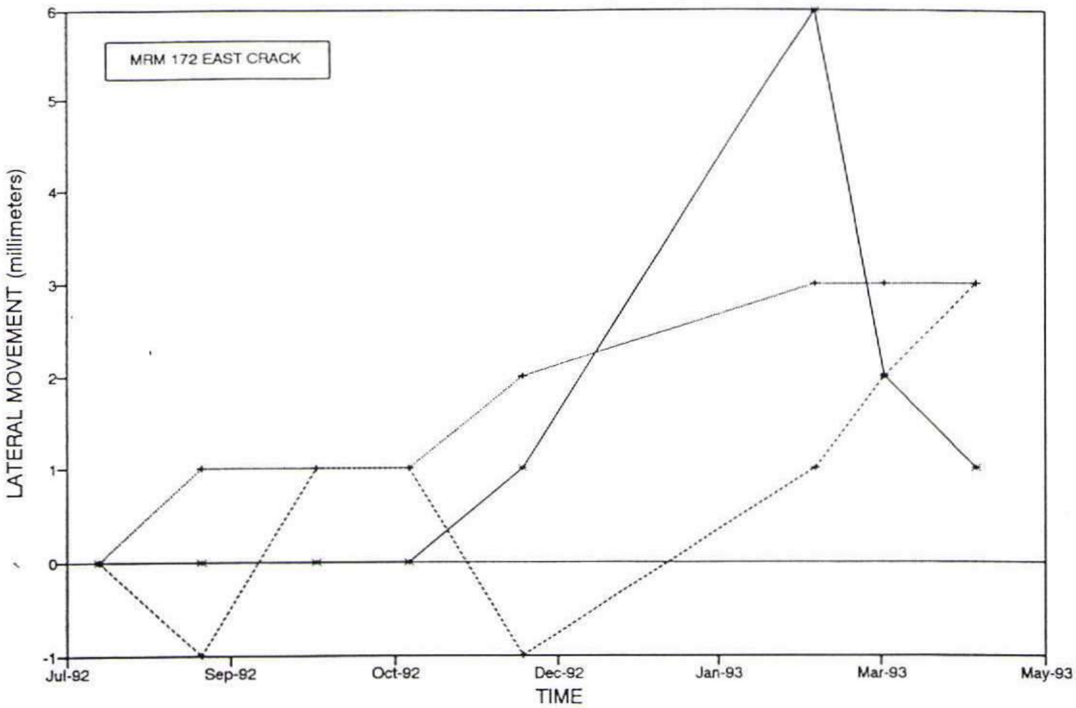


FIGURE V-17. US212 RIDGEVIEW WEST LATERAL MOVEMENT

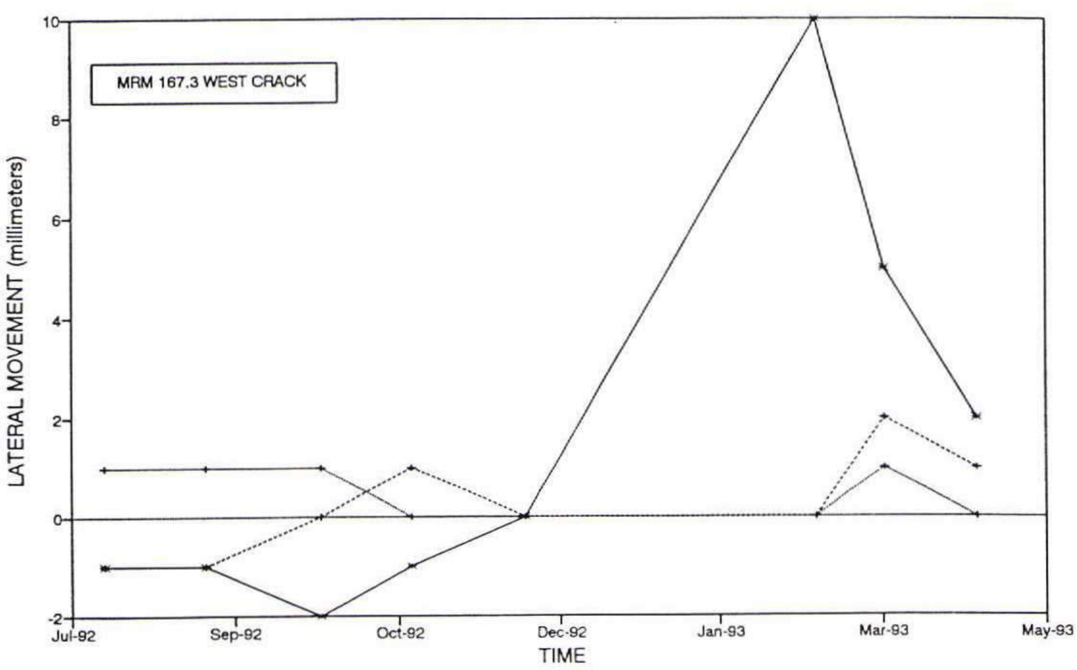


FIGURE V-18. US212 RIDGEVIEW WEST LATERAL MOVEMENT

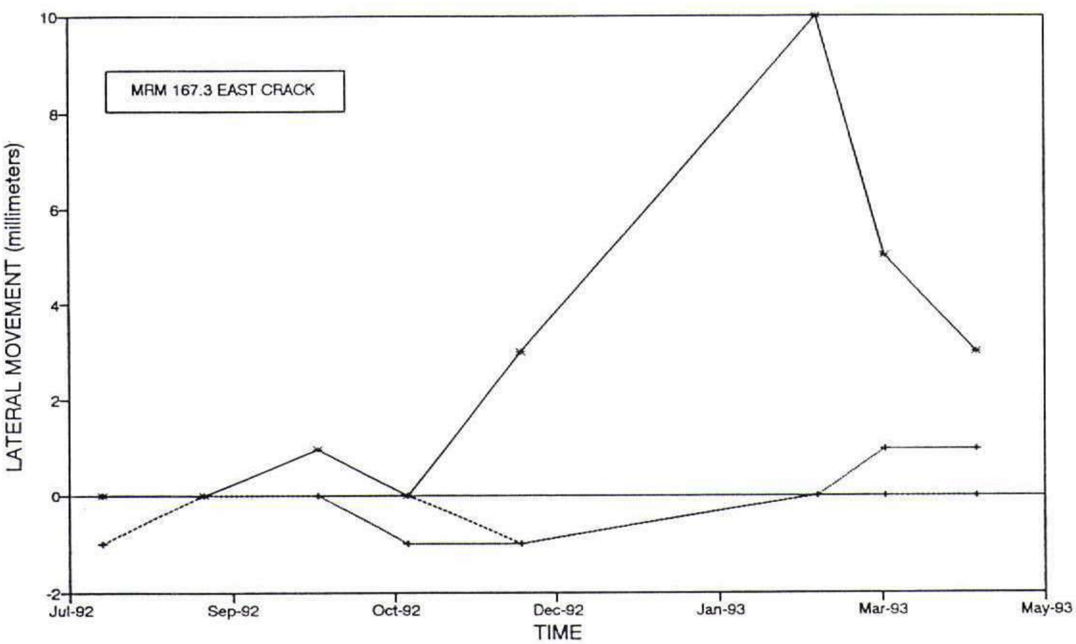


FIGURE V-19. US212 RIDGEVIEW WEST LATERAL MOVEMENT

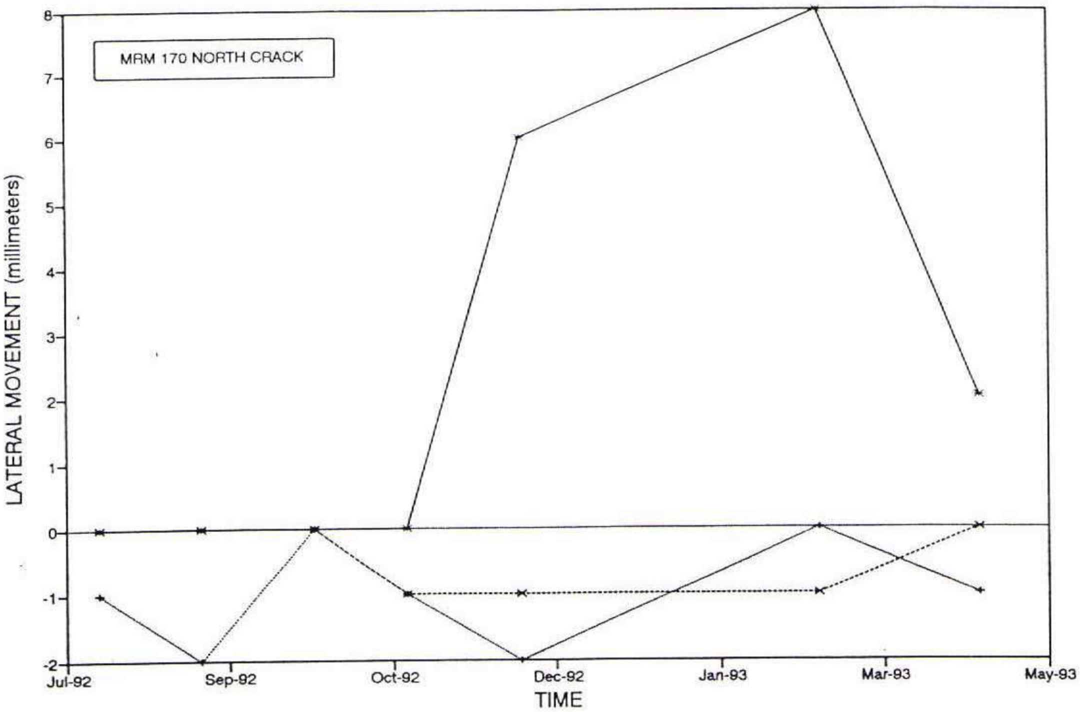


**FIGURE V-20. SD73 FAITH SOUTH
LATERAL MOVEMENT**

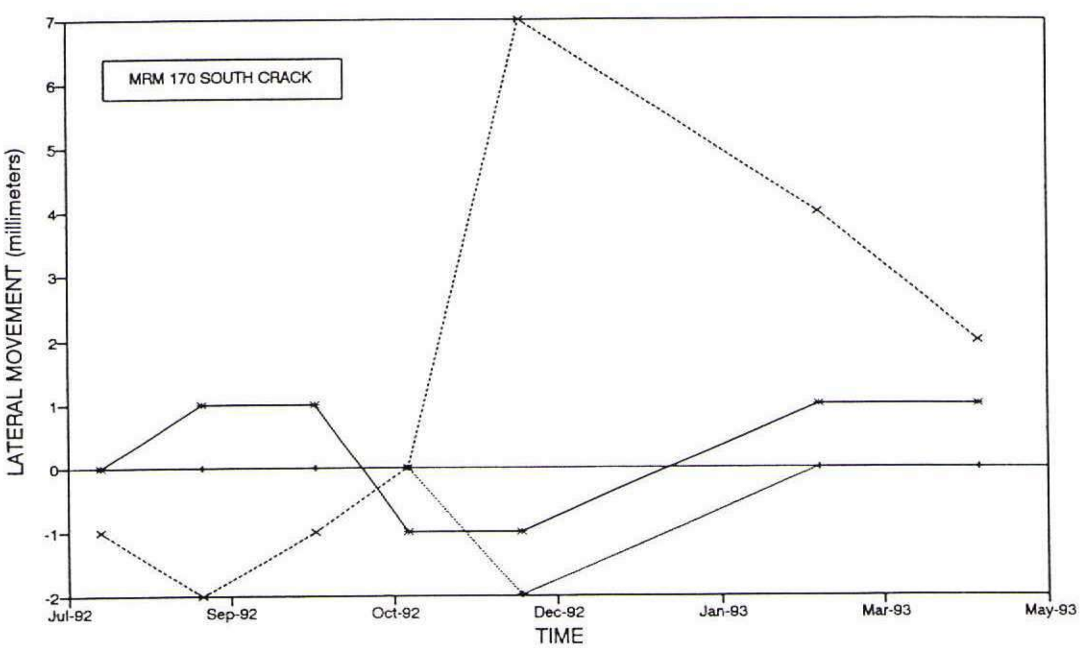


**FIGURE V-21. SD73 FAITH SOUTH
LATERAL MOVEMENT**

North 1 pt — Over Crack — South 1 pt

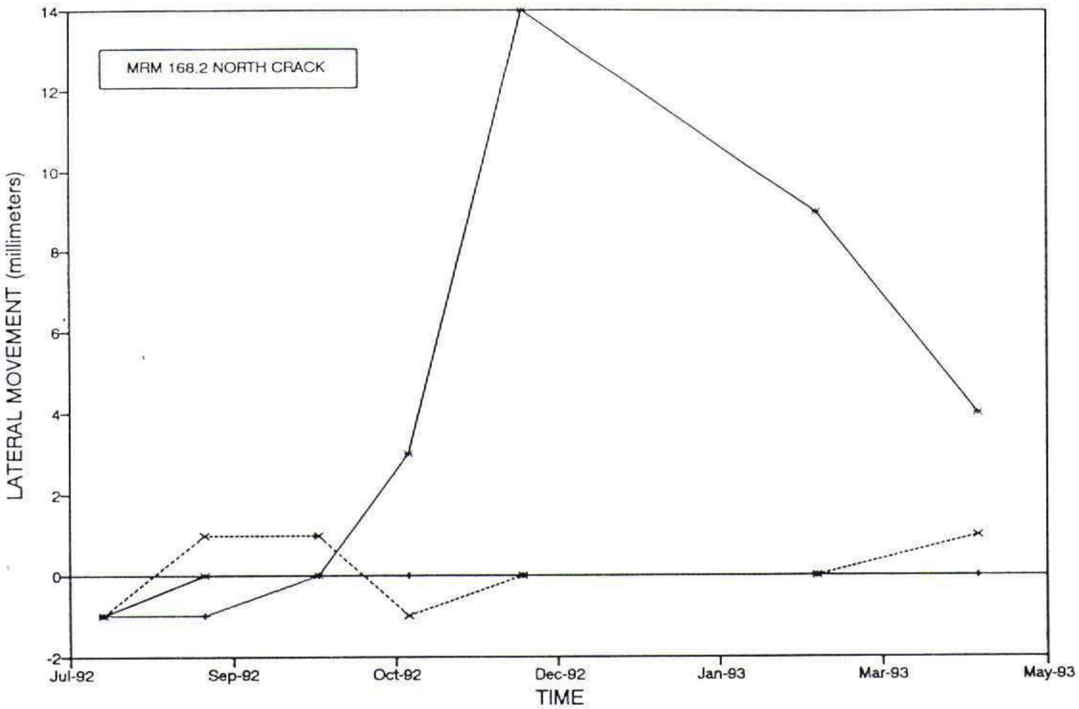


**FIGURE V-22. SD73 FAITH SOUTH
LATERAL MOVEMENT**

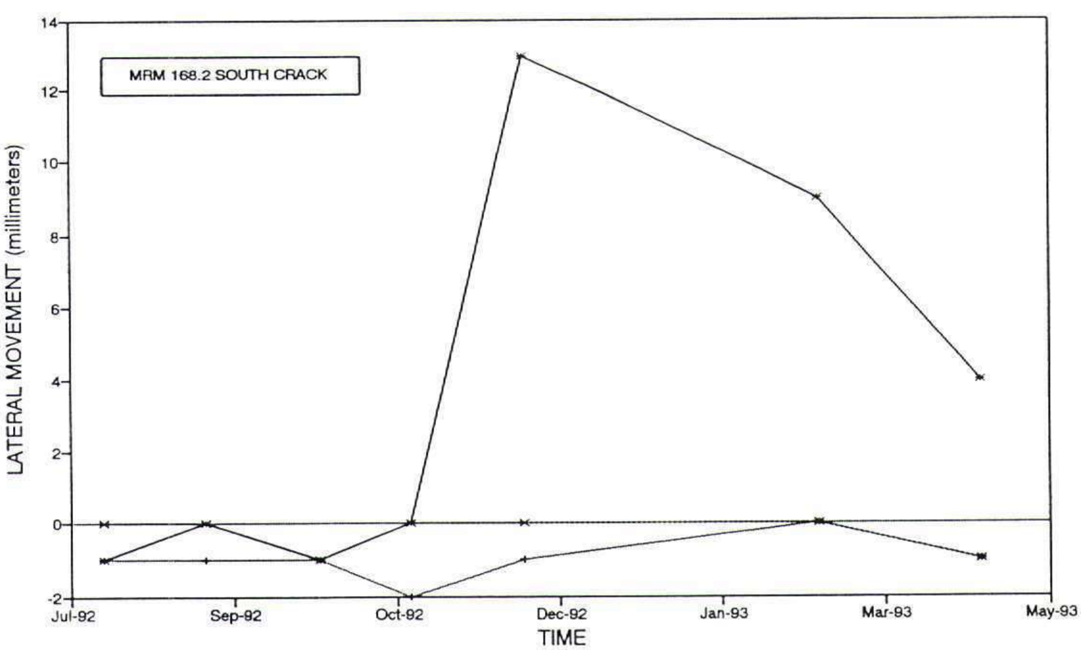


**FIGURE V-23. SD73 FAITH SOUTH
LATERAL MOVEMENT**

North 1 pt — Over Crack — South 1 pt

150 varied from a 2 millimeter shrinkage to a 13 millimeter expansion at the crack. The average minimum fluctuation was a two millimeter shrinkage, while the average maximum fluctuation was an 11 millimeter expansion. The changes in lateral movement versus time are shown in figures V-24 - V-27.

