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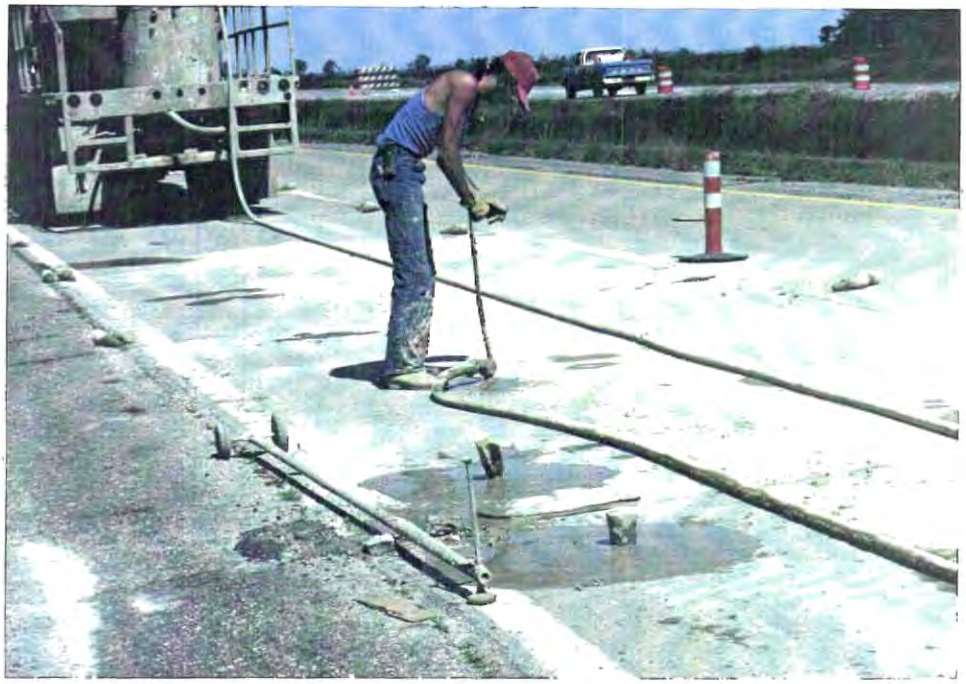

Evaluation of Undersealing of Undoweled Plain Jointed PCC Pavements in South Dakota

Study SD92-01
Final Report

Prepared by
South Dakota State University
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March 31, 1994

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16. Abstract

Pumping and faulting of undoweled Portland cement concrete (PCC) pavements has become a serious problem in South Dakota. Repair techniques to correct these distresses include diamond grinding, full/partial depth repairs, load transfer restoration, installation of edge drains, crack sealing, joint resealing, and slab stabilization (undersealing). Undersealing can be described as a method where a flowable mixture is pumped under the slab to fill the voids and restore support to the pavement. There is a disagreement within the South Dakota Department of Transportation (SDDOT) as to the use and effectiveness of undersealing as a rehabilitation technique. The objectives of the research were to describe the deterioration of load transfer and the development of faulting on undersealed and nonundersealed PCC pavements, to determine the effectiveness of undersealing, to determine when and where undersealing is appropriate, and to develop design, construction, and inspection guidelines for undersealing.

The work included a review of the literature relevant to undersealing; a national survey of current highway undersealing practices for PCC pavements; and field evaluation of four projects which were undersealed in South Dakota in 1987, 1989, 1992, and 1993.

Research results indicate that all undersealed pavement sections performed better than nonundersealed pavements. Lower faulting and corner deflections, and higher load transfer efficiency values were generally obtained for the undersealed pavements. Best performance was obtained immediately after grouting and performance generally degraded with time. Undersealing was effective in filling voids beneath the slabs as evidenced by the removal of two concrete panels. However, the cement-fly ash grout was generally cracked, suggesting that the grout is not durable for long-term performance. Undersealing, when done in conjunction with other rehabilitation techniques, can provide more than seven years of service life until additional rehabilitation or complete reconstruction is warranted.

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determination of the optimal time in the pavement life cycle to perform undersealing work, and adequate construction practices. There is a considerable disagreement within the South Dakota Department of Transportation (SDDOT) as to the effectiveness of undersealing as a rehabilitation technique.

1.2. Scope of the Study

Research objectives: The overall research objective was to determine the applicability, effectiveness, durability and proper guidelines for undersealing.

Specific objectives: The research study is intended to accomplish the following four specific objectives:

1. describe the deterioration of load transfer and the development of faulting on undoweled plain jointed PCC pavements,
2. determine the effectiveness of undersealing,
3. determine when and where undersealing is appropriate, and
4. develop design, construction, and inspection guidelines for undersealing.

Chapter II includes the review of the literature relevant to undersealing of PCC pavements. Chapter III summarizes the results of a survey that was submitted to all fifty state highway agencies and selected contractors. Chapter IV summarizes the results of the interview that was held with SDDOT personnel who are (or were) involved in undersealing related projects. Chapter V discusses the field performance of undersealed and non-undersealed undoweled plain jointed PCC pavement sections in South Dakota. The field data obtained from the removal of two concrete panels from two test sites are presented in Chapter VI. Results obtained from the PREDICT program are depicted in Chapter VII. Chapter VIII presents the cost analysis data. Findings, conclusions, and recommendations are presented in chapter IX.

CHAPTER II

LITERATURE REVIEW

2.1 Development of Pumping and Faulting in PCC Pavements

2.1.1 General

Pumping is a load-actuated erosion phenomenon by which material from the pavement foundation or shoulder is moved about by water. When fine material is removed from beneath the pavement slab by pumping action, there is a reduction in slab support, which leads to a greater slab deflection and more severe pumping. This process may lead to faulting at the joints if positive load transfer is not present. The rehabilitation strategies necessary to correct pumping and faulting of slabs are dependent upon the condition of the pavement. Such corrective actions may range from adequate pavement undersealing to complete resurfacing of the pavement.

2.1.2 Pumping

Pumping is the ejection of water and fine materials from the slab foundation through joints, cracks, and edges. This leads to the formation of voids under the slab which increases stresses and strains in the pavement. Continuous pumping leads to slab faulting and cracking, and therefore pumping should be considered in any rigid pavement design and analysis.

Pumping can be detected visually from the presence of fine material near the joints and cracks. Joint and crack faulting is another indication of pumping. The presence of voids beneath the slab can be detected through the use of nondestructive techniques such as the Falling Weight Deflectometer (FWD) and the Ground Penetrating Radar (GPR).

In order for pumping to occur, the pavement must move down under the traffic load, together with the presence of trapped free water and fine particles under the slab. When addressing pumping, there are several factors that should be considered. They include traffic, subgrade soils, subbases/bases, slab properties, and the surrounding environment.