V-2.1.6 Highway 34 White Owl To Howes Corner

The visual survey performed on the 16.6 mile project between Howes corner and White Owl revealed no cracks. As a result, the distances between the observation points were not measured for this project.

V-2.2 VERTICAL MEASUREMENTS

The pavement sections between the benchmarks were surveyed for changes in vertical elevation at the observation points to the nearest .002 foot. Baseline elevations were taken on May 21, 1992, except for the project on SD Highway 34, as discussions on how to monitor this section were continued. As a careful crack survey on foot revealed no cracks on the entire project, it was decided to establish baseline elevations for 100 foot long sections at MRM 109.4 and MRM 108.6. Along these sections, PK(survey) nails were driven at approximately 10 foot spacing on July 25, 1992. Permanent vertical reference points were not considered necessary, since differential vertical movements along the pavement were felt to be of greatest importance.

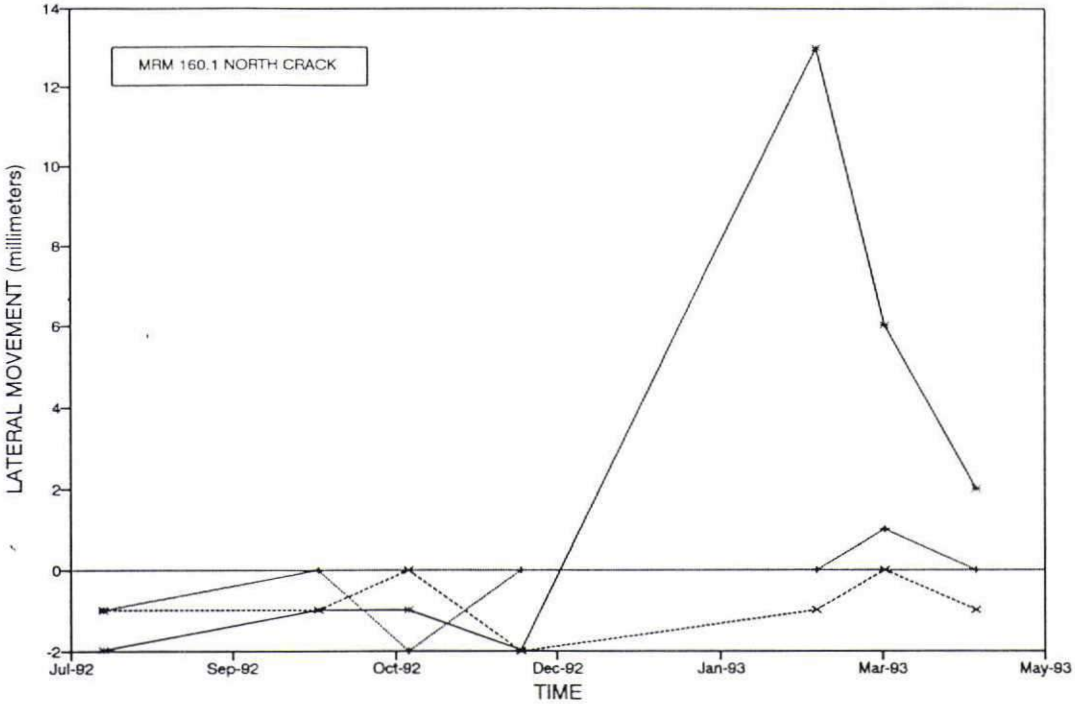


FIGURE V-24. SD73 HOWES NORTH LATERAL MOVEMENT

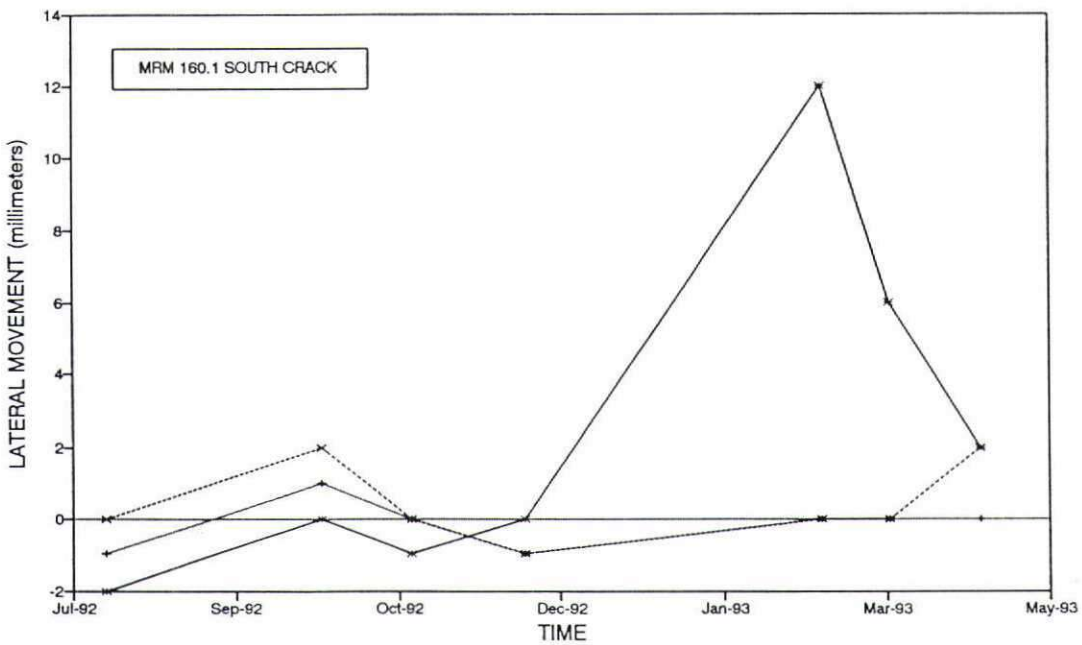


FIGURE V-25. SD73 HOWES NORTH LATERAL MOVEMENT

North 1 pt   Over Crack   South 1 pt

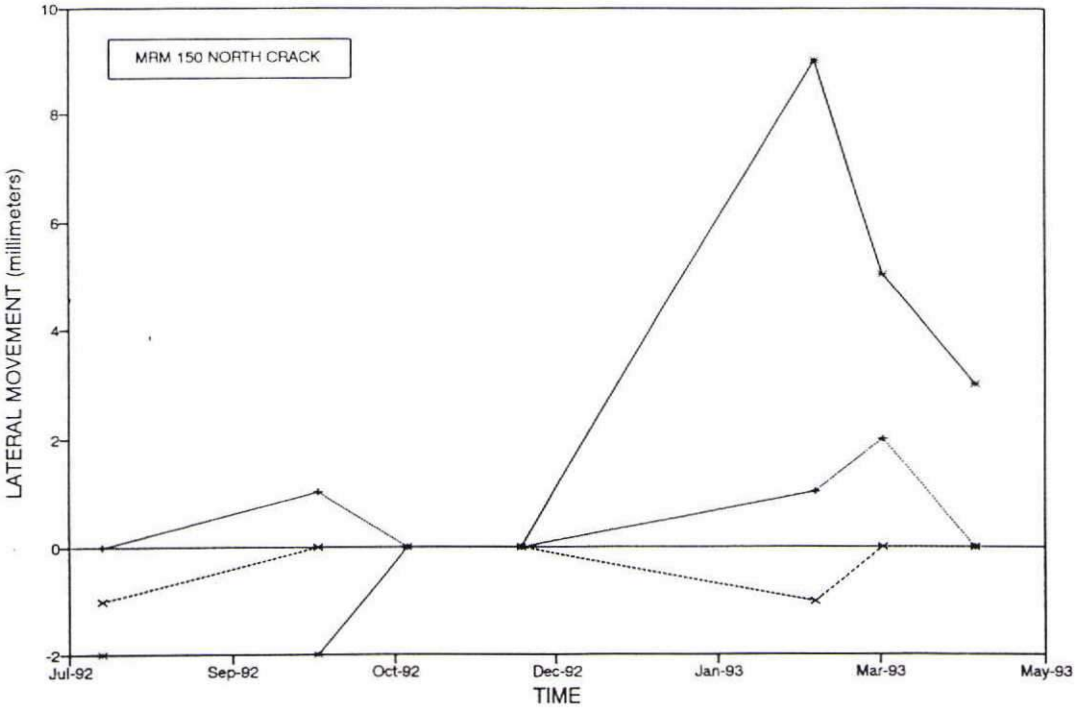


FIGURE V-26. SD73 HOWES NORTH LATERAL MOVEMENT

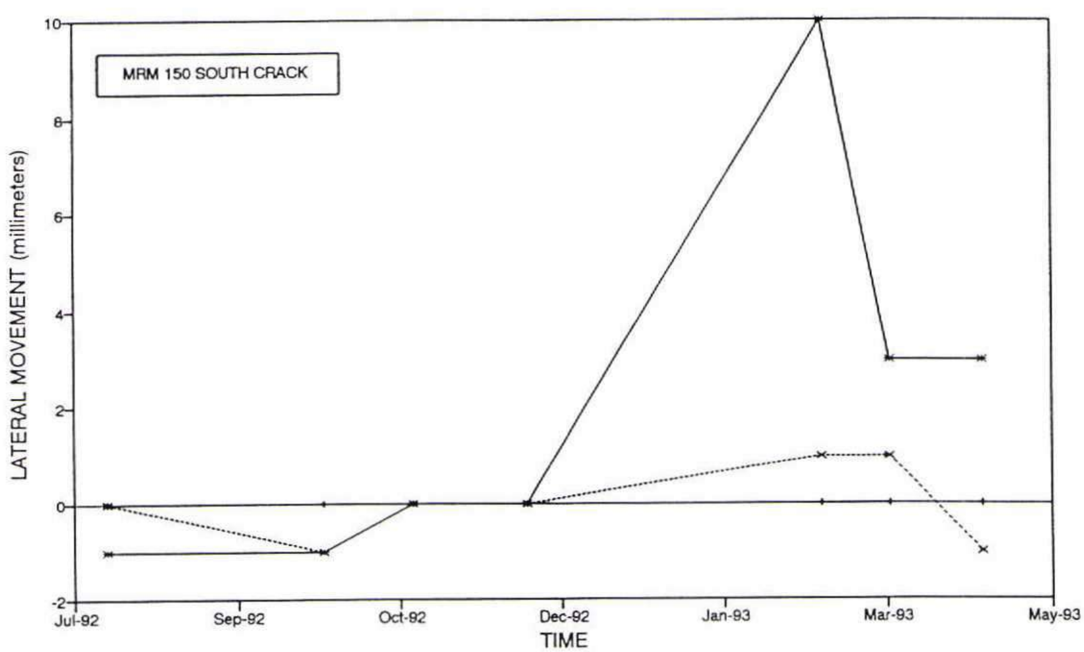


FIGURE V-27. SD73 HOWES NORTH LATERAL MOVEMENT

North 1 pt — Over Crack — South 1 pt

Changes in vertical movement versus elapsed time from the baseline elevation measurement, were graphed at each crack location observed. Elevations of 14 observation points were taken per crack, however, since the plotting of all points resulted in too much clutter, the graphs only depict the movement of six observation points. The six observation points were located at ten and five feet on either side of the crack and immediately next to the crack and are considered representative of the movement of the other eight observation points. Due to the massive amount of data, elevation changes at only one crack at each of the control sections are shown. Also, movements at every crack are not included, and only graphs showing typical results are presented.

V-2.2.1 US212 LaPlant to the Missouri River

Typical vertical movements versus time for the control sections at MRM 204.2, 198.4, 189.7, and 188 are shown in figures V-28 - V-31. The maximum vertical movements for control sections 204.2, 198.4, 189.7, and 188 are summarized in Table V-2.

TABLE V-2. MAXIMUM VERTICAL MOVEMENT

CONTROL SECTION	MAXIMUM MEASUREMENT (feet)	DATE	ELAPSED TIME (days)
204.2	0.042	March 23	306
198.4	0.092	March 2	285
189.7	0.075	April 20	334
188.0	0.082	August 25	96

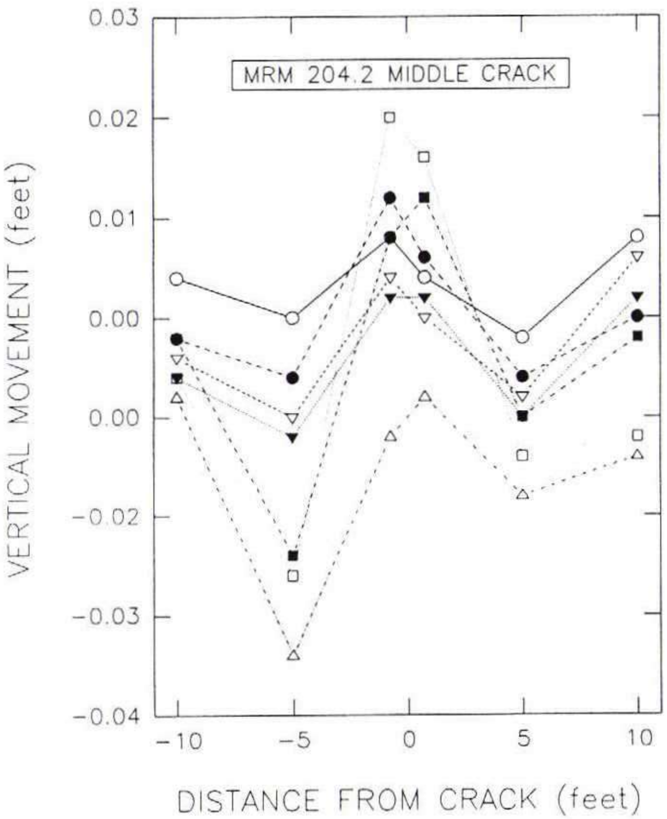


FIGURE V-28. US212 LAPLANT TO RIVER
VERTICAL MOVEMENT

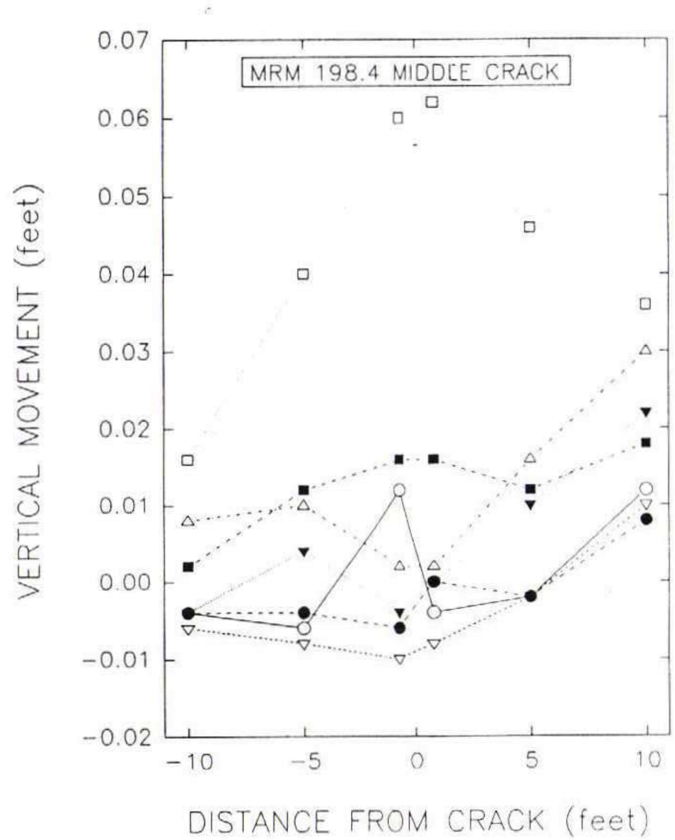


FIGURE V-29. US212 LAPLANT TO RIVER
VERTICAL MOVEMENT

ELAPSED TIME FROM ORIGINAL MEASUREMENT

○ 65 DAYS 7/25/92	● 96 DAYS 8/25/92	▽ 131 DAYS 9/29/92	▼ 159 DAYS 10/27/92	□ 285 DAYS 3/2/93	■ 306 DAYS 3/23/93	△ 334 DAYS 4/20/93