Traffic

The cumulative traffic loading (18-kip ESAL) was found to be the most significant variable affecting pumping (1). Neal (2) has observed that in dual-lane pavements, almost all the pumping occurs in the outside lanes which are normally used by heavy slow trucks. Pumping also occurs in low speed zones such as on uphill grades (2). The distribution of axle loads during the day can also be important. Loads applied on the slab in the curled position (morning time) can be more severe than when the same loads are applied in the middle of the day (3). The number of heavy vehicles, the magnitude of the axle loads, and tire pressures have increased appreciably in the last few years. As a result, the severity of pumping and faulting and the extent of their occurrence have increased.

Subgrade Soils

Generally, coarse-grained soils such as sands, sandy clays, and sandy clay loams have exhibited little pumping. Fine-grained subgrades in which silt and clay sizes predominate were found to be susceptible to pumping. This behavior can be related to the ability of coarse-grained soils to drain free water from the pavement structure. Pumping was also found to occur in granular soils where the plasticity index (PI) of the fine material is more than 7 percent (2). Pumping can be delayed if the subgrade is compacted to its maximum dry density at the optimum moisture content (2). Also, fine-grained subgrades can be a major source of pumping water through capillary action.

Sub-bases/Bases

Pavements built over well-graded subbases where more than 50% of material is gravel and sand have shown little or no pumping (2). However, the increases in heavy vehicle axle loads and tire pressures enforce the necessity for providing non-erodible subbases. Non-treated subbases can be a major source of fine material produced through erosion. Bases constructed of lean concrete or asphaltic concrete were found to be satisfactory and much more resistant to abrasion (4). The presence of an asphaltic curing membrane between the pavement slab and the subbase will increase the possibility of pumping. Caltrans researchers (4) found that cement-treated bases will not provide a non-erodible support for the pavement slab all the time.

Slab Properties

When the slab thickness and/or the modulus of rupture of concrete is increased, the slab resistance to deflection will be increased which in turn reduces pumping (5). Figure 2.1 shows that increasing the pavement thickness from 9 inches to 15 inches

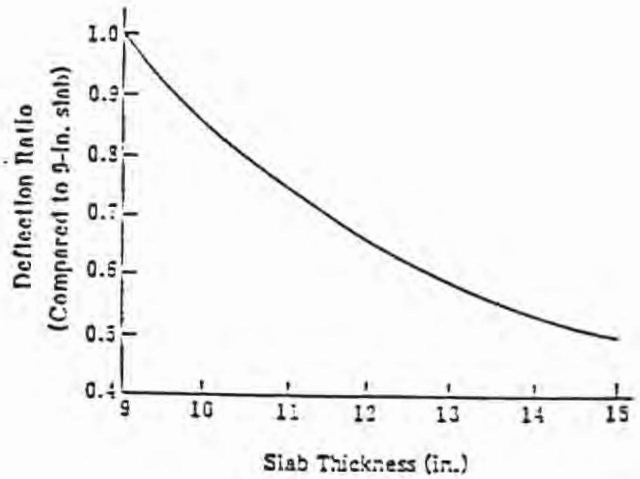


Figure 2.1. Deflection Versus Slab Thickness Based on Westergaard's Corner Theory (5).

will reduce deflection by 50% (5).

Pavements having longitudinal subdrains will exhibit less visible pumping than those without drains. Mix design of the concrete, aggregate type, the permeability of the concrete, and the side slopes of the pavement will all work together to minimize surface water penetration to the subbase and the subgrade. Neither joint sealants nor load transfer devices were found to be effective in preventing pumping (2). These devices, on the other hand, have reduced the magnitude of faulting at joints which pump. The longer the expansion joint, the less likely pumping will occur (2). No relation between pumping and pavement age was found.

Environmental Factors

The amount, intensity, and distribution of precipitation determine the availability of water which is a main factor contributing to pumping. Temperature change causes an upward warping at the edges of the slab. This warping reduces base support and results in the separation of the slab from the base at the joints and the edges. The number of freeze-thaw and wet-dry cycles affects the erosion potential of the pavement base/subbase (3).

In summary, the factors involved in the pumping of PCC pavements are as follows: (1) free water under the slab; (2) unstabilized or erodible material under or adjacent to the slab; (3) cracks or joints in the pavement; and (4) deflection of slabs under heavy moving wheel loads. If any of these factors are absent, pumping will not occur.

2.1.3 Faulting

Faulting is the differential vertical displacement of abutting slabs at joints or cracks creating a "step" deformation in the pavement surface. As shown in Figure 2.2, the low slab is always the one that vehicles contact after crossing the joint.

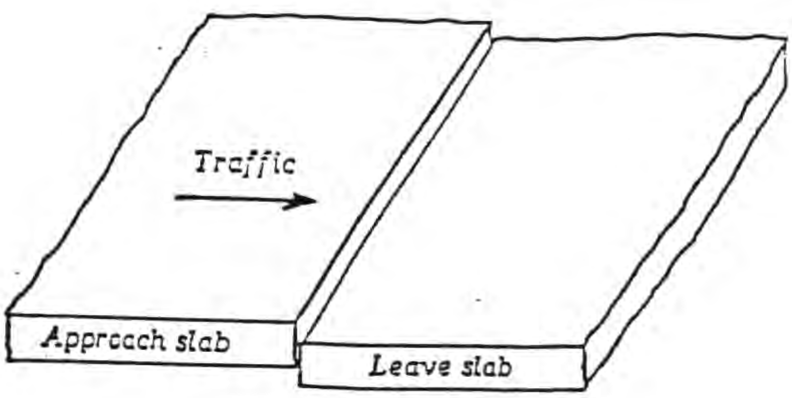


Figure 2.2. Faulting Configuration (6).

The loss of support from beneath the slab due to excessive pumping causes a stair step effect on the pavement surface. Figure 2.3 shows the development of faulting in undoweled plain concrete pavements built over cement-treated bases (5). Faulting can also occur when a joint is hand-finished leaving one side of the slab higher than the other side (5). Faulting most often occurs at transverse joints and is associated with the loss of load transfer across the joints. Faulting is difficult to recognize visually in its early stages. Faulting begins almost immediately after a pavement is opened to traffic and increases with time (4). Caltrans researchers (4) found that the slowest rate of faulting increase occurs in the semiarid regions while the greatest rate often happens in the mountain and coastal regions where more rainfall or equivalent snowfall occurs. Those pavements constructed in the valley region exhibit faulting rates between the two extremes.

Woodstorm et. al., (7) have classified four stages of faulting in concrete pavements and they are as follows:

1. Stage 1: Initial step-off less than 0.0625 inch is observed with shoulder cracks on the down traffic side of the joint.
2. Stage 2: The step-off will increase more than 0.0625 inch. The cracking will be extended longitudinally and is accompanied by a shoulder depression in the vicinity of the joint. This step-off causes sufficient

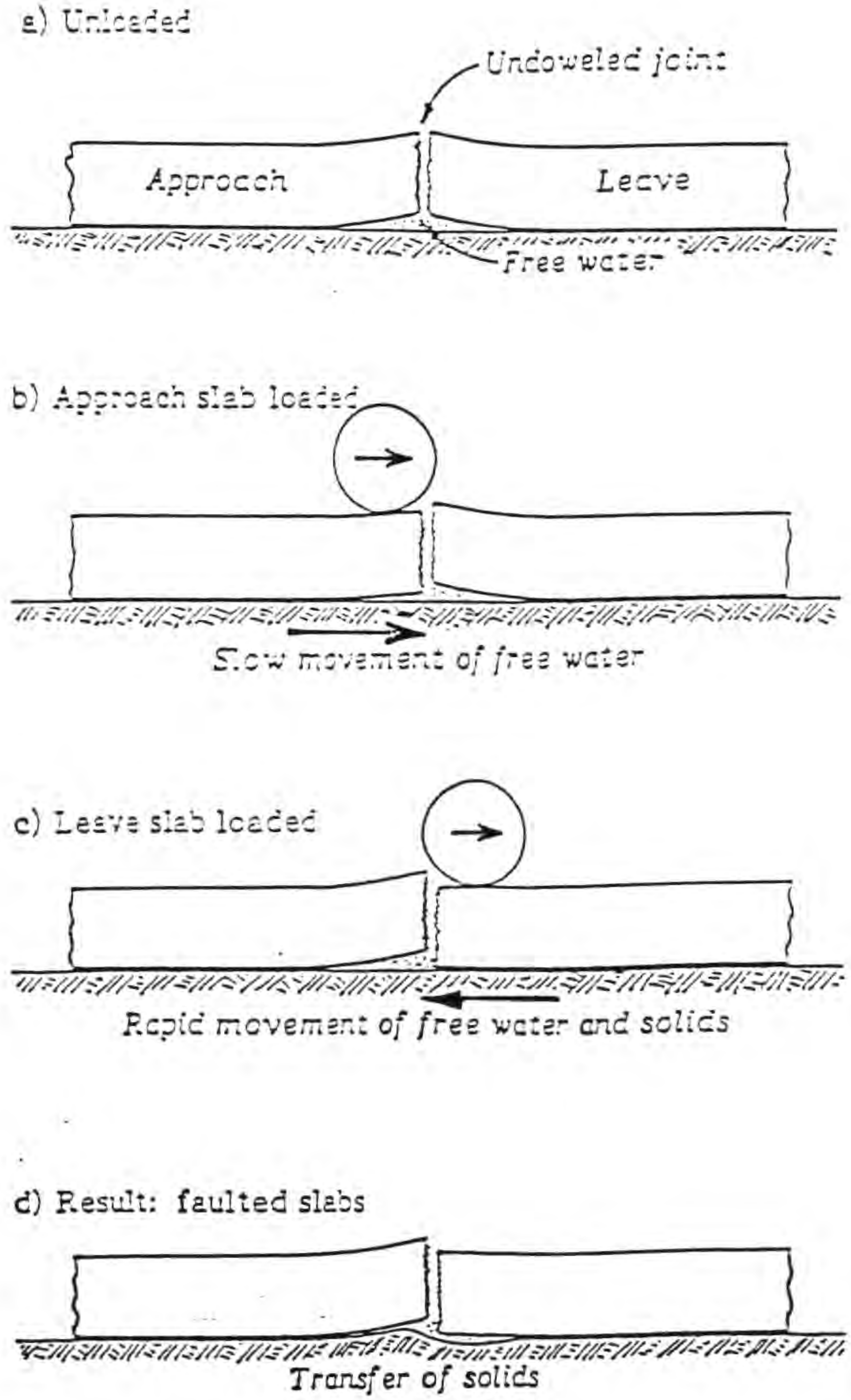


Figure 2.3. Development of Faulting in Undoweled Concrete Pavement Built Over Cement Treated Bases (5).

audible noise from the impact of the tire.

3. Stage 3: The riding quality of the pavement becomes very rough. This stage occurs when the fault exceeds 0.125 inch which will cause annoying shock to the motorist.
4. Stage 4: The slab condition will no longer be able to carry the traffic.

Maintenance is to take place as soon as stage two symptoms are recognized. The depressions are to be filled with localized patches and the cracks are to be filled with a suitable sealant. To restore the riding quality in stage 3, grinding the pavement together with removing a tapered segment from the slab surface is the best solution. An overlay becomes a necessity if pavement faulting is in its fourth stage.

It is possible to prevent or reduce faulting by eliminating those factors that cause and/or aggravate faulting. Some of these prevention methods can be as follows:

1. Avoid the use of erodible bases and untreated shoulders. Lean concrete bases have shown the best results (4). The effect of different combinations of granular and stabilized bases with and without dowels on the development of faulting in jointed plain concrete pavements (JPCP) is shown in Figure 2.4 (1).
2. Provide a drainage system that is capable of

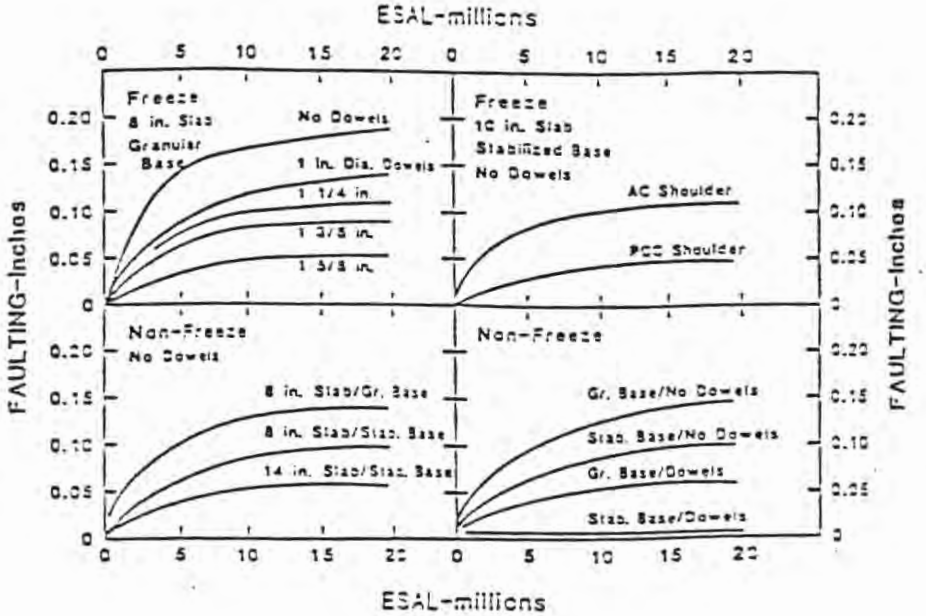


Figure 2.4. The Effect of Different Combinations of Granular and Stabilized Bases on the Development of Faulting in JPCP (5).

draining water from under the slab.