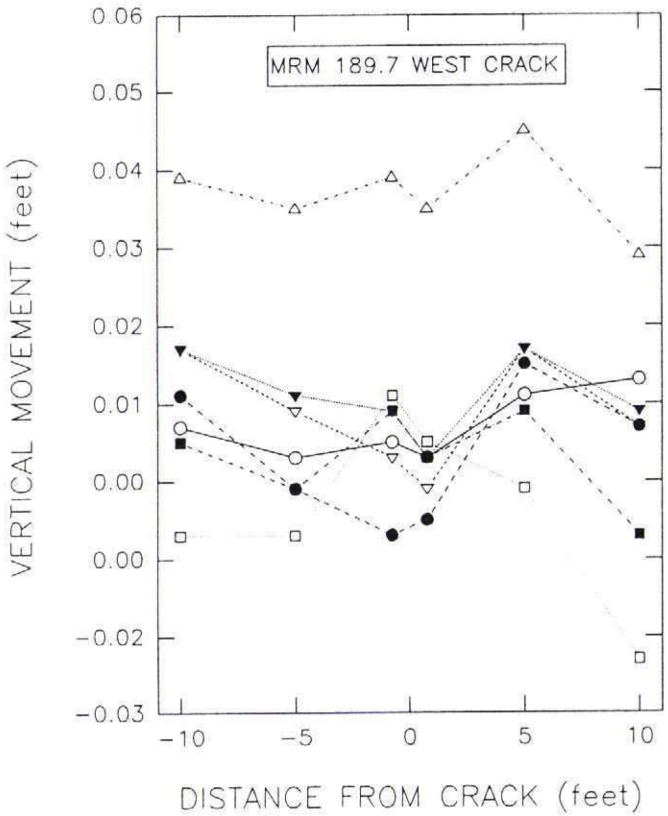


FIGURE V-30. US212 LAPLANT TO RIVER VERTICAL MOVEMENT

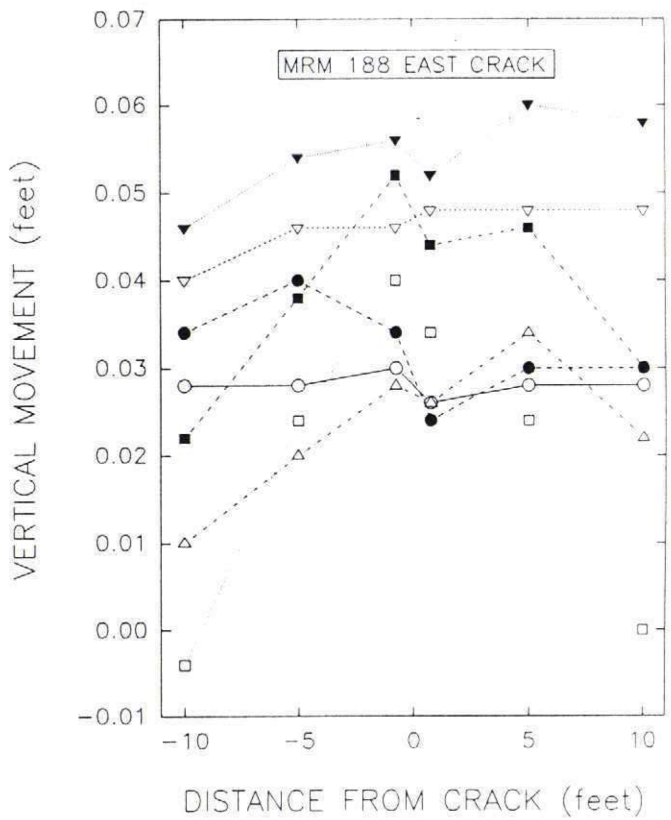


FIGURE V-31. US212 LAPLANT TO RIVER VERTICAL MOVEMENT

ELAPSED TIME FROM ORIGINAL MEASUREMENT						
○ 65 DAYS	● 96 DAYS	▽ 131 DAYS	▼ 159 DAYS	□ 285 DAYS	■ 306 DAYS	△ 334 DAYS
7/25/92	8/25/92	9/29/92	10/27/92	3/2/93	3/23/93	4/20/93

The maximum vertical movements generally occurred at the location nearest to the crack, however, significant movements are also taking place farther away from the crack.

V-2.2.2 US212 Ridgeview to LaPlant

The majority of the maximum vertical movements in control sections MRM 184.2 and 182.9 were detected between 285 and 306 days after the baseline elevations were established. The maximum vertical movement of 0.072 feet, for the control section at MRM 184.2, was recorded on April 20, 1993, while for the control section at MRM 182.9, a maximum vertical movement of 0.046 was recorded on March 23, 1993. The vertical movements near the crack are consistently greater than five feet from the crack. Typical changes in vertical movement versus time for the control sections at MRM 184.2 and 182.9 are presented in Figure V-32.

V-2.2.3 US212 Ridgeview West 12.1 Miles

Typical vertical movement versus time for the control sections at MRM 172 and 167.3 are shown in Figure V-33. At the control section at MRM 172, a maximum vertical movement of 0.038 feet, was measured on September 29, 1992 and the maximum vertical movement of 0.046 feet, for control section 182.9, was observed on March 2, 1993. As on the previous project, the vertical movements near the crack are slightly greater than five feet from the crack.

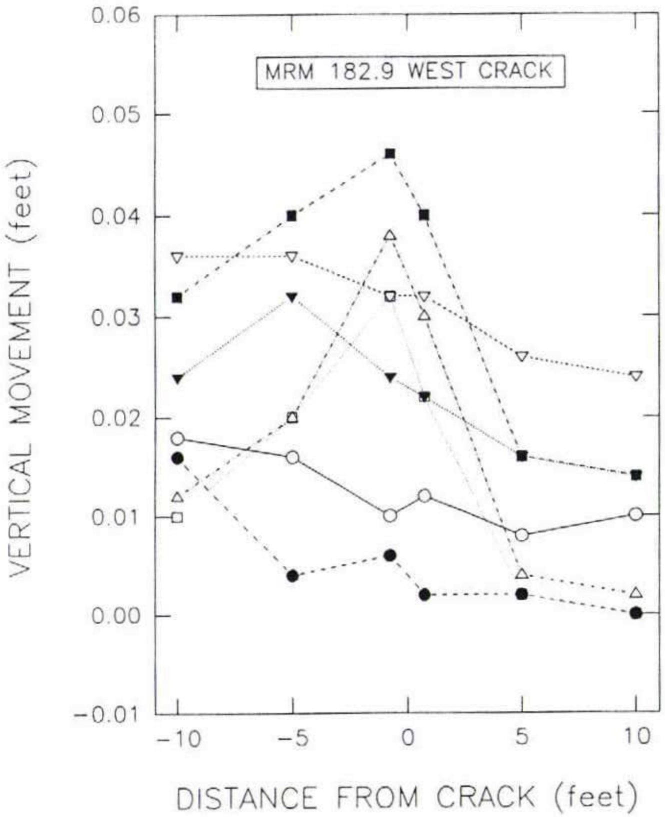


FIGURE V-32. US212 RIDGEVIEW-LAPLANT VERTICAL MOVEMENT

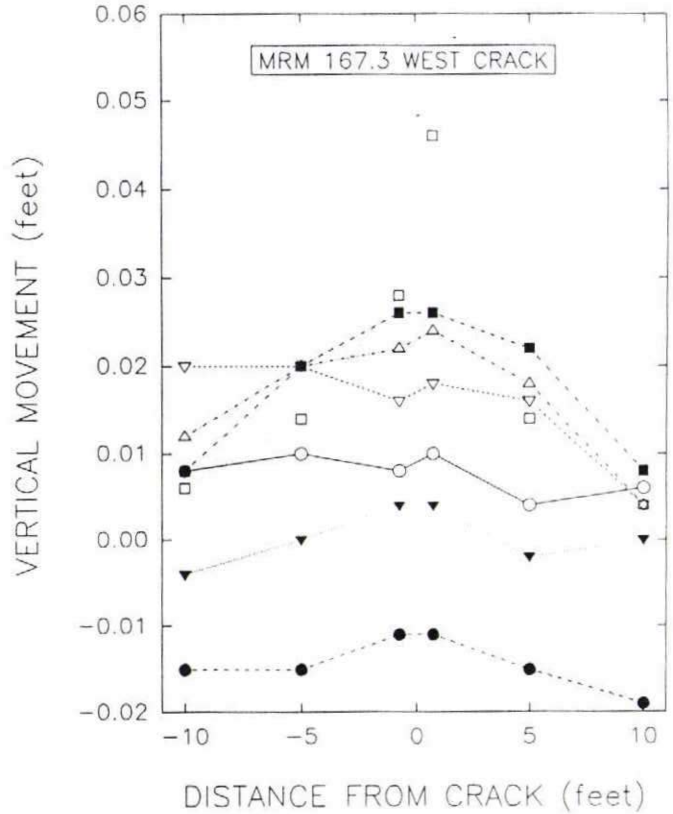


FIGURE V-33. US212 RIDGEVIEW WEST VERTICAL MOVEMENT

ELAPSED TIME FROM ORIGINAL MEASUREMENT

○ 65 DAYS	● 96 DAYS	▽ 131 DAYS	▼ 159 DAYS	□ 285 DAYS	■ 306 DAYS	△ 334 DAYS
7/25/92	8/25/92	9/29/92	10/27/92	3/2/93	3/23/93	4/20/93

V-2.2.4 Highway 73 Faith South

The maximum vertical movement for control section 170 was recorded only 65 days after the baseline elevations were established. During this elapsed time period SD Highway 73 was chip sealed. Subsequent measurements indicate a gradual heave, reaching a maximum of 0.074 feet at MRP 170 on March 2, 1993 (elapsed time of 285 days from the baseline elevation measurements), and 220 days after the elevation measurements were affected by the chip sealing.

The chip sealing had a non detectable effect on the control section at MRM 168.2, although at this location, the maximum vertical movements were recorded 285 days after the baseline elevations were established, which follows the pattern of control sections located on US212 which were not chipsealed. A maximum vertical movement of 0.056 feet, for control section at MRM 168.2, was detected on March 23, 1993.

The vertical movements near the crack for the control sections at MRM 170 and 168.2 are consistently greater than five feet from the crack. Typical changes in vertical movement versus time at these locations are shown in Figure V-34.

V-2.2.5 Highway 73 Howes Corner North

The chip sealing also appeared to affect the vertical movements at this project, as settlements of up to 0.050 feet were evident in the elevations taken up to March 2,

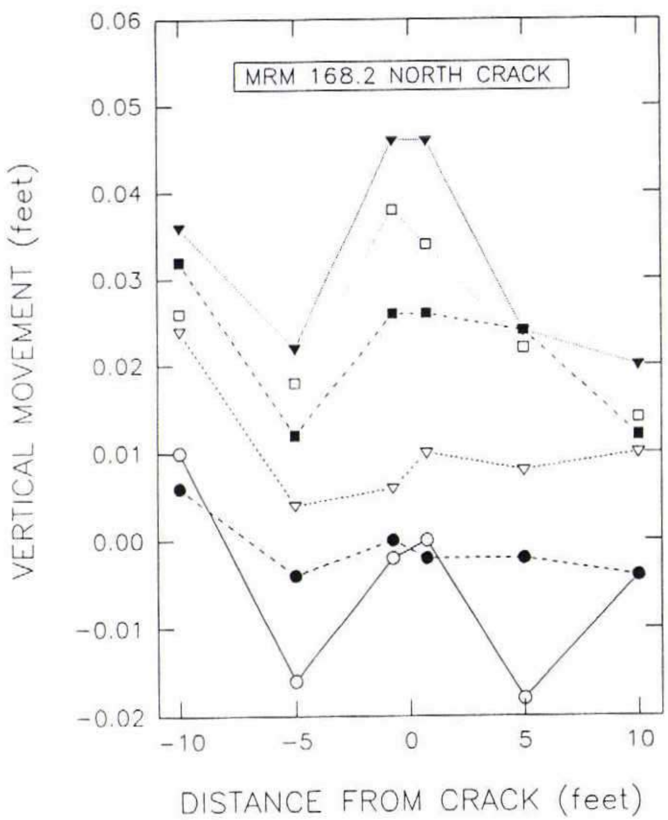


FIGURE V-34. SD73 FAITH SOUTH
VERTICAL MOVEMENT

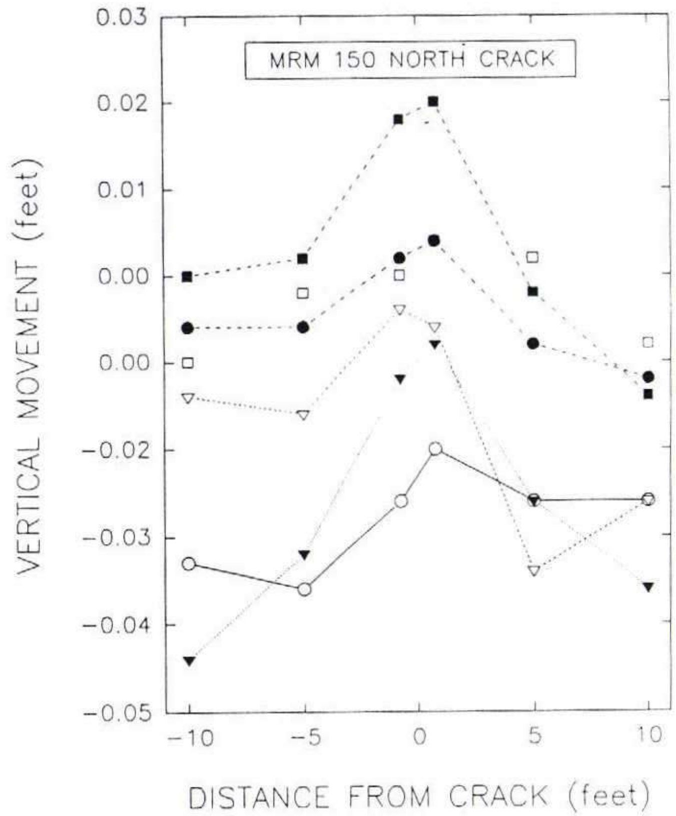


FIGURE V-35. SD73 HOWES NORTH
VERTICAL MOVEMENT

ELAPSED TIME FROM ORIGINAL MEASUREMENT

○ 65 DAYS	● 131 DAYS	▽ 159 DAYS	▼ 285 DAYS	□ 306 DAYS	■ 334DAYS
7/25/92	9/29/92	10/27/92	3/2/93	3/23/93	4/20/93

1993. However heaves were recorded after March 23, 1993. A heave of 0.028 feet, for the control section at MRM 160.1, was detected on April 20, 1993 and a maximum heave of 0.060 feet, for control section 150, was detected on March 23, 1993. Vertical movements near the crack are consistently greater than five feet from the crack. Typical vertical movements versus time are shown in Figure V-35.

V-2.2.6 Highway 34 White Owl To Howes Corner

Graphs showing the changes in vertical movement versus time for the control sections at MRM 109.4 are in Figure V-36. Observation points at both control sections reveal only minor movements with no specific trend. For the control section at MRM 109.4, a maximum heave of 0.032 feet was recorded on April 20, 1993 and for control section 108.6, a maximum heave of 0.022 feet was recorded on August 25, 1992. The majority of the maximum vertical movements were detected by the August 25, 1992 survey, only 31 days after the baseline elevation survey was performed.

V-3 SUBSURFACE INVESTIGATION

The field investigation also consisted of obtaining samples of the base and subgrade material at four month intervals. The samples of the base were taken off the augers, while the subgrade material was sampled using three inch diameter Shelby tubes. Sampling was discontinued when refusal was reached, hence, maximum depth was in some cases limited to two feet below the top of the subgrade. Samples were

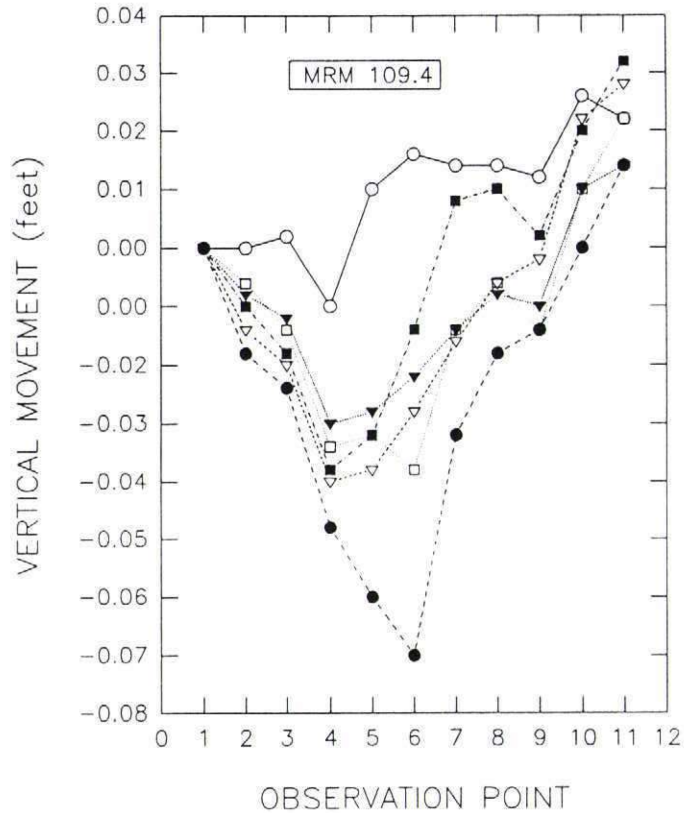


FIGURE V–36. SD34 WHITE OWL TO HOWES
VERTICAL MOVEMENT

ELAPSED TIME FROM ORIGINAL MEASUREMENT

○ 65 DAYS 7/25/92 ● 131 DAYS 9/29/92 ▽ 159 DAYS 10/27/92 ▼ 285 DAYS 3/2/93 □ 306 DAYS 3/23/93 ■ 334DAYS 4/20/93 obtained in June and November of 1992 and in April of 1993 from all of the control sections. In June of 1992 soil samples were taken at the crack and five feet from the crack. Subsequent soil samples, collected in November 1992 and April 1993, were taken at the crack only, with the exception of the project from LaPlant to the Missouri River. On this project two samples were taken; one at the crack and the other five feet from the crack.