3. Periodic stabilization, if possible, of the loose fines under the pavement slab (7).

4. A joint sealant that will prevent surface water and incompressible foreign materials from entering the openings.

5. Sealing the cracks and providing surface overlays. These overlays will not only prevent surface water from entering the slab but will also sometimes improve the structural capacity of the pavement.

6. Increasing the slab potential to resist deflection. This can be achieved by increasing the slab thickness either in the original pavement design or by providing overlays. Until now, it is difficult to predict what is the minimum slab thickness that will prevent the occurrence of faulting.

The maintenance procedures necessary to correct pumping and faulting problems are dependent upon the condition of the pavement and they can involve one or more rehabilitation techniques. These include slabjacking, undersealing, full/partial-depth patching, load transfer restoration, overlays, grinding, crack sealing, joint resealing, and subdrainage.

2.1.4 Rehabilitation Alternatives

Slabjacking

"Slabjacking" or "mudjacking" or "pressure grouting" can be defined as the injection of grout under the pavement in order to fill the voids and to raise the sunken slab to its correct grade. The jacking of a concrete pavement offers the necessary means to achieve longer life and better rideability. Slabjacking has been a vital part of highway maintenance strategies in the United States for the past 60 years. The grout material exerts an upward pressure on the slab which is dependent on the viscosity of the grout and the depth of the voids (5). A successful slabjacking is one which yields a well-distributed support below the slab and at the same time fills all the existing voids.

Hole-drilling patterns for pavement jacking should be determined in the field by the crew superintendent (8,9). The hole patterns should take into consideration the size or the length of the pavement to be raised, the elevation difference, the subgrade conditions, and the location of joints and cracks.

Slabjacking can be carried out through vertical holes drilled in the pavement or through pipes driven horizontally beneath the slab. In the State of Washington (10), it is found that "coning" of the slurry directly under the vertical holes without flowing evenly under the slab will cause many cracks to radiate from these holes. Horizontal slurry injection does not interfere with the flow of traffic as with the vertical-hole method, and therefore it is safer and less costly for traffic control. For the purpose of avoiding slab cracking, lifting of the slab should be controlled and should be done in increments of 0.125 inch (8,9). The volume of the required grout is 20 to 25 percent more than the volume of the visible settlement (11).

The grout material used for slabjacking should be durable, non-erodible, have adequate compressive strength, and enough flowability to fill all the voids, and can not be displaced laterally after it has lost fluidity. Typical materials used in grouting include: sand-cement grouts, cement-pozzolan grouts, cement-lime grouts, silicone rubber foam, polyurethane and others.

Subsealing

"Subsealing", "slab stabilization", "voids filling" or "undersealing" is another method to return slab support to the pavement and possibly retard the rate of deterioration caused by the development of voids under the pavement. Subsealing can be performed by pumping a flowable mixture under the slab to fill the voids and restore support to the pavement (6). Subsealing does not eliminate faulting, but rather prevent further faulting and slab deterioration. Further details about subsealing will be provided later in the chapter.

Full-Depth Patching

Patching is the removal or the replacement of the existing surface. Full-depth repair is required to restore the structural integrity of deteriorated slabs. The areas of the slab that has been affected by faulting should be clearly defined. A full depth saw-cut around the total patched area should be carried out in order to avoid damaging the surrounding concrete. The existing subbase/base should be cleaned and uniformly compacted.

Overlays

The application of a properly designed overlay can provide a cost-effective method for correcting faulting. The overlay could be an asphaltic cement concrete or Portland cement concrete surface. When applying asphaltic overlays, consideration should be taken to avoid future reflective cracks. Also, before applying the overlay, the slab should be stabilized by either undersealing or breaking the slab into small sections (5).

Grinding

Grinding can be defined as "the grinding out-of-spec concrete flat work to tolerance without causing any damage or lessening the structural integrity of the surface (12)." The various types of grinding machines used today smooth faulted highway joints to an impact-free-surface by shoving away the high side of the joint and blending it with the low side. Grinding, as well as overlays addresses the symptoms but not the cause of faulting. The inclusion of grinding will lead to a better riding quality and a higher skid resistance of the pavement.

Subdrainage

The quick removal of water from under the slab is very essential in reducing the progression of pumping and faulting. A subdrainage system is usually provided during the construction of pavement and shoulders, but it can also be added to an existing pavement. Neal and Woodstrom (7) conducted several studies on the different types of subdrainage systems and found that the saturation period was significantly reduced when using subdrainage techniques. A recent development of a transverse drainage system allows the retrofit installation of slotted drain pipes in the existing pavement subbase beneath major transverse joints and cracks (13). Figure 2.5 shows the installation procedure for such a system (13).

2.2 Undersealing of PCC Pavements

2.2.1 General

The practice of injecting grout material under concrete pavements for the purpose of stabilizing slabs and controlling pumping has been used extensively in the United States for

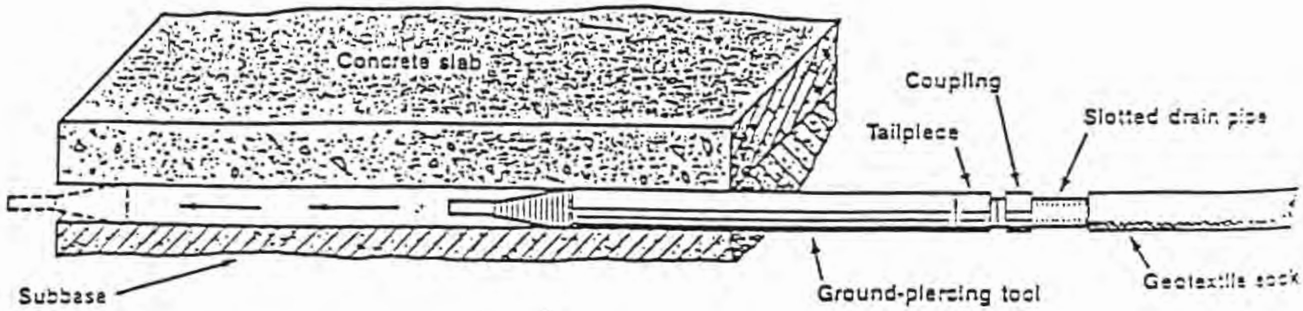


Figure 2.5. Retrofit Installation of Drain Pipe Below Transverse Joints and Cracks (13).

several years. Slab undersealing restores support for the faulted pavement structure by filling the voids under the slab with a suitable grout without intentionally raising the pavement. Figure 2.6 shows the mechanism of an undersealing process (6). Because undersealing does not correct faulting, pavement resurfacing or grinding should be considered for the stabilized slab in order to enhance riding quality and increase the Serviceability Index (SI).

James et. al., (14) stated that pavement undersealing performed in conjunction with patching and resurfacing is an effective rehabilitation technique. If no provisions are made to prevent infiltration after undersealing the slab, the pumping process may resume (15). These provisions must include resealing of joints and cracks and improving the drainage system.