V-3.1 BASE MATERIAL

V-3.1.1 Index Properties

The granular base material on all projects is very similar, therefore, only one test was performed per control section. The tests were performed on the samples taken in June 1992. The test results are summarized in Table V-3.

TABLE V-3. BASE MATERIAL PROPERTIES

PROJECT	CONTROL SECTION	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
US212E	204.2	25	21	4
	198.4	24	21	3
	189.7	23	19	4
	188	24	20	4
US212M	184.2	21	21	0
	182.9	22	19	3
US212W	172	24	20	4
	167.3	23	18	5
SD73N	170	22	19	3
	168.2	23	16	7
SD73S	160.1	23	17	6
	150	23	18	5
SD34	109.4	20	19	1
	108.6	23	19	4

The base material consists of natural gravely sand with less than 5 percent passing the Number 200 Sieve. The gravel and sand particles were found to be sub-rounded and the Plasticity of the fines is, except for one sample, less than 6 percent. The soil classification in accordance with the AASHTO classification system is A-1-a. The observed base material thicknesses are presented in Table V-9 together with the observed pavement thickness measurements.

V-3.1.2 Lateral Moisture Content Variation

The moisture content tests to determine the base material lateral distributions were performed on base material obtained initially, in June 1992. The average moisture content for the base material was determined at the crack and a distance five feet from the crack for each control section. The lateral distribution profile for each project was determined by comparing the moisture content distributions for all control sections within a project.

Since the base material generally was less than a foot thick, no vertical distribution was determined. The results are shown in Table V-4.

Very little variation in moisture content can be seen in the subgrade, however. The trend is toward a slightly higher moisture content at the crack as compared to five feet from the crack.

TABLE V-4. BASE MATERIAL MOISTURE CONTENT

PROJECT LOCATION	AVERAGE MOISTURE CONTENT AT CRACK	5' FROM CRACK
US212 LaPlant to the Missouri River	7.8	7.4
US212 Ridgeview to LaPlant	7.2	6.7
US212 Ridgeview 12.1 Miles West	6.6	6.6
SD73 Faith 14.1 Miles South	6.4	6.2
SD73 Howes Corner 14.2 Miles North	6.3	6.6
SD34 Howes Corner to White Owl		5.6

V-3.2 SUBGRADE MATERIAL ANALYSIS

The initial samples of the subgrade material, obtained in June 1992, were tested to determine the index properties and swell potential. Swell potential was determined under a confining pressure of 100 psf. The swell tests were performed on samples of the subgrade material taken both at the location of the cracks and outside the cracks for each control section. For the US Highway 212 projects, 22 swell tests were performed with 15 of those tests performed on subgrade material from LaPlant to the Missouri River. For the SD Highway 73 projects 8 swell tests, 2 per control section, were performed. Only one swell test was performed on SD Highway 34. The results for the subgrade material analysis are summarized in Table V-5. In accordance with the AASHTO classification system, all soils fall in the category A-7-6.

V-3.3 SUBGRADE MATERIAL MOISTURE CONTENT DISTRIBUTION

The initial moisture contents were obtained from the subgrade material obtained in June 1992. The average moisture content for the subgrade material was determined at the crack location and a distance five feet from the crack for each control section, and at depths of 3, 9, 15, and 21 inches below the base. However, due to the massive amount of data, typical moisture contents are only shown graphically for depths of 3 and 21 inches below the subgrade. The lateral distribution profile for each project was determined by comparing the moisture contents at all sample depths, for all control sections within a project, five feet from the crack and at the crack location. The vertical distribution profile for each project was determined by comparing the moisture contents at all sample depths regardless of the lateral sample placement.

V-3.3.1 US212 LaPlant to Missouri River

Lateral and vertical moisture content distributions for the control sections at MRP 204.2, 198.4, 189.7, and 188 are shown in figures V-37 and V-38 for depths of 3 and 21 inches below the base. As shown in the figures, the moisture content at the crack location is generally higher than those

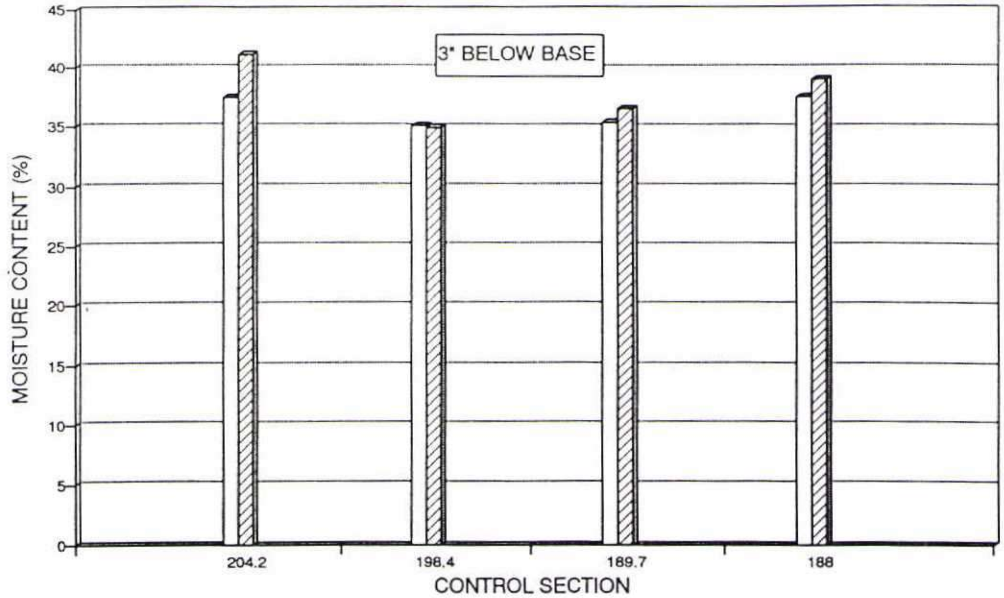


FIGURE V-37. US212 LAPLANT TO RIVER SUBGRADE MOISTURE DISTRIBUTION

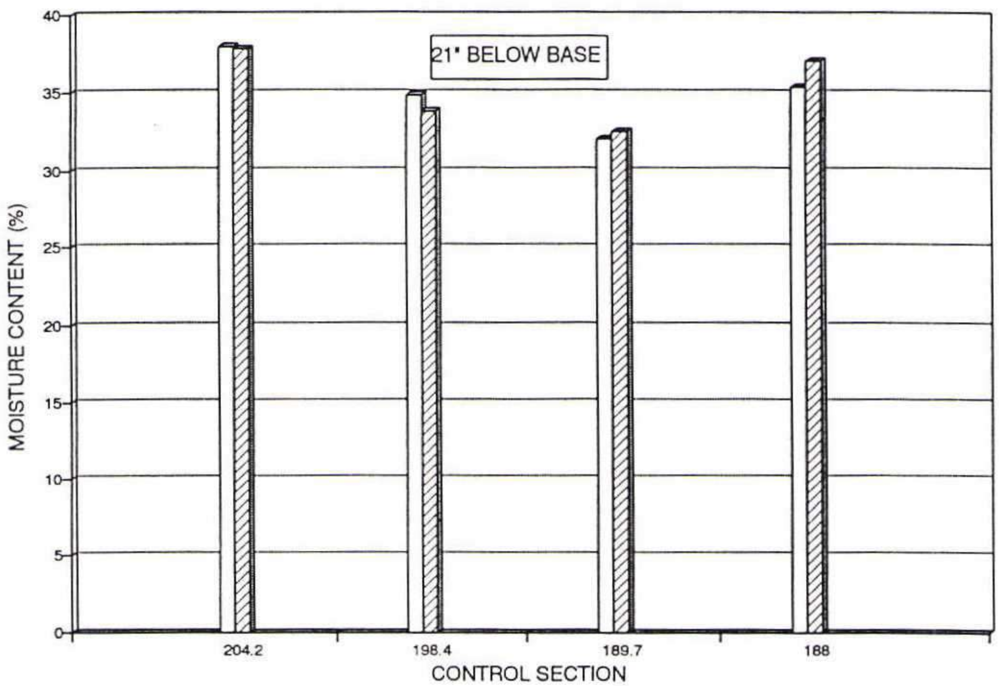
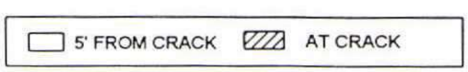


FIGURE V-38. US212 LAPLANT TO RIVER SUBGRADE MOISTURE DISTRIBUTION



at five feet from the crack location, at a depth of 3 inches below the base. The same trend is also true for depths of 9 and 15 inches below the base. However, at a depth of 21 inches below the base, the moisture content distribution shows little trend.

The moisture content was also found to be the highest near the subbase material. When comparing the moisture contents at a depth of 3 inches below the base material with those at greater depths, it was found that the moisture content at depths of 9, 15 and 21 inches below the base were, 3.6, 3.8, and 2.0 percent lower, respectively.

V-3.3.2 US212 Ridgeview to LaPlant

The lateral and vertical moisture content distributions for the control sections at MRM 184.2 and 182.9 are shown in figures V-39 - V-40. The figures show that the moisture content at the crack location is consistently greater than five feet from the crack location, regardless of the sample depth. On the average, the moisture content decreased 4.2%, 4.0%, 0.6%, from the crack to 5 feet from the crack at 3, 9, and 15 inches below the base, respectively. At a depth of 21 inches below base, however, the moisture content 5 feet from the crack was 2.8% higher than at the crack.

The moisture content also decreased from 3 inches below the base to 15 inches below the base. At a depth of 9 inches,

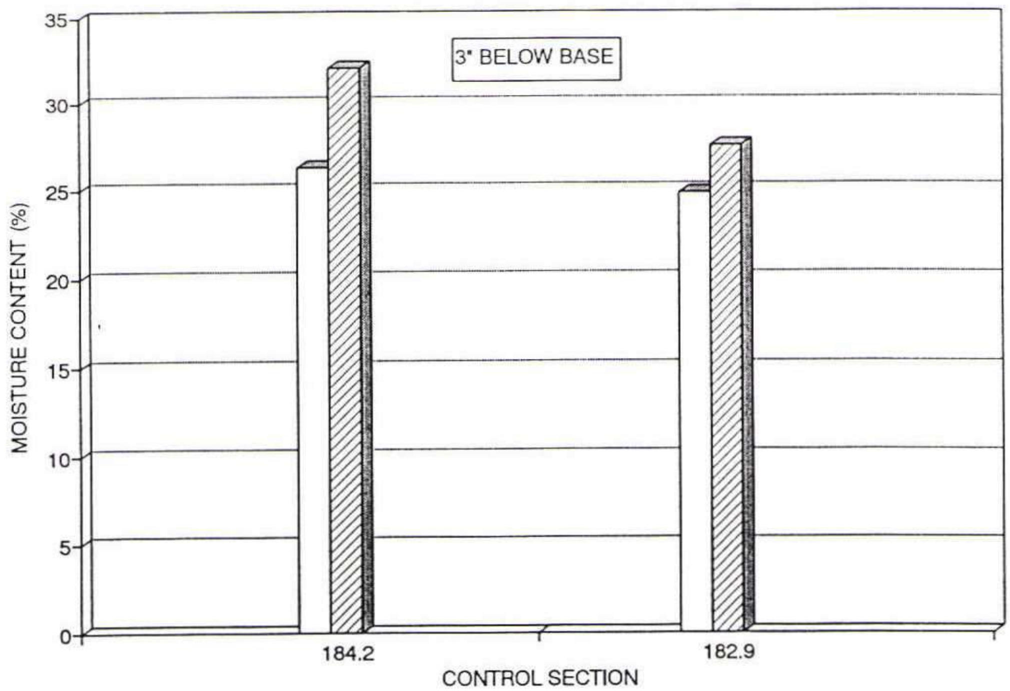


FIGURE V-39. US212 RIDGEVIEW TO LAPLANT SUBGRADE MOISTURE DISTRIBUTION

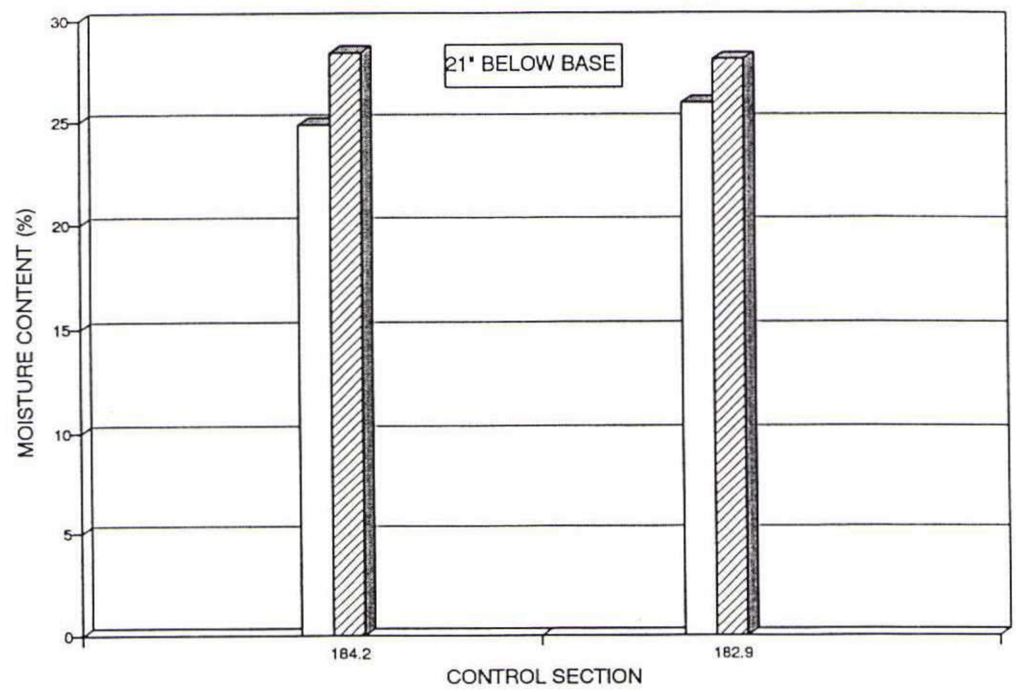


FIGURE V-40. US212 RIDGEVIEW TO LAPLANT SUBGRADE MOISTURE DISTRIBUTION

5' FROM CRACK AT CRACK

the moisture content had decreased 1.2 percent, while at a depth of 15 inches below the base, the moisture content was 2 percent below that at 3 inches below the base.

V-3.3.3 US212 Ridgeview West 12.1 Miles

The lateral and vertical moisture content distributions for the control sections at MRM 172 and 167.3 are shown graphically in figures V-41 and V-42. At this location, regardless of the sample depth, the moisture content at the crack location is again consistently greater than at five feet from the crack. The lateral distribution profile shows a decrease in moisture content from the crack to 5 feet from the crack of 6.5, 3.2, 1.4, and 5.5 percent at depths of 3, 9, 15, and 21 inches below base, respectively.

The moisture content also tends to decrease with depth. The vertical distribution profile shows a 1.6 percent decrease in the moisture content both from 3 to 9 inches and 3 to 15 inches below the base, while the moisture content decreased 3.8 percent from 3 to 21 inches below the base material.