The undersealing process has not been modelled yet and there

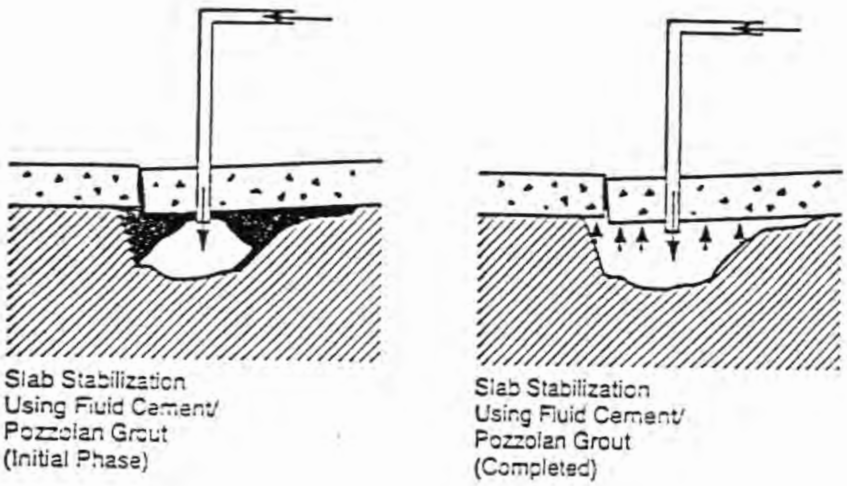


Figure 2.6. Mechanism of an Undersealing Process (6).

is a lack of coordination of undersealing experience among the states (16). This causes greater hesitation by states to use undersealing as a rehabilitation technique.

The basic problems associated with undersealing are as follows (15):

1. Determination of the grout material properties (type, flowability, durability, and quantity).
2. Determination of proper construction procedures.
3. Determination of which and when slabs should be undersealed.
4. Determination of void sizes.
5. Determination of the effectiveness of the

undersealing, and

6. Assessment of life expectancy and unit costs associated with undersealing.

2.2.2 Determination of the Grout Material Properties

The first research on undersealing materials was conducted by A.C. Benkelman in 1931 (17). Benkelman found that the sand content in cement grout mixtures should not be below 50% and that the organic material should be as low as possible. Since then, several materials have been tried as undersealants with an objective to reduce future distresses in the pavement. It was always found that distresses would somehow resume after a short period of time. The type of grout used in undersealing is an important factor in obtaining a good, stable slab. The flowability of a stiff material and its ability to distribute and fill narrow voids under the slab is somehow not significant. However, if the material has low viscosity and is soupy, the strength characteristics of the grout will be low and shrinkage will be expected. Generally, a stiff grout is needed for achieving strength, and a more fluid mixture is needed for filling the voids. The different types of undersealing materials include bituminous materials, cement-limestone dust grouts, sand-cement grouts, cement-pozzolan grouts, cement-water grouts, epoxies, silica foam grouts, and methymethacrylate. A FHWA survey (18) revealed that 11 states have used asphalt-cement undersealing and 26 states have used cement-grout undersealing. However, another FHWA study (16) revealed that based on economics, ease of application, and compatibility with the available mixing and pumping equipment, no material has demonstrated a clear superiority over the others.

Bituminous Materials

Various states have used different types of bituminous materials for undersealing concrete pavements. In 1943 and 1944 the State of Ohio (19) used a grade A oil asphalt filler (AASHTO M18-42A). This material gave very satisfactory results with no bitumen exuding out from joints and cracks. The asphalt was heated from 350° to 400° F. The success of oil asphalt filler was due to the fact that it forms a tight seal beneath the pavement and thus prevents the entrance of water. Furthermore, the stability of the sealant was not affected by the water coming from the subgrade. The fact that bituminous material spread more evenly than slurries will make it easier to control when being pumped under the slab (19). Also, the State of Missouri achieved the same good results when the oil asphalt filler was used (20). However, a study conducted by the State of Illinois (21) revealed that oil asphalt filler appears to be brittle and cracks at low temperatures. This will damage its intended function as a flexible barrier against the pumping of subgrade fines under heavy traffic.

Within normal climatic changes, No. 0 Korite is less susceptible to changes in physical characteristics with changes in temperature than the oil asphalt filler (21). Thus, the use of No. 0 Korite is more preferable as an undersealing material than the oil asphalt filler.

In 1952, the State of New York made a statewide investigation of concrete pavements joint support (22). The result of the investigation indicated that asphalt emulsion which was pumped under pressure had filled the voids beneath the pavement and stabilized about 1 to 2 inches of the subgrade. The study found that heating the emulsified asphalt from 130° to 140° F before pumping will yield good results.

However, another study completed by the State of New York in 1964 (23) indicated that bituminous materials should not be used as undersealing materials because they are greatly affected by temperature changes. Rapid cooling may prevent complete filling of voids below the slab. Furthermore, the extreme high temperatures that are needed to heat bituminous materials make undersealing a potentially hazardous operation and great care must be taken to adequately safeguard the crew (20). Using a regular paving asphalt cement grade could lead to large amounts of asphalt cement extruding out of the joints onto the pavement surface in the future (24).

In 1980, the State of Indiana (25) began using oxidized asphalt as a new undersealing material. Oxidized asphalt tends to spread throughout even the smallest voids seeking out all poor support areas in the base. From a single hole in the pavement, asphalt was able to spread up to 12 ft in the horizontal direction and thus reduce the number of drilled injection holes. Indiana's specifications call for 15 to 30 penetration asphalt with a softening point of 180° to 200° F. The hard material was used to restore support and the high softening point was intended to eliminate seepage problems on hot days.

The Asphalt Institute (24) recommends the use of an asphalt-cement that meets the requirements of asphalt for undersealing Portland cement concrete pavements as per AASHTO M238 (ASTM D3141). AASHTO M238 specifications are given in Table 2.1 (24).

Cement-Limestone Dust Grouts

The limestone-dust was used for the first time with cement grouts in New York in the early 1960's (23). A wetting agent was added to the mix in order to reduce surface tension and increase flowability. The limestone-dust particles should be spherical in shape rather than flat platelet grains in order to have good flowability. The limestone-dust used for undersealing should meet the requirements of AASHTO M17 for a mineral filler (26). However, the FHWA (18) requires the use of ground limestone that meets the requirements of AASHTO M216. A minimum of 95 percent of the material fraction should pass the No. 30 sieve and a minimum of 30 percent should pass the No. 200 sieve.

Georgia (15) tested various types of cement grouts and found that the best strength was obtained by using cement and limestone dust. However, Louisiana (27) stopped using cement-limestone dust mixtures and recommended the use of sand-cement grouts.

Sand-Cement Grouts

In 1976, the State of Mississippi (28) conducted a study covering the use of fine sand in combination with different

Table 2.1. Requirements for Asphalt-Cement Materials Used in Undersealing Portland Cement Concrete Pavements (AASHTO M238) (24).

Asphalt Cement Property	Minimum	Maximum
Ring and Ball Softening Point	180°F (82°C)	200°F (93°C)
Flash Point (COC)	425°F (218°C)	
Penetration:		
32°F (0°C), 200 g, 60 seconds	5	
77°F (25°C), 100 g, 6 seconds	15	30
115°F (46°C), 50 g, 5 seconds		60
Ductility:		
77°F (25°C) 5cm/min, (cm)	2	
Loss on Heating:		
at 325°F (163°C) For 5 hours (%)		.5
Penetration of Residue (% of original)	70	
Bitumen Soluble in Trichloroethylene (%)	99	

Note: Except for solubility requirements, the technical requirements of this specification agree with ASTM D3141.

cement types and various admixtures as undersealants. The results indicated that the best mixture is a blend of fine sand with Type I Portland cement and flake calcium chloride. The recommended sand gradation is presented in Table 2.2 (28). Calcium chloride was used to accelerate the setting of the grout and thus return the pavement to traffic use as soon as possible. Coarser sand gradation will be more abrasive to the equipment and the particles will not stay in suspension thus giving less workability and flowability to the mix. Also, the angularity of the sand particles may make it difficult for the grout to flow into narrow and discontinuous spaces (11). Another problem associated with sand-cement grouts is that sand particles frequently come out of suspension under pressure. The use of bentonite and other materials to keep the sand particles in suspension has a negative impact on the grout strength (11). The use of sand slurries could also lead to infiltration of sand particles into transverse joints causing blowup problems (24).

Cement-Pozzolan Grouts

Cement-Pozzolan grouts have a definite advantage over other materials and are becoming more widely used in undersealing. The properties responsible for enhancing pozzolanic grouts include particle shape, gradation, and pozzolanic activity of the pozzolanic material. In addition to the silt-sized particles, the pozzolanic material contains a small and effective amount of clay-sized particles. The clay-sized particles fill the voids in the pozzolan matrix which increases the durability

Table 2.2. Sand Requirements for Grout Mixes (28).

Sieve Size	Percent Passing
No. 10	100
No. 60	40-90
No. 200	0-50
Percent Silt	0-25
Percent Clay	0-12
Plasticity Index (PI)	NP
Organic Content	0-5

NP: non-plastic

of the grout and reduces segregation during pumping and injection. The pozzolanic materials have spherical and smooth particles which makes pozzolanic grouts easier to pump than those grouts having angular sand. The spherical shape is necessary to enhance the flow characteristics of the grout and to increase its ability to fill narrow voids. Since the hydration of cement produces calcium hydroxide ($Ca(OH)_2$), additional cementation will be generated when pozzolans are mixed with cement. Ultimately this produces a stronger mix (11,24). Generally, pozzolanic grouts become nonsoluble, nonerodable, and incompressible sealants after setting. However, laboratory tests have shown that different reactions could occur when adding cement to pozzolans from different sources (29). Therefore, each admixture must be tested and evaluated prior to final approval. Georgia (15) indicated that pozzolanic grouts loose strength due to the addition of water needed to obtain a good mixture consistency.

An Illinois study (14) found that pozzolan-cement slurries achieved better strength and higher flowability than cement-limestone dust slurries. If fly ash is used in the grout, it should meet the requirements of ASTM C618 for mineral fillers (26). A typical mix design can be as follows (24,29):

- 1 part Type I or II cement. Type III can also be used for early strength,
- 3 parts pozzolan,
- Water for proper fluidity,
- Accelerator may be used if the ambient temperature is below 50°F,and
- Other additives.

Cement-Water Grouts

Cement-water grouts are preferred to be used where the voids are small or when poor filling indicates that regrouting is necessary (29).

As indicated earlier, cement-grouts are used more often for undersealing than asphaltic materials. The State of Georgia (30) approved the use of the five mix proportions as presented in Table 2.3. Georgia uses Type 3 grout for most of its projects. Types 1 and 2 grouts are used in special cases where there are large voids under bridge approach slabs. The relation between

Table 2.3. Grout Mixes Used in the State of Georgia for Undersealing (30).

Materials	Grout Type				
	1	2	3	4	5
Cement	25	25	25	25	100
Limestone Dust		25	75	50	
Fly Ash	25			25	
Fine Aggregates	50	50			

the seven-day compressive strength and the water to cement ratio of various cement grout mixes is shown in Figure 2.7 (15).

Testing of Grouts

The grout mixture must have enough flowability to penetrate very thin voids. Furthermore, it must have sufficient strength and durability to resist loading, moisture, and temperature effects.

The fluidity of the cement grouts is measured using a flow cone specified by ASTM C939. The time required to empty the cone from a specific level measures the flowability of the grout mixture. The flow in seconds (also called time of efflux) can also be determined using the Corps of Engineers Test Method CRD-C611. Typical flow times for cement-limestone dust grouts are 16 to 22 seconds. Cement-fly ash grouts have flow times ranging from 10 to 16 seconds (10).

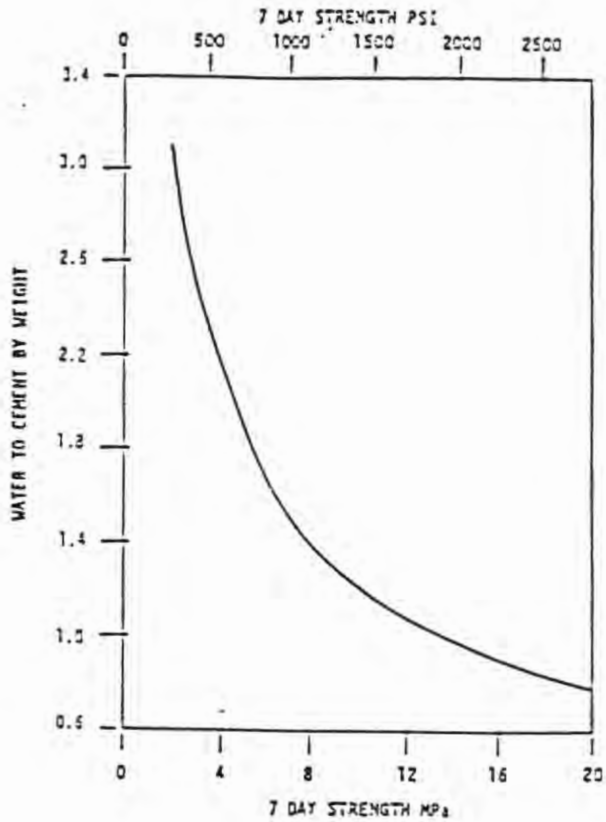


Figure 2.7. The Relationship Between the Seven-Day Compressive Strength and the Water to Cement Ratio of Various Cement Grout Mixtures (15).

Several laboratory tests should be conducted on the grouts. These tests include AASHTO T197 (Proctor Needle test) to determine the setting time (usually between 4 and 6 hours) (6). However, thin layers of grout under the pavement may set much more quickly. The compressive strength of the grout should also be tested. Caltrans (29) recommends a seven-day compressive strength of 750 psi to prevent grout erosion. However, the seven-day strength should not be less than 800 psi as measured using AASHTO T106 or ASTM C109 (13).