V-3.3.4 Highway 73 Faith South

The lateral and vertical moisture content distributions for control sections MRM 170 and 168.2 are shown in figures V-43 and V-44. At this project, it was found that the moisture content at five feet from the crack were less than those at the crack at all depths except at 21 inches below the base. On the average, the lateral distribution profile shows

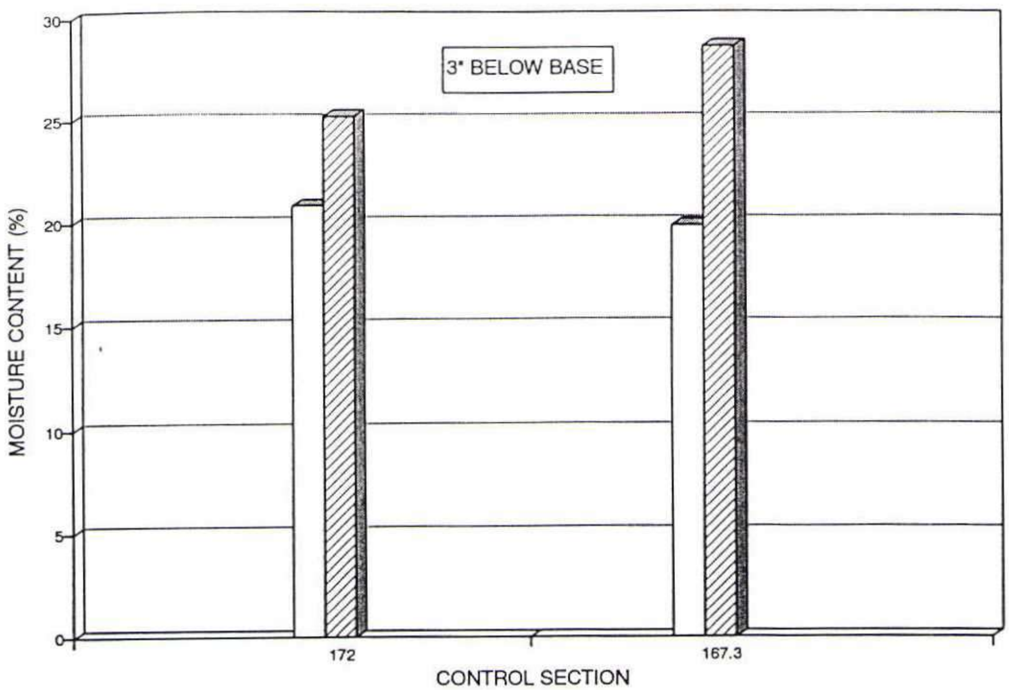


FIGURE V-41. US212 RIDGEVIEW WEST SUBGRADE MOISTURE DISTRIBUTION

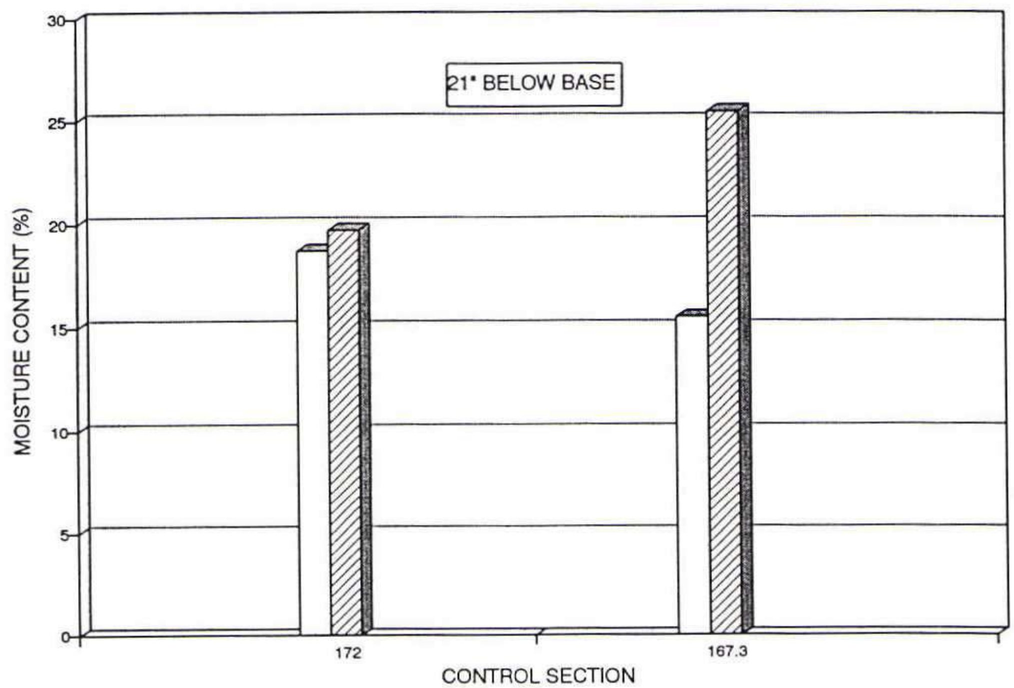
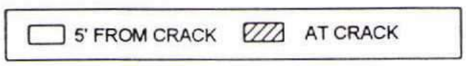


FIGURE V-42. US212 RIDGEVIEW WEST SUBGRADE MOISTURE DISTRIBUTION



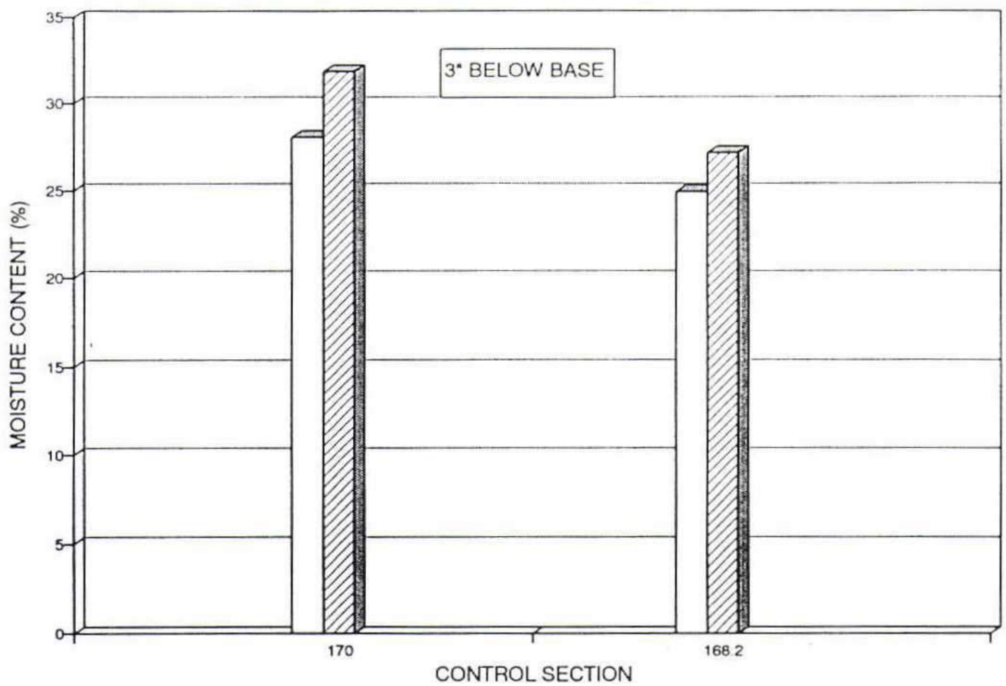


FIGURE V-43. SD73 FAITH SOUTH SUBGRADE MOISTURE DISTRIBUTION

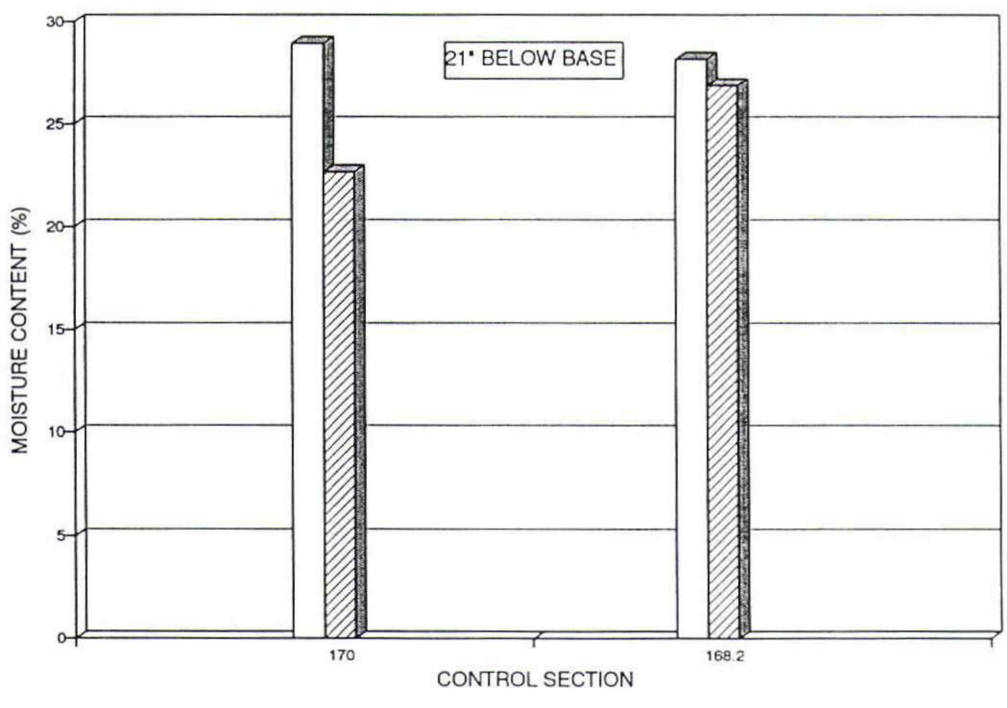
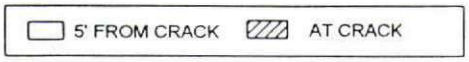


FIGURE V-44. SD73 FAITH SOUTH SUBGRADE MOISTURE DISTRIBUTION



decreases of 3.0, 3.2, and 2.1 percent at depths of 3, 9 and 15 inches below the base increase, respectively. At a depth of 21 inches below the base, the moisture content was 3.8 percent higher 5 feet from crack.

The moisture content distribution also shows a slight increase to a depth of 9 inches below the base. From 9 inches below the base, the moisture content decreases with depth. For the control sections at MRM 170 and 168.2, the vertical distribution profile shows a 0.3 percent increase in moisture content from 3 to 9 inches below the base, while comparisons of the moisture content from 3 to 15 and 3 to 21 inches below the base show a 0.8 and 1.9 percent decrease, respectively.

V-3.3.5 Highway 73 Howes Corner North

The moisture content variation, laterally and vertically, for the control sections at MRM 160.1 and 150 are shown graphically in Figures V-45 and V-46. As can be seen, at a depth of 3 inches below the base, the moisture content at MRM 160.1 at the crack location is slightly smaller as compared to five feet from the crack. The opposite is the case at MRM 150. However, the trend of decreasing moisture away from the crack was again found at depths of 9, 15, and 21 inches below the base material. The average moisture profiles show a decrease in moisture content from the crack to 5 feet from the crack of 2.1 percent at depths of 9 and

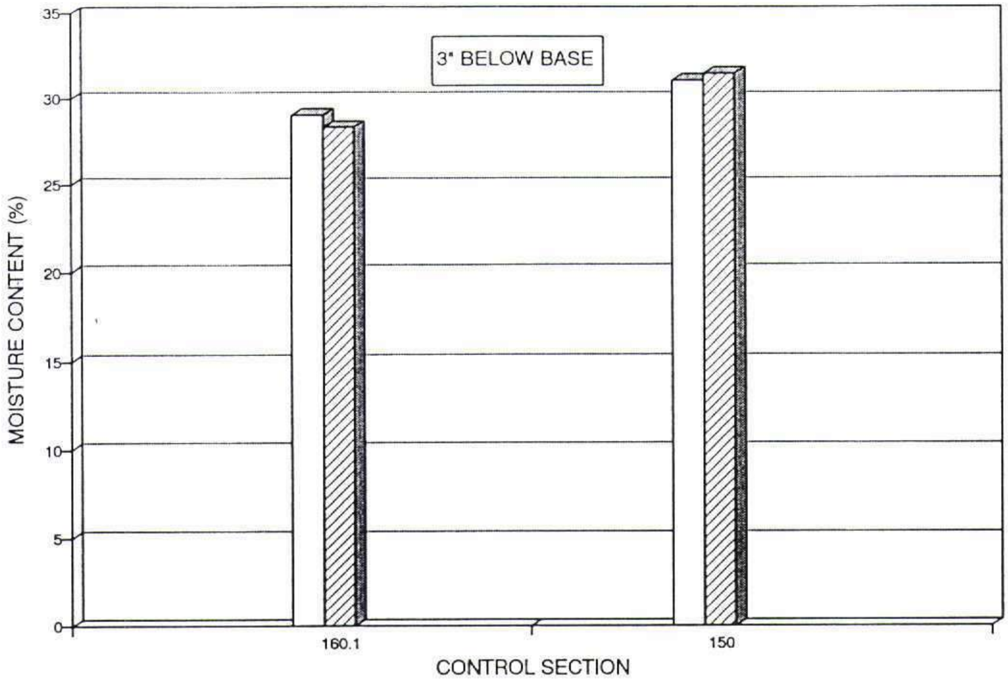


FIGURE V-45. SD73 HOWES NORTH SUBGRADE MOISTURE DISTRIBUTION

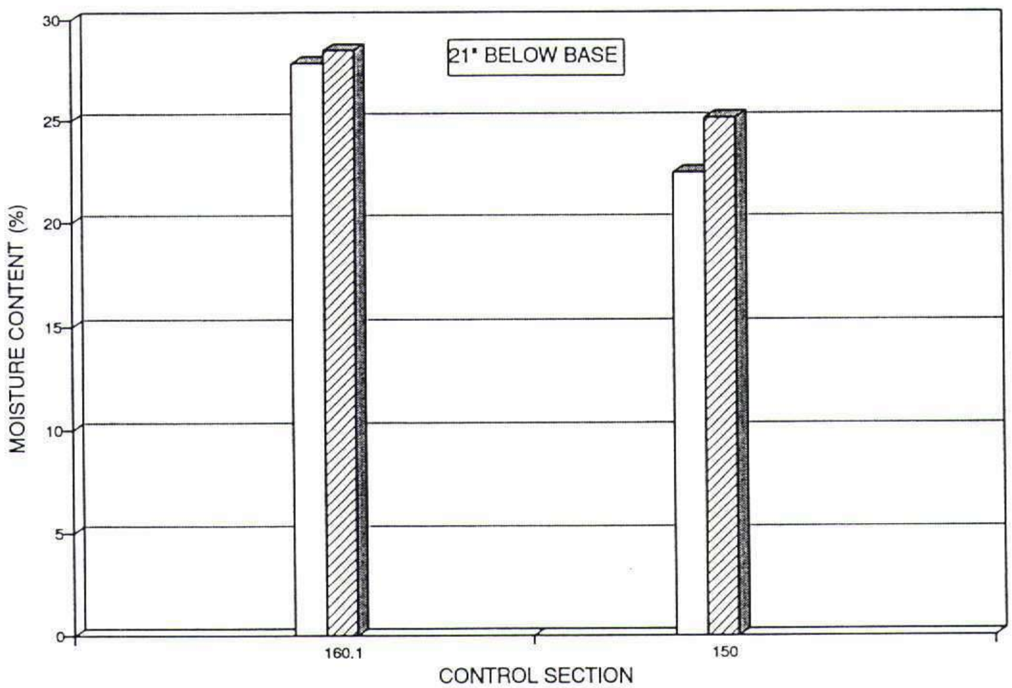


FIGURE V-46. SD73 HOWES NORTH SUBGRADE MOISTURE DISTRIBUTION

5' FROM CRACK
AT CRACK

15 inches, and 1.7 percent at a depth of 21 inches below the base.

Comparing moisture content with depth, the moisture content decreases with depth. The resulting vertical distribution profile shows a 0.8, 3.4, and 4.0 percent decrease in the moisture content from 3 to 9 inches, 3 to 15 inches, and 3 to 21 inches below the base, respectively.

V-3.3.6 Highway 34 White Owl To Howes Corner

There are no cracks within either control section for this project. Therefore, the lateral moisture content in terms of distance from a crack could not be determined. Any moisture changes recorded laterally would be the result of random variations of placement moisture content, and not of infiltration from the surface.

The variation in moisture content vertically shows a decrease with depth. The vertical distribution profile for this project shows a 3.5, 3.8, and a 2.9 decrease in the moisture content from 3 to 9 inches, 3 to 15 inches, and 3 to 21 inches below base, respectively. It should be noted, however, that subgrade moisture contents at this project are substantially lower than at other projects (Table V-5).

V-3.4 TEST PIT

On April 21, 1993 a test pit was dug perpendicular to the crack near the control section at MRM 189.7 on US Highway 212 from LaPlant to the Missouri River. The pit allowed the

TABLE V-5. RESULTS OF TESTS ON SUBGRADE SAMPLES OBTAINED JUNE 1992

PROJECT / CONTROL SECTION	LOCATION	LIQUID LIMIT	PLASTIC LIMIT	DRY DENSITY (PSF)	NATURAL MOISTURE CONTENT (%)	SWELL POTENTIAL (%)
US212E -- LaPlant to Missouri River						
204.2						
	WC @ Crack	78	50	83.6	36.6	2.5
	WC 20' West			82.3	32.6	2.2
	WC @ Crack			77.8	37.4	1.3
	EC 5' West			77.8	33.7	1.4
198.4						
	WC @ Crack		46	76.9	34.0	0.4
	WC @ Crack			86	33.3	2.0
	WC 5' East			83.2	31.8	2.8
	EC 37' West			91.6	29.7	1.0
189.7						
	EC @ Crack	68	58	82	31.8	2.7
	EC 5' East			91.3	28.1	4.4
	MC 5' WEST			83.8	35.3	1.1
188						
	EC @ Crack	83	58	78.1	32.7	3.7
	EC 16' West			77.0	34.5	2.8
	WC @ Crack			79.9	35.8	3.5
	WC 5' West			80.7	35.4	1.5
US212M -- Ridgeview to LaPlant						
184.2						
	EL @ Crack	73	50	85.2	24.4	8.1
182.9						
	WL @ Crack	74	52	86.4	26.5	7.5
	EL 5' East			79.8	27.1	0.4

E - East; N - North; S - South ; W - West; C - Crack; L - Lane; M - Middle

TABLE V-5 CONTINUED.