Water should be tested to make sure that it is free from injurious quantities of soil, acid, alkali, or vegetable matter. The water should also be reasonably clear and should not be brackish.

Other tests may be performed on grouts include density, specific gravity, and shrinkage.

Consistency tests should also be performed on asphaltic sealants. Such tests include the penetration, viscosity, flash point, and softening point.

Additives

Various additives may be specified to enhance the properties of the grout. Most of these additives are added to increase the flowability and the strength of the grout. For example, the addition of powdered ammonium lignin sulphonate increases the fluidity, the strength, and the density of the grout without adding water or cement (11).

Additives must be tested and evaluated in the laboratory prior to their use with pozzolanic grouts. Different sources of pozzolanic materials will react differently with the same admixture. An Illinois study (14) found that superplasticizers added to fly ash grouts will yield the best results.

Generally, calcium chloride is added to accelerate setting time and expansion agents are added to offset shrinkage. Other types of additives that may be used include fluidifiers and water reducers.

Estimation of Grout Quantities

Material quantities required for grout undersealing vary considerably depending upon the conditions of the pavement and the pattern of the grout injection holes (29). Also, the quantity needed is highly dependent upon the skill of the crew who are monitoring the grouting process (24).

To estimate the quantity of cement-grouts required to underseal faulted slabs, the following procedure could be followed (31):

1. Select a representative section of the pavement within each mile and measure corner deflections using the Falling Weight Deflectometer (FWD).
2. Plot the deflection data on a graph and determine the proportion of the joints that have loss of support.
3. The estimated overall dry grout quantities needed to stabilize the slabs is calculated using this equation:

   GROUT = PJG * AGT * TNJ

   where: GROUT = Total grout quantity for the project (ft^3 of dry materials).

   PJG = Proportion of joints requiring grouting.

   AGT = Average grout-take per joint grouted (typically 1 to 3 ft^3).

   TNJ = Total number of joints in the project.

   It is recommended to add 10 to 20 percent more grout to the total quantities in order to allow for some over run and for regrouting of some joints (24).

The Asphalt Institute estimated that 10 gallons of asphalt will be needed per hole in order to stabilize slabs with minor voids and low severity pumping (24). For projects with extensive pumping, the required asphalt will vary from 30 to 40 gallons per hole. The State of Indiana (25) has shown that 15 to 18 gallons of oxidized asphalt per hole will be sufficient to stabilize jointed pavements.

2.2.3 Determination of Proper Construction Procedures

Injection Holes

Care must be taken while drilling injection holes. A downward pressure exceeding 200 psi can cause breakouts at the bottom of the slab (6,18). During drilling; debris, dust and gravel may plug the voids which results in stopping the grout from flowing into the void paths. Therefore, water or air must be blown into the injection holes before grouting begins to flush away the debris.

The holes should be vertical and made carefully so that they are circular in shape. The diameter of the hole could range from 1.25 to 2.125 inch depending on the agency specifications. However, Louisiana (27) found no advantage or lifting superiority attributable to larger hole sizes. Depth of the grout holes will vary depending on the underlying materials and the construction practices (18). If a granular, non-stabilized subbase is present, the drilled hole should extend a few inches below the bottom of the slab because the depth of the void is uncertain (18,24). With stabilized subbases, the hole should be drilled to the bottom of the subbase and reach no more than 3 inches into the subgrade because voids are generally located between the subbase and the subgrade (18,24).

For safety and economic reasons, drilling should be done just prior to the pressure grouting process so that both crews can be protected by the same traffic control devices (29). Also, it is better to confine the drilling and grouting operations to a single traffic lane at one time.

The number of drilled holes will significantly affect the cost of an undersealing project and such costs depend on the hole pattern (16). The hole pattern depends further on the void size and location and on the design type of the PCC pavement. However, if after grouting a flow between the holes cannot be achieved, additional holes should be added to the existing hole pattern. Indiana (25) used one hole for each slab section to underseal with oxidized asphalt. Figure 2.8 shows a typical hole pattern used in undersealing plain concrete pavements.

Pumping and Monitoring

The bottom of the injection pipe should not be lower than the bottom of the slab (18). This will allow grout to fill all voids below the concrete slab as well as those voids that exist below the subbase. Overgrouting shall not be allowed because it will cause unnecessary lifting and thereby create additional voids and/or induce more stresses in the slab.

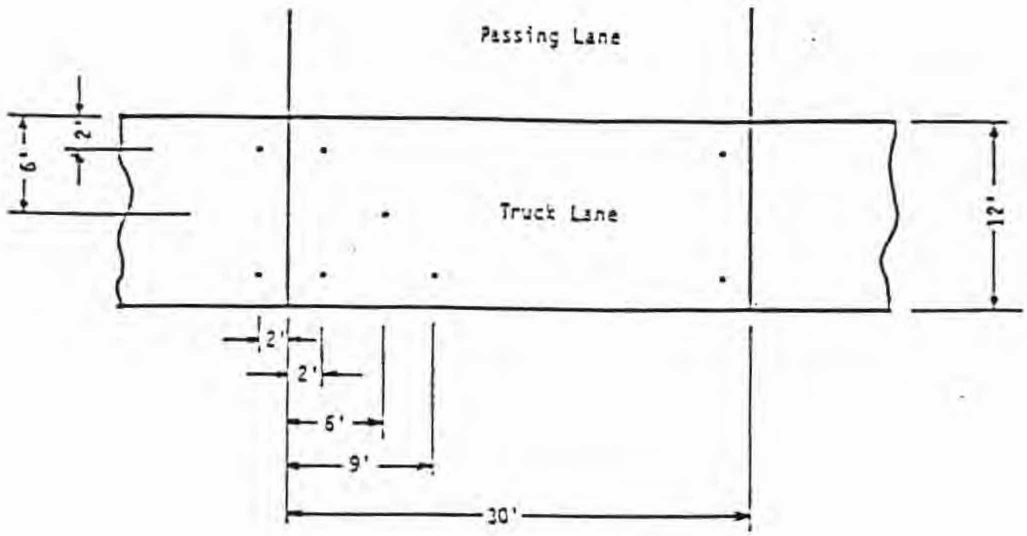


Figure 2.8. A Typical Hole Pattern Used in Undersealing Plain Concrete Pavements (18).

The maximum allowable continuous pumping pressure should not exceed 125 psi (18). Typical pumping pressure should be in the range of 40 to 60 psi (6). However, very low pressures will prolong the injection time. At the start of pumping, a surge of up to 300 psi is sometimes encountered for a maximum of 3 seconds (18). This surge will help the grout to overcome the friction in the grout lines and help free any debris blocking the injection holes.