PROJECT CONTROL SECTION	LOCATION	LIQUID LIMIT	PLASTIC LIMIT	DRY DENSITY (PSF)	NATURAL MOISTURE CONTENT (%)	SWELL POTENTIAL (%)
US212W -- Ridgeview west 12.1 miles						
172						
	EL @ Crack	59	34	81.7	22.2	1.4
	EL 5' West			79.4	19.0	3.7
167.3						
	WL @ Crack	50	33	89.8	25.4	1.2
	WL 5' West			94.7	19.2	7.3
SD73N -- Faith south						
170						
	NC @ Crack	46	31	98.6	21.1	1.2
	NC 5' North			99.8	18.4	0.5
168.2						
	SC @ Crack	50	32	92.2	27.4	1.7
	SC 5' South			89.8	24.9	0.9
SD73S -- Howes Corner north						
160.1						
	SC @ Crack	56	37	91.6	25.8	3.7
	SC 5' North			91.2	24.3	1.7
150						
	NC @ Crack	63	45	85.4	28.2	1.9
	SC 5' North			90.7	26.3	2.4
SD34 -- White Owl - Howes Corner						
108.6						
		55	40	102.	18.4	6.4

E - East; N - North; S - South ; W - West; C - Crack; L - Lane; M - Middle subsurface conditions to be studied and sampled in detail. The test pit extended a distance of 6 feet on each side of the crack, was 5.5 feet deep and approximately 3 feet wide. The test pit remained open for one day. Photographs of the test pit and material excavated are shown in Photos V-1 and V-2.

Samples were taken on a one square foot pattern to a depth of four feet below the top of the subgrade. Samples were also taken from the midpoint of the base material. These samples were tested for moisture content in order to obtain a detailed moisture content distribution verses depth and distance from the crack. The lateral and vertical moisture content distributions for the test pit are shown in Figure V-47 and Table V-6.

The lateral moisture content distribution shows a decline as the distance from the crack increases. The amount of the decline varies depending upon the depth of the sample. Below the top of the subgrade material, the moisture content decreases as the depth of the sample increases.

The maximum measured moisture content of 42.4% occurred at the crack location at a depth of one foot below the base/subgrade interface.

PHOTO V-1. US212 LAPLANT TO RIVER
TEST PIT

PHOTO V-2. US212 LAPLANT TO RIVER
TEST PIT SHALE FRAGMENT

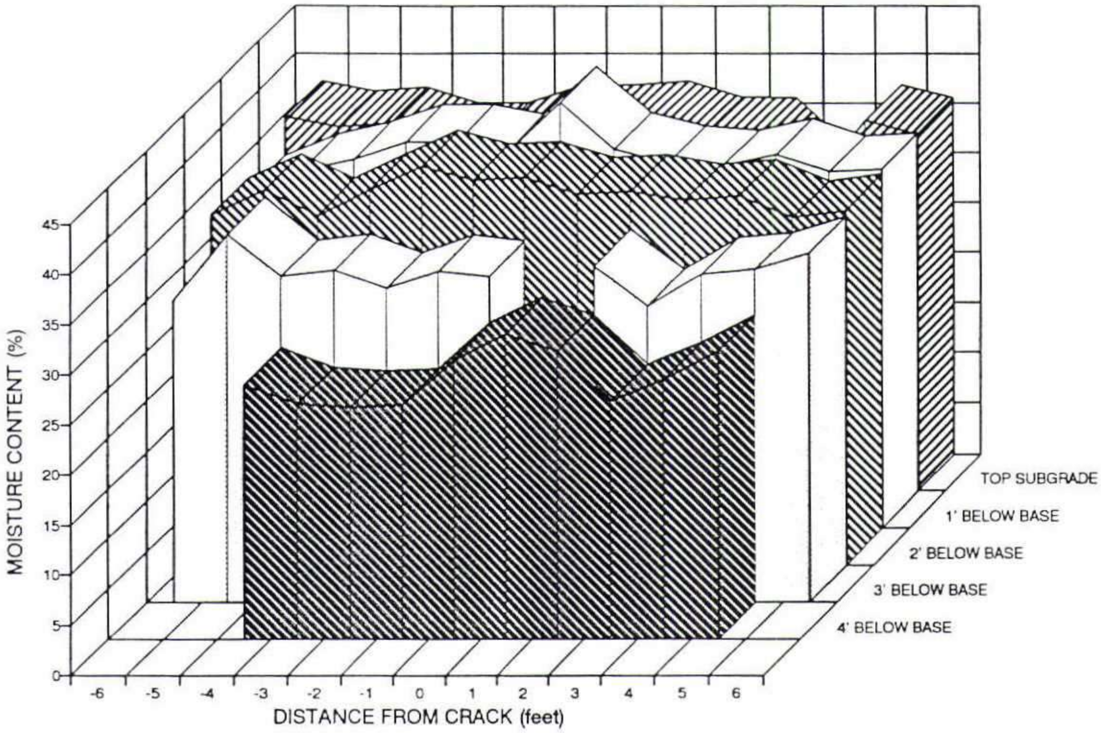


FIGURE V-47. TEST PIT SUBGRADE MOISTURE DISTRIBUTION

TABLE V-6. TEST PIT MOISTURE CONTENT DISTRIBUTION

	DISTANCE FROM CRACK (Feet)												
	WEST						CRACK						EAST
LOCATION	- 6	- 5	- 4	- 3	- 2	- 1	0	1	2	3	4	5	6
Mid-depth base	7.56	7.82	7.55	7.82	7.99	8.48	9.86	8.53	8.21	7.76	7.67	7.88	7.82
Top Subgrade	37.3	36.3	36.7	35.1	35.1	36.8	36.9	37.2	35.7	35.5	30.6	36.9	35.5
1' below base	33.6	35.7	36.8	38.5	38.5	37.6	42.4	37.7	36.3	36.0	37.1	35.4	35.7
2' below base	35.0	37.4	34.9	37.4	39.6	38.3	38.5	36.9	37.2	36.2	36.7	34.5	35.2
3' below base	30.1	36.7	32.4	33.0	31.2	32.9	32.3	----	33.3	29.4	32.5	33.0	34.6
4' below base	----	----	25.3	23.3	23.0	23.2	27.7	30.3	28.5	23.6	25.8	28.5	----

V-3.5 SUBGRADE MOISTURE CONTENT DISTRIBUTION VERSUS TIME

At all projects, the subgrade material was analyzed for moisture content variation with time and season.

V-3.5.1 Base Material

The tests on samples of the base material obtained in June 1992, November 1992, and April 1993, show a general trend towards increasing moisture content with time (See Figure V-48). The average moisture content fluctuation for each project is listed in Table V-7.

TABLE V-7. BASE MOISTURE CONTENT VERSUS TIME

Project	June '92 - November '92	June '92 - April '93
US212E	0.2% increase	0.9% increase
US212M	0.9% increase	1.7% increase
US212W	0.6% increase	0.3% increase
SD73N	0.1% increase	0.2% increase
SD73S	0.3% decrease	0.9% increase
SD34	2.1% decrease	1.2% decrease

V-3.5.2 Subgrade Material

Sampling was performed in June 1992, November 1992, and April 1993 at depths of 3, 9, 15 and 21 inches below the base (Figures V-49 through V-52). No trend in variation of moisture content with time appears to be present on any project except for the project from Ridgeview 12.1 miles west. At this location, the moisture content appears to increase with each subsequent sampling, regardless of the sample depth.

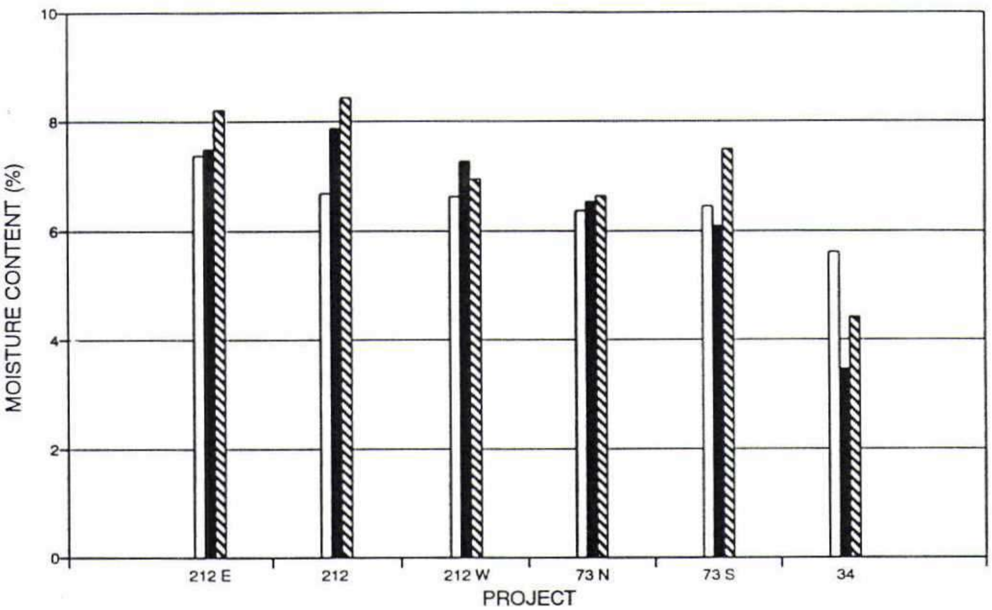


FIGURE V-48. BASE MATERIAL MOISTURE DISTRIBUTION

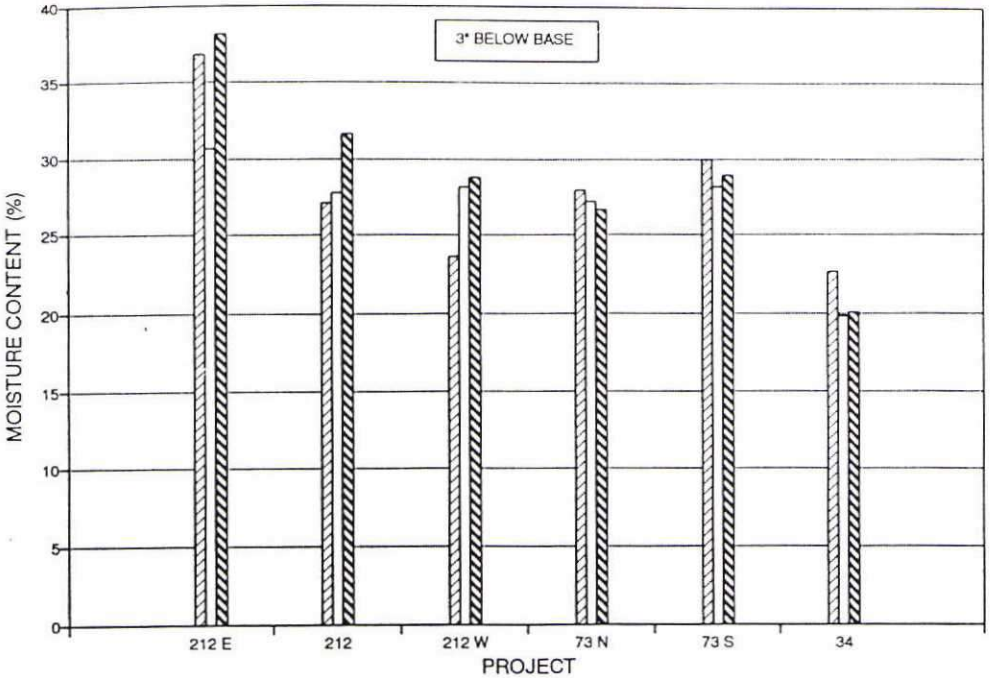


FIGURE V-49. SUBGRADE MATERIAL MOISTURE DISTRIBUTION

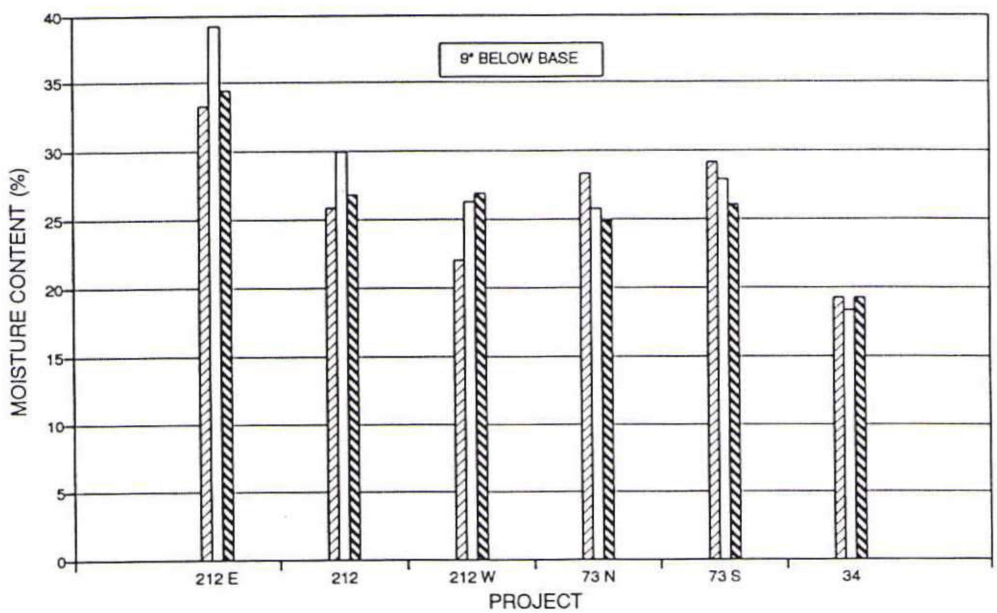
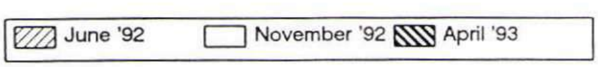


FIGURE V-50. SUBGRADE MATERIAL MOISTURE DISTRIBUTION



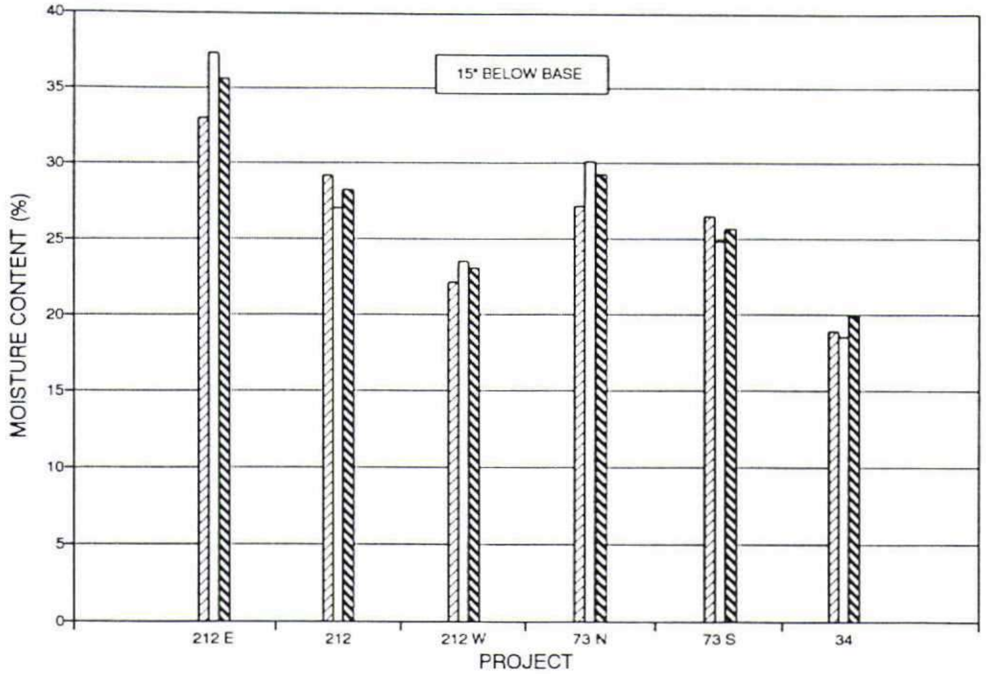


FIGURE V-51. SUBGRADE MATERIAL MOISTURE DISTRIBUTION

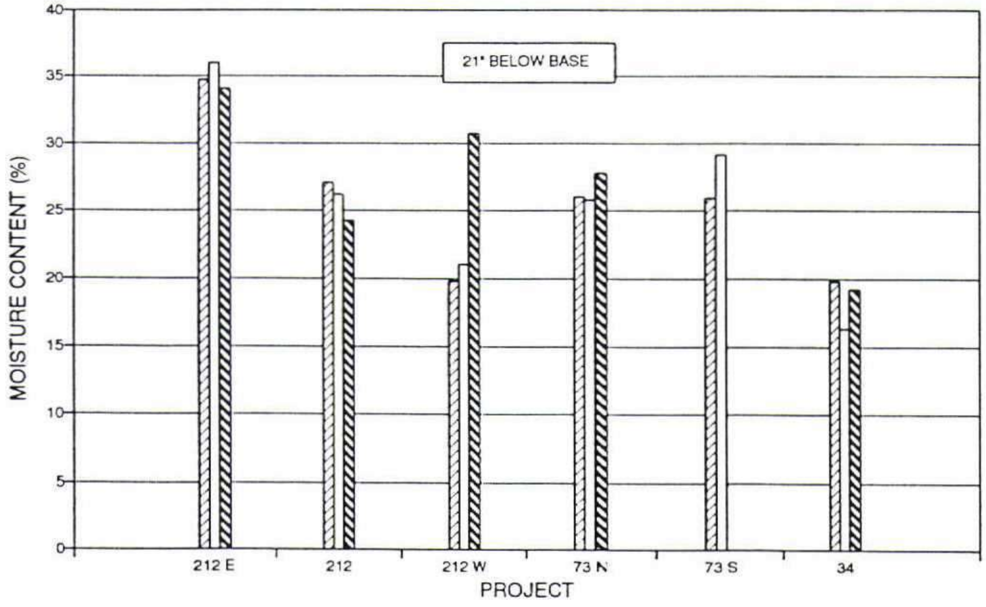
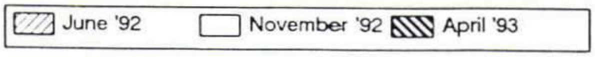


FIGURE V-52. SUBGRADE MATERIAL MOISTURE DISTRIBUTION



V-4 PAVEMENT

In addition to measuring the asphalt thickness at each borehole location, cores were taken of the asphalt at each of the control sections (a total of 14 cores). The cores were approximately four inches in diameter. The asphalt cores were measured and exposed to different temperatures to evaluate their expansion or shrinkage characteristics. Extraction tests were also performed to determine the oil content of the asphalt and may be related to the observed shrinkage.

A crack survey was also performed for each of the projects in order to determine crack frequency.

V-4.1 CRACK SURVEY

A crack survey was performed at the beginning of the investigation. The number of cracks was determined by counting the number of cracks seen between mile reference posts. On Highway 34, however, the survey revealed no cracks. The survey here was repeated by walking the entire 15.6 miles by personnel from both the SDSM&T and the SDDOT, without any cracks being seen. The crack frequency on the remaining projects varied from as few as 3 cracks per mile on the project from Ridgeview to LaPlant, to as much as 150 cracks per mile on US212 from LaPlant to the Missouri river. The details of the crack survey are presented in Table V-8 below:

TABLE V-8. RESULTS OF CRACK SURVEY

PROJECT LOCATION	CRACKS PER MILE
US212 LaPlant to the Missouri River	120-150
US212 Ridgeview to LaPlant	3
US212 Ridgeview 12.1 Miles West	50-60
SD73 Faith 14.1 Miles South	80-100
SD73 Howes Corner 14.2 Miles North	20-60
SD34 Howes Corner to White Owl	NONE*

*Four cracks were detected during April of 1993

V-4.2 PAVEMENT THICKNESS

The pavement thickness was measured at all borehole locations and in the test pit. In general, the pavement ranged from 4 to five inches in thickness. It should be noted, however, that the pavement thickness at the milled sections on the project from LaPlant to the Missouri river was as low as 3.5 inches.

TABLE V-9. DESIGN AND OBSERVED DIMENSIONS

LOCATION	ASPHALT		BASE/SUBBASE	BASE+SUBBASE
	*DESIGN	OBSERVED	DESIGN	OBSERVED
SD34	1.5/1/4	4.5	6 + 8	12
SD73S	2/1/4.5	4.5	5 + 6	11
SD73N	1.5/1/3	4	6 + 8	11
US212W	1.5/1/4	4.5	8 + 8	10
US212M	2.5/4	5	8 + 8	10
US212E	2.5/4	4	6 + 10	10

*Ultimate Section
All dimensions in inches

V-4.3 ASPHALT EXTRACTION TESTS

Asphalt extraction tests were run on all cores. At most locations, the oil content was calculated for both the wearing course and the base course. The extraction was performed using Reflux type extraction equipment and 1,1,1 trichloroethane for solvent. The results are shown below in Table V-10:

TABLE V-10. RESULTS OF EXTRACTION TESTS

PROJECT	CONTROL SECTION	AC CONTENT (%) SURFACE	BASE	OVERALL
US212E	204.2	5.6	5.5	-
	198.4	6.5	6.1	-
	189.7	5.6	6.4	-
	188	3.3	3.9	-
US212M	184.2	5.9	4.9	-
	182.9	5.6	6.8	-
US212W	172	-	-	7.1
	167.3	-	-	5.8
SD73N	170	6.5	6.7	-
	68.2	6.5	5.9	-
SD73S	160.1	5.9	5.7	-
	150	-	-	6.4
SD34	109.4	6.6	7.1	-
	108.6	-	-	-

It should be noted that severe corrosion of the copper condensers occurred during the extraction tests on the asphalt taken from the sections on US212 from LaPlant to the Missouri river, and the asphalt residue formed curd like particles. Also, extreme hardening of the asphalt took place following breaking up and cooling of the cores. In accordance with conversations with Dan Johnston from the SDDOT, corrosion of the condensers is caused by breakdown of the solvent into hydrochloric acid. Breakdown of the solvent has been associated with the presence of material in the asphalt which forms free radicals, which act as a catalyst in breaking down the solvent. Commonly, large amounts of heavy metals such as Vanadium are typically to blame. A high Vanadium content in asphalt can cause hardening, which has been observed to result in a high frequency of transverse cracks.

CHAPTER VI

SUMMARY AND DISCUSSION

V1-1 GENERAL

The evaluation of the data is correlated with the variables listed in Table VI-1, Table VI-2, and Figure VI-1. Table VI-1 represents the soil variables, while Table VI-2 represents data regarding construction procedures. Figure VI-1 shows the daily maximum and minimum temperatures as recorded at Faith during the period covering the investigation. The difference in in situ moisture contents between projects as observed during the course of the investigation (Figures V-49 through V-52), was also considered.

VI-2 LATERAL MOVEMENTS

Lateral field measurements were performed for a period of 12 months to monitor lateral movement over the crack and away from the crack location. The results indicate that the maximum lateral movements for all of the projects, over all crack observation intervals, were recorded on March 2, 1993. As can be seen from Figure VI-1, March 2 was at the end of an extended cold period which began in the first week of January. March 2 may therefore be close to the date representing the lowest soil temperatures of the year. The measurements show a trend toward closing of the cracks as the air and ground temperature increased during the spring of 1993. The magnitude of the movements across the cracks,

TABLE VI-1. SOIL VARIABLES

Project	P I	Average Moisture Content less Optimum at Placement (%)	Change in Moisture Content Since Placement at Crack 3" Below Base(%)	Change in Moisture Content Since Placement at Crack 21" Below Base(%)	Average Compaction (%)	% Swell Potential
US 212 LaPlant to River	46	+.1	5.5	3.5	99.7	0.4-4.4
US 212 Ridgeview to LaPlant	46	+.2	1	0	98.9	0.4-8.1
US 212 Ridgeview West	36	+.4	0	-3	98.9	1.2-7.3
SD 73 Faith South	31	-.2	5.5	2.5	99.1	0.5-1.7
SD 73 Howes Corner North	36	-1.1	6.5	3.5	99.9	0.9-3.7
SD 34 Howes Corner West	31	+1.0	-	-	100.2	6.4

TABLE VI - 2. CONSTRUCTION VARIABLES

Project	Type of Overexcavation	Overexcavation Depth (feet)	Shale Blocks in Fill	Separation of Salvage and New Subgrade
US 212 LaPlant to River	Shoulder	4 to 6	Yes	No
US 212 Ridgeview to LaPlant	Toe	4	No	Yes
US 212 Ridgeview West	Shoulder	3 to 4	No	No
SD 73 Faith South	Toe	3	No	Yes
SD 73 Howes Corner North	Toe	3	No	Yes
SD 34 Howes Corner West	Toe	3	No	Yes

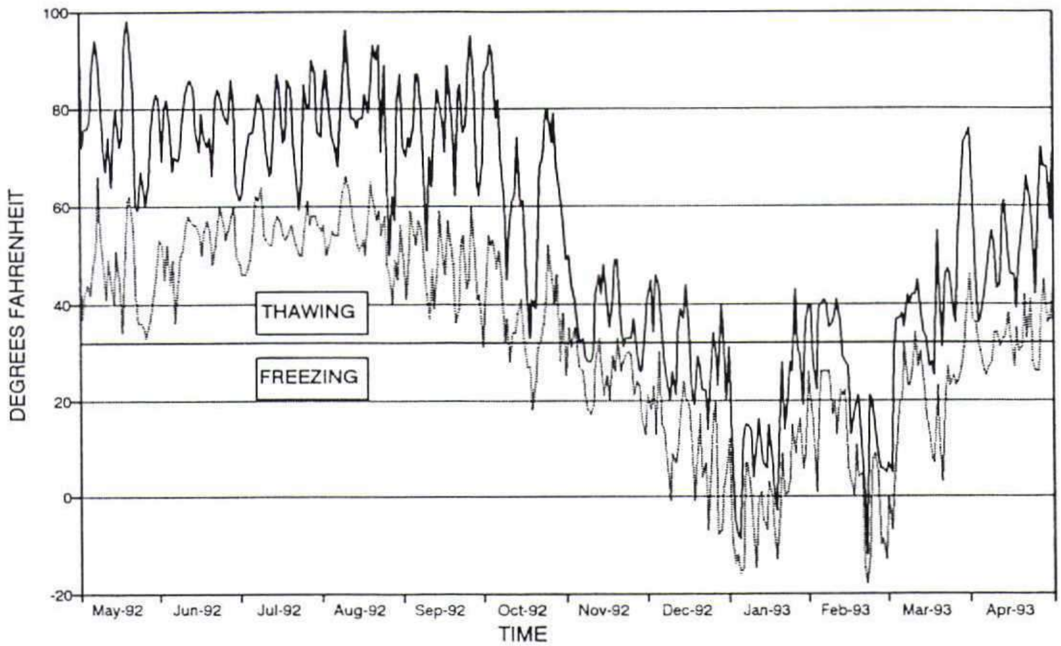


FIGURE VI-1. DAILY TEMPERATURES FAITH, SOUTH DAKOTA

— MAXIMUM — MINIMUM

as evidenced by the changes seen on Photos VI-1 through VI-8, were on the order of 10 millimeters or greater. The movements are surprisingly large, and have undoubtedly led to the distress and separation of the crack sealant from the walls of the crack. On all projects, including SD Highway 73 where crack and chip sealing had been performed during the summer of 1992 (Photos VI-7 and VI-8), the crack sealant had separated from the walls of the cracks, and left openings large enough to see through the pavement down to the base. The lateral movement may be attributed to both temperature effects and permanent shrinkage of the asphalt. For the observation intervals not spanning the cracks, the lateral movements recorded on all projects, varied between plus and minus two millimeters. This magnitude may be within the margin of error for the measurements, since it was observed that occasional nails had been hit by snow plows and shifted as much as 2 millimeters. The movements between the reference points not spanning the crack just reflect the contraction and expansion for the one foot distance between the observation points. On the other hand, the difference in movements across the cracks, reflect the total shrinkage of the slab between cracks.

No lateral movements were measured on SD Highway 34, although the widths of the cracks which developed during the winter 1992/1993 were on the order of 1/4 inch wide.

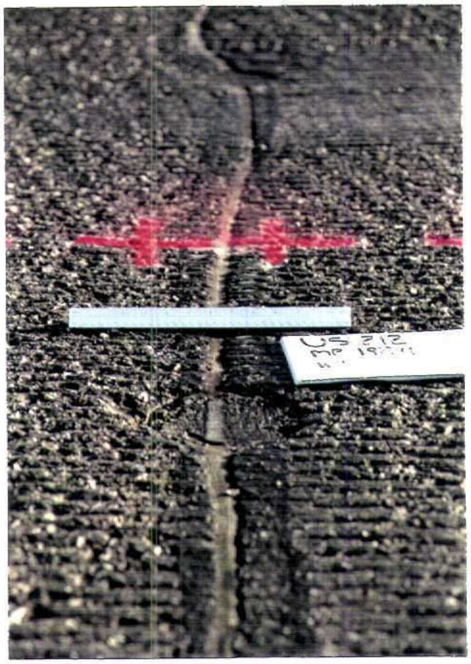


PHOTO VI-1. US212 LAPLANT TO RIVER
MRM 198.4 WEST CRACK
CRACK WIDTH: SEPTEMBER 1992

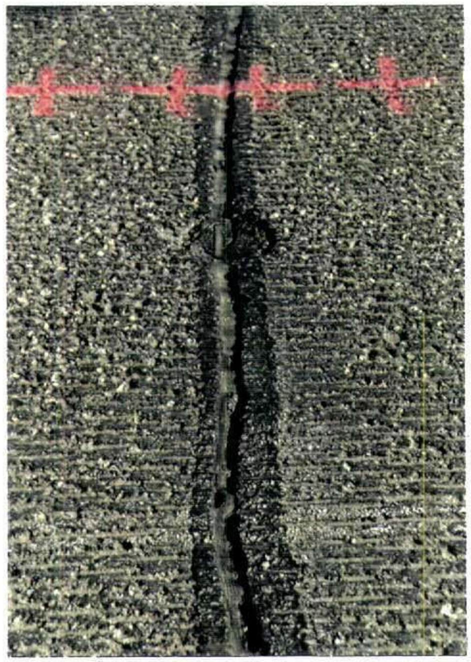


PHOTO VI-2. US212 LAPLANT TO RIVER
MRM 198.4 WEST CRACK
CRACK WIDTH: MARCH 1993

PHOTO VI-3. US212 RIDGEVIEW-LAPLANT
MRM 182.9 WEST CRACK
CRACK WIDTH: SEPTEMBER 1992

PHOTO VI-4. US212 RIDGEVIEW-LAPLANT
MRM 182.9 WEST CRACK
CRACK WIDTH: MARCH 1993

PHOTO VI-5. US212 RIDGEVIEW WEST
MRM 167.3 WEST CRACK
CRACK WIDTH: SEPTEMBER 1992

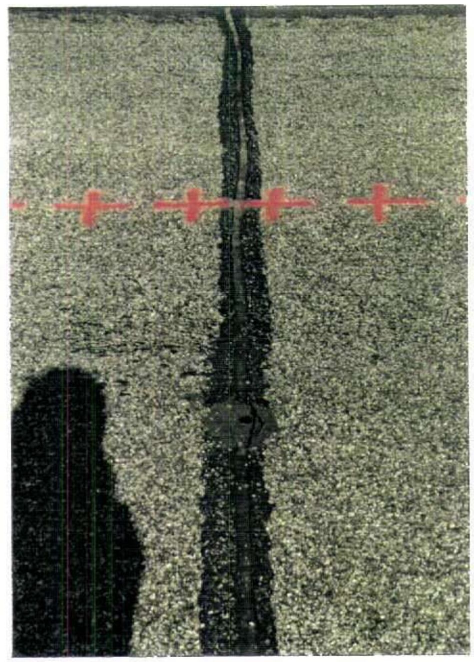


PHOTO VI-6. US212 RIDGEVIEW WEST
MRM 167.3 WEST CRACK
CRACK WIDTH: MARCH 1993

PHOTO VI-7. SD73 FAITH SOUTH
MRM 168.2 NORTH CRACK
CRACK WIDTH: SEPTEMBER 1992

PHOTO VI-8. SD73 FAITH SOUTH
MRM 168.2 NORTH CRACK
CRACK WIDTH: MARCH 1993

VI-3 VERTICAL MOVEMENTS

Periodic vertical field elevations were taken for a period of 12 months to monitor heave development. The results indicate that the majority of the maximum vertical movements for all of the projects were recorded in the spring of 1993, while frost was still in the ground. Based on the temperature records presented in Figure VI-1, maximum frost depth, and consequently maximum frost heave, may have occurred close to the first week of March. The heaves observed during the late winter of 1993, and which also reversed in the April, 1993 survey, are undoubtedly due to frost heave. It is also likely, however, that additional heaves due to swelling of the subgrade soil occurred, as the subgrade moisture content at the cracks were observed to increase during the course of the project.

Since the maximum amount of heaves have occurred at the crack, it is likely that a substantial portion of the heaves are due to subgrade expansion. It is also likely that the source of bulk of the water has infiltrated through the cracks. This is also supported by the moisture distribution in the subgrade. The moisture content generally decreases away from the crack and with depth, indicating a migration of moisture from a source at the crack. Additional evidence of the heaves being caused by soil subgrade swelling may also be seen as the heaves decrease at the shoulders. Just beyond the shoulder, water will evaporate as there is no asphalt cover. Vegetation will also tend to reduce the moisture content. In addition, the asphalt shoulders are less than half as thick as the pavement mat, and is comprised of rolled recycled asphalt. The shoulder asphalt will therefore allow more evaporation from the base and subgrade to take place than through the pavement mat.

The magnitude of the maximum vertical movement varies from project to project. There are no consistent trends within each project or each control section. The majority of the movement appears immediately next to the crack and usually is less than a 0.050 feet increase. However, the maximum vertical movements from LaPlant to the Missouri River are in the magnitude of 0.042 to 0.092 feet. These represent the largest vertical movements observed among all of the projects on US Highway 212, Highway 73, and Highway 34. The smallest vertical movements were observed on Highway 34. The maximum vertical movement within this project was 0.032 feet. The other four projects all had vertical movements within 0.050 feet.

VI-4 SWELL POTENTIAL

Swell potential for each project was determined under a confining stress equivalent to that in the field. A summary of the average swelling potentials, dry densities, and moisture contents for all of the projects is listed in Table VI-3. below:

TABLE VI-3. SWELL POTENTIAL

PROJECT	SWELL POTENTIAL	DRY DENSITY	MOISTURE CONTENT IN PLACE	OPTIMUM
US212E	2.2 %	82.2 psf	33.5 %	32 %
US212M	5.3 %	83.8 psf	26.0 %	26 %
US212W	3.4 %	86.4 psf	21.4 %	28 %
SD73N	1.1 %	95.1 psf	23.0 %	22 %
SD73S	2.4 %	89.7 psf	26.2 %	23 %
SD34	6.4 %	102.3 psf	18.4 %	18 %

From the above table, it is evident that the samples with high density and low moisture content in relation to optimum moisture content have the highest swell potential on US Highway 212, the smallest swell potential was recorded from LaPlant to the Missouri River, (the project which has heaved the most), and the largest swell potential was recorded from Ridgeview to LaPlant, the project that has very little to no heaving. The project from Ridgeview west 12.1 miles has experienced some heaving and has a swelling potential between those found on the other US212 projects.

The smallest average swelling potential, both in terms of free swell and plasticity index, was recorded on the project from Faith south on Highway 73. Although the moisture changes since construction on this project are significant, the amount of heaves that would have taken place are most likely small. The project from Howes Corner north had a swelling potential slightly larger than that found on the project from LaPlant to the Missouri River. The largest average swelling potential recorded was on

Highway 34 from White Owl to Howes Corner. This project was completed in 1991 and has the potential to experience large amounts of heave if water infiltration through the cracks is permitted.

Based on the results of the swell tests on samples placed at or close to optimum water content and 100 percent compaction, the subgrade soils along US Highway 212 from LaPlant to the Missouri River may be estimated to have had a swell potential between four and eight percent. Based on the moisture content increases since construction on US Highway 212 (Table VI-1), a significant amount of the potential swell at the cracks on this project is likely to have been realized. Assuming that this swell potential is fully realized at the top of the subgrade and that expansion, due to the overburden pressure, is negligible at a depth of four feet below the top of the subgrade, total heaves should have been on the order of one to two inches. Additional heaves may have been caused by frost action, however, most of the heaves due to frost action would reverse following thawing.

VI-5 MOISTURE CONTENT DISTRIBUTION

The magnitude of the moisture content for the base and subgrade material varies from project to project, but it appears that regardless of the type, or depth of the sample, the project to project distribution trend is similar. Also, as can be seen from Table VI-1, the largest changes in moisture content since construction appear to have occurred on US Highway 212 from LaPlant to the Missouri River and on the projects on SD Highway 73.

The moisture contents of the subgrade varies from project to project on US Highway 212, with the highest moisture recorded from LaPlant to the Missouri River and the smallest values from Ridgeview west 12.1 miles. The two projects on Highway 73, from Faith south and Howes Corner north, appear to have moisture contents of the same magnitude, which are slightly greater than found from Ridgeview west 12.1 miles. The smallest moisture contents by far are located on Highway 34. These moisture contents are significantly smaller than those recorded for the other projects. This may be attributed to the fact that there are no cracks providing a means for water infiltration within these control sections. However, it should also be noted that optimum moisture content on the projects decreased from project to project towards the west, with the lowest optimum moisture contents on Highway 34.

VI-5.1 LATERAL DISTRIBUTION

The general trend for all of the projects, in the base and the subgrade material, is for the moisture content to be greater at the crack location versus five feet from the crack. In subgrade material the average moisture content at the cracks was 6.5 percent higher than at a distance of 5 feet from the cracks. In the base material, the average difference was only 0.2 percent, however, at individual locations, the difference was substantially higher.

VI-5.2 VERTICAL DISTRIBUTION

The trend for all of the projects was for the moisture content to decrease with increasing depth below the base. The average moisture content decrease ranges from a minimum of 1.9% to a maximum of 4.0% as the sample depth increases from 3 to 21 inches.

VI-6 CONSTRUCTION PROCEDURES

An analysis of the compaction data in Chapter IV show that the vast majority of the subgrade soils are compacted to densities between 95 and 105 percent of the maximum dry density as obtained from the AASHTO T-99 method of compaction, and at an average density close to 100 percent. The average moisture content is slightly below optimum, hence more than half the soils are compacted dry of optimum. In accordance with the literature study, (27), the subgrade soils should be compacted wet of optimum, and at lower densities. In order to reduce swell, the SDDOT (27) has stated that for expansive soils, the density should not be lower than 92 percent compaction with a target density of 95 percent. The placement moisture content should not be less than optimum, and the target moisture content should be 3 percent above optimum. This roughly corresponds to an average density of 95 percent, and an average moisture content of 3 percent above optimum. From this point of view, the soils are overcompacted by roughly five percent, and placed at an overall moisture content 3 percent too dry. Thus, for the projects investigated in this study, most of the subgrade soils fail to meet the specified criteria, and therefore may have a high swell potential.

Based on the Plasticity Indexes of the soils for the different projects it appears that the projects on US212 have the highest swell potential, while the lowest swell potential is on the projects on Highway 73 and Highway 34. This is not supported by the swell testing, however. Considerable swelling has already taken place on the section of US212 from LaPlant to the River, and the laboratory test most likely represent additional swell potential (See Table VI-3.) On the other hand, the potential for future swell on the section of US212 from Ridgeview to LaPlant is high if extensive cracking should develop. Such is also the case on Highway 34, where the average placement moisture content is at approximately 1 percent on the dry side of optimum, and the average degree of compaction is in excess of 100 percent. The swell potential on the section of US212 from Ridgeview west to 12.1 miles is also, based on the swell tests, high and evidence of increasing heaves at the cracks on this section is already evident.

The construction methods and subgrade fill handling used on the different projects are shown in Table VI-2. The method of undercutting may have an effect on swelling and heave development. On the projects on US212 from Ridgeview west 12.1 miles, and from LaPlant to the Missouri River, undercutting was performed from shoulder to shoulder, thereby leaving a trough in the undisturbed foundation soil where water may accumulate. These are the only two sections which show evidence of heaves significant enough to affect rideability. On all other projects, undercutting was performed from toe to toe, leaving free drainage from the subgrade to the ditches.

The method of fill preparation is also likely to have affected swell. On the section from LaPlant to the Missouri River, large intact shale fragments were placed in the subgrade, and did not get broken down during compaction, as evidenced by the sample taken from the subgrade (See Photo V-2). On all other projects, the subgrade fill was free of shale fragments, and have not experienced the same problems with heaves as the section from LaPlant to the River.

V-7. ASPHALT

As discussed in Chapter VI, the high frequency of transverse cracks occurring on the section from LaPlant to the Missouri River may in part be caused by abnormalities with the AC chemistry. It should also be mentioned that in December of 1989, a severe cold spell occurred between December 12 and December 22. On December 12, the maximum temperature was 0°C (32°F), while the minimum temperature on December 15 was -5°C (23°F). From December 13 through December 21, the daily temperatures fluctuated between close to -18°C (0°F) and -36°C (-33°F). This may have caused a "temperature shock", and resulted in rapid pavement shrinkage and subsequent transverse crack formation. The section of US212 from Ridgeview to LaPlant was paved in the summer of 1990, and was not subjected to similar temperature fluctuations during the following winter. In addition, AC-5 was used for the wearing course, which may also have contributed to the low frequency of transverse cracks.

It should also be mentioned that the pavement on Highway 73 from Faith South was paved during 1989 and 1990, and the portion of the pavement placed in 1989 was therefore subjected to the same cold spell the first winter after paving as the section on US212 from LaPlant to the River. This may have contributed to the high frequency of cracks (100 per mile) on portions of Highway 73.

CHAPTER VII

CONCLUSIONS

VII-1 GENERAL

The conclusions reached herein were mainly drawn from the observations, analysis of measurements and laboratory tests performed, and on previous experiences as stated in the literature. The results of the field investigation are presented in Chapter V, and the results are discussed in Chapter VI.

VII-2 CAUSES OF HEAVES

Based on the results of the investigation, it is our opinion that the heaves at the cracks on US212 from LaPlant to the Missouri River were caused by swelling of the subgrade soils, as water infiltrated the subgrade through the cracks in the pavement and the base material. All indications are, based on the distribution of moisture content in the base material and subgrade around the cracks, that the source of the moisture is through the cracks. Frost heave, which is partly irreversible, and depends on moisture content, may also have contributed. It is also believed that most of the heaves seen on US Highway 212 from LaPlant to the Missouri River during the late winter of 1993 were also caused by frost heave. Although the subgrade material hardly can be classified as frost susceptible, it is conceivable that with a moisture content of nearly 40 percent, approximately 5 percent expansion will take place as the water freezes.

This corresponds to a heave of approximately one half inch, which is close to that observed during the late winter of 1993.

It is believed that several factors are contributing to the severity of the heaves. These factors are summarized below:

1. The subgrade soils are highly overcompacted, with over half the material placed at moisture contents dry of optimum. This produced a subgrade material with a high swell potential.

2. Large fragments of relatively dry shale were placed as part of the subgrade and were not broken down by the compaction process. The examination of samples and observation of soils excavated from the test pit indicated that the shale fragments were close to saturated, hence the full swell potential of the shale was realized.

3. The undercutting was performed from shoulder to shoulder, possibly resulting in water pooling in the trough formed in the foundation material. This is supported by the test pit excavation, where an abrupt drop in moisture content was observed at the transition from the subgrade to the undisturbed foundation material.

4. Crack sealing was not done until two years following the surfacing. Since the cracks developed prior to and during the first winter following surfacing, significant amounts of moisture infiltrated the subgrade.

5. Due to the large seasonal variations in crack width (greater than 10 millimeters), it appears that the cracks seals separate from the walls of the cracks as they open again the following season. The crack sealing may therefore be ineffective.

VII-3 CRACK FORMATION

The high frequency of cracks on US Highway 212 from LaPlant to the Missouri River, may in part have been caused by components in the asphalt, and the temperature shock the pavement was exposed to during the first winter following paving. The fact that the pavement cracked shortly following placement is an indication that there were problems with the asphalt itself. It should be mentioned that the crack frequency on US Highway 212 from LaPlant to the Missouri River is not unique, however, the difference in behavior from the adjacent project from Ridgeview to LaPlant also indicates asphalt problems.

CHAPTER VIII

RECOMMENDATIONS

VIII-1 GENERAL

Several remedial measures may provide solutions to the heaves on US Highway 212 from LaPlant to the Missouri River. The longevity of the measures will vary with the measure and the maintenance requirements for the solution chosen. A range of solutions are outlined in subsequent sections. Complete reconstruction should also be considered, provided an impermeable membrane is used between the subgrade and the base material. Milling and overlaying the existing pavement may provide a short term solution, but is likely to require considerable maintenance in the future. Even though the heaves at the crack as caused by subgrade swelling may be tapering off, the moisture redistribution and moisture infiltration at new cracks will continue to cause heaves. The uneven moisture content distribution near the cracks will most likely produce seasonal heaves due to frost, and will be a problem for all measures except complete reconstruction. Remediation procedures that do not include measures to reduce or minimize reflective cracking and water infiltration through transverse cracks are in all likelihood going to require considerable amounts of maintenance and frequent resurfacing.

VIII-2 REBUILDING

Rebuilding may consist of complete reconstruction including the top one foot of the subgrade, or reconstruction from the base.

VIII-2.1 COMPLETE RECONSTRUCTION

It is likely that the best product will be obtained by removing the pavement and the base material and reworking the upper one foot of the subgrade by disking, moisture conditioning, and recompaction. An impermeable polypropylene membrane (12 mil thickness) should then be placed. In cut sections, the membrane should extend into the ditches and up the cut slope a vertical distance of 3 feet. In fill sections, the membrane should be extended 3 feet vertically down the slope. Alternately, the membrane may be buried vertically to a depth of eight feet at the outside of the paved portion of the shoulder. It is also recommended that a tack coat be applied to the membrane to minimize slippage between the base material and the membrane. The base material may then be placed over the membrane, followed by shaping and rolling of the base material and paving.

VIII-2.2 RECONSTRUCTION FROM BASE

The rebuilding may also be performed by rebuilding from the base. This procedure would consist of removal of the existing pavement, and shaping and rolling of the exposed base material. Excavation to the subgrade should be avoided, since it is unlikely that it will support construction equipment at its present water content. Following shaping and rolling of the base material, an impermeable membrane should be placed. In cut sections, the membrane should extend into the ditches and up the cut slope a vertical distance of 3 feet. In fill sections, the membrane should be extended 3 feet vertically down the slope. Alternately, the membrane may be buried vertically to a depth of eight feet at the outside of the paved portion of the shoulder. Following placement of the membrane, paving may commence. It should be noted that this procedure will not alter the moisture content distribution in the subgrade, and seasonal heaves due to frost may occur. However, since access of water to the subgrade from the surface will be minimized, moisture redistribution may take place with time and reduce the problem.

VIII-3 REMEDIATION

VIII-3.1 RUBBERIZED ASPHALT MEMBRANE

Based on the results of this study and the evaluation of construction and remedial procedures used by other states, it is recommended that the section of US Highway 212 between LaPlant and the Missouri River either be rebuilt from the subgrade up, or be overlaid using a procedure developed and successfully applied at the Arizona Department of Transportation. The remedial procedure involves the sealing of existing cracks, placing a 1.5 inch thick rubberized asphalt membrane over the existing pavement, and placing a new two inch thick wearing asphalt course above the membrane. Following placement of the membrane, chips should be placed over the membrane. It is also recommended, in cut sections, that the membrane be placed continuously across the pavement, into the ditches, and three feet up the cut slope opposite the ditch. In fill sections, the membrane should extend a distance of 12 feet down the slope from the shoulder. This is to prevent moisture from infiltrating the swelling soils and being drawn under the pavement by capillary action. The earth shoulder slopes should also be primed with a light shot of emulsion prior to placement of the asphalt rubber membrane. Following placement of the membrane on the slopes, a six inch thick layer of soil should be placed over the membrane.

VIII-3.2 MILLING AND PLACEMENT OF POLYPROPYLENE MEMBRANE

Low volume streets have been successfully reconditioned by milling, placement of a polypropylene membrane, and adding a new wearing surface. In areas where large variations in crack widths occur, however, it is unlikely that the polypropylene membrane alone will be able to withstand the tension developed without tearing as the cracks widen, and thus not be able to prevent reflective cracking.

VIII-3.3 MILLING OF PAVEMENT AND DELAYED PAVING

The moisture content in the subgrade may be redistributed and brought to equilibrium if the pavement is removed and replaced with a temporary, relatively pervious surface. In this regard, the pavement may be milled, rolled, and covered with a temporary bearing surface. This will allow the base and subgrade to "breathe", and the moisture content to equilibrate. A permanent surface may then be placed. The disadvantage with this method will be problems with rough rideability and possibly low bearing capacity until a permanent surface is placed. In the long run, this solution should require a minimum of maintenance, provided consideration is given to measures that will minimize water infiltration through pavement cracks.

VIII-4 COST

Based on information provided by the Wyoming and Arizona Departments of Transportation, the cost of a 12 mil polypropylene membrane is on the order of $1.10 per square yard placed. This cost includes trenching along the shoulders and the cost of the membrane to a depth of four feet. The cost of the rubberized asphalt membrane, including chips, is in the range of $1.80 to $2.00 per square yard. This cost does not include regrading of the shoulder slopes and placement of the soil.

VIII-5 MAINTENANCE

As concluded by this study, heaves at transverse cracks are primarily caused by water infiltrating the subgrade through the cracks, it is recommended that on new sections, crack sealing be performed following the first winter after paving, and not on a fixed time schedule of two years after paving as is the current practice. It is understood that the SDDOT uses different types of crack sealant depending on whether the sealing is routine maintenance or scheduled contracted, routing and sealing of cracks. Based on the observation of the performance of the crack seals, it is recommended that the SDDOT consider an alternate crack sealant for routine maintenance. Chip sealing is not related to the heave problem and can be performed according to the current scheduling. Due to the relatively large fluctuation in crack width, it is recommended that the South Dakota Department of Transportation initiate research on sealer material which is able to withstand the movements observed without separating from the walls of the cracks.

VIII-6 CONSTRUCTION PROCEDURES

On new construction as well as reconstruction projects in areas involving expansive soils, it is recommended that design procedures be adopted that minimize the infiltration of moisture through pavement cracks, and that construction procedures be directed towards minimizing the swelling of expansive soils. In order to minimize infiltration of moisture through pavement cracks it is recommended that the SDDOT adopt the use of polypropylene membranes placed between the subgrade and the granular base. In order to minimize subgrade swell potential, it is recommended that density and placement moisture criteria be implemented in accordance with those recommended below:

1. Subgrade material with high swell potential should be compacted at an average density of 95 percent, and with tolerances such that the standard deviation will be 3 percent or less.

2. The average placement moisture content should be above optimum. Preferably, the moisture content should be at three percent wet of optimum, with tolerances such that the standard deviation is less than 3 percent.

3. Disking should be used on all projects in order to break down shale blocks and to produce an even as possible moisture content.

4. Shoulder to shoulder undercutting should be avoided on projects where the subgrade material consists of low permeability soils. Shoulder to shoulder undercutting may produce a "tub" effect where water may accumulate in the cut.

VIII-7 ASPHALT MATERIALS

In order to minimize crack development, it is recommended that the SDDOT continue to monitor the performance of different AC material and aggregate mixes, and use those which have shown the best resistance to transverse cracks. It is also recommended that the SDDOT implement a study of the AC material on US Highway 212 from LaPlant to the Missouri River.

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