During grouting, movement of the slab must be monitored by using sensitive dial gauges. The gauge must be capable of reading movements of one thousands of an inch (6). The base of the gauge should be placed on the shoulder, 3 to 4 feet from the slab that is being monitored. The injection packer should have a positive cutoff valve and a return line for recirculation of the undersealant. The undersealant should be stopped whenever one of these conditions is reached (18):

1. The maximum allowable pressure of 125 psi is obtained (except for the initial pumping pressure which could reach up to 300 psi),
2. The slab lift at the corners approaches 0.125 inch. Some agencies require maximum uplift of 0.05 inch (6,14),
3. The grout is observed flowing from adjacent holes, cracks, or joints,
4. A maximum pumping time of 10-30 seconds depending on flow rates and local conditions (16), or
5. The grout is being pumped unnecessarily under the shoulder as indicated by lifting of the slab.

After grouting has been completed, the packer should remain in the injection hole for 30 seconds to allow the undersealant to harden sufficiently (16). After removing the packer, the hole should be plugged immediately with a temporary tapered wooden plug. However, Georgia (18) found that it is better to leave the grouted holes open without the temporary plugs so that extra grout can escape out. After removing the wooden plugs, each hole should be filled with a reasonably stiff grout or an approved concrete mixture and compacted.

When a minimum of 24 hours has elapsed after the completion of an undersealing operation, the slab movement should be measured by means of deflection devices (29). If high deflections have been measured (more than 0.025 inch), additional grouting using new holes should be done (18). The process is repeated at a minimum of 24-hour intervals until stability is achieved. However, no matter what the stability results are, no more than two properly performed grouting operations should take place unless the engineer approves a third grouting that reaches the subgrade (18).

As a final step in the construction sequence, the grout remaining on the pavement and shoulder should be washed off to avoid unsightly discoloration. After a sufficient time has been allowed for the setting of the grout, the road could be opened to traffic.

Equipment For Cement Grouts

Equipment needed for a cement grout undersealing operation should consist of the following items:

1. A grout plant: It must be capable of accurately measuring, proportioning by volume or weight, and mixing the various materials composing the grout. The plant must also contain a pump capable of applying a pressure of at least 50 to 250 psi at the end of the discharge pipe. To provide quicker cutoff times for the grout, a return hose should be

specified to minimize slab movement or overgrouting and to provide better control of the injection pressure. To eliminate the problem of initial set in the injection hose, a return system should be provided (18). Pozzolan grouts require the use of a colloidal mixture to achieve a true colloidal mix that will stay in suspension and resist dilution by free water. It is not recommended to use ready mix trucks because they are designed for aggregate mixes and not for cement grouts (18).

2. Air compressors to drive pneumatic hammers.
3. Pneumatic hammers equipped with drills or other drills that will cut 1.5 to 2.5 inches holes through the pavement. The drill should apply a downward pressure less than 200 psi to avoid spalling of the concrete adjacent to the injection hole at the bottom of the slab (18). However, the use of hydraulic drills that have high frequency with low impact force has been found by the Georgia DOT to be superior to pneumatic drills (18). Core drilling of injection holes was not found to be cost-effective (6).
4. A Flow Cone to determine the consistency of

the grout. The flow cone must meet the requirements of the Corps of Engineers Test Method CRD C611 or ASTM C939 (32). Figure 2.9 shows a cross section of the flow cone (29).

5. Vertical slab movement detection equipment: they must be capable of reading deflection measurements within one thousands of an inch accuracy.

6. Deflection device: the deflection device must be capable of applying at least 8000 pounds of force. It can also be a loaded vehicle that has a single axle which can be loaded to 18-kips evenly distributed between the two sides (18).

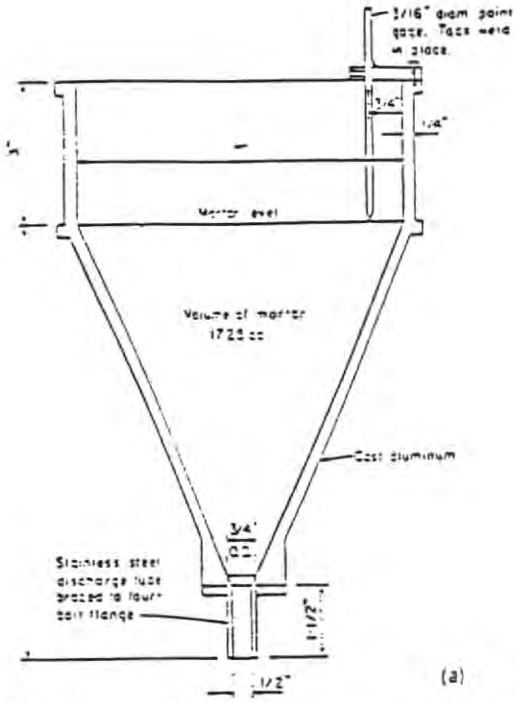
7. Water tank and pumps for the delivery of water to the plant.

8. Miscellaneous items include hoses, cylindrical wooden plugs, grout packers, steel drill, hole washing tools, and a stopwatch to measure the flow.

Equipment For Asphalt-Cement Grouts

In addition to the drills, the deflection devices, the uplift monitoring gauges, the air compressors, an asphalt undersealing operation requires the following equipment:

1. A pressure distributer tank truck: it must be able to heat the asphalt to the required



(a)

(b).

Figure 2.9. (a) Cross Section of the Flow Cone. (b) Grout Discharge and Timing Check (29).

temperature and to circulate the material during the heating process. The pumping equipment must have the capability to inject the asphalt at a pressure up to 80 psi (24).

2. Nozzles to deliver the asphalt and the air to the holes.

3. Proper safety clothing and face shields. These items are necessary to protect the operator from the hazards involved with the extremely hot asphalt.

Weather Limitations

Generally, slab stabilization should not be performed under these weather conditions:

1. The ambient temperature is below 40°F (18, 24).
2. The subgrade and the subbase are frozen.
3. There is an abnormal amount of moisture in the subgrade.

Atmospheric temperatures will generally control the amount of calcium chloride that can be added to cement grouts (18).

2.2.4 Determination of When and Which Slabs Should be Undersealed

Undersealing slabs that do not need it can cause unnecessary uplift of the pavement and thus produce a high point with a void on each side of the joint. Only slabs which have lost support due to the presence of voids should be undersealed. Loss of support can be detected : (1) visually, (2) by using deflection measurements, and (3) by using remote sensing.

Visual Indications of Loss of Support

Loss of support can be detected visually from (16):

(a) Ejected water and fine material presence on the surface of the pavement or shoulder.

(b) Staining by ejected water and fine material.

(c) Faulted slabs.

(d) Long wavelength deviations from grade (may indicate differential movement of the support layers).

Deflection Measurements

Void detection using deflection measurements should be conducted during the preliminary evaluation and after the completion of an undersealing operation. Deflection measurements should be taken on both sides of the joint to indicate if there is any load transfer. Figure 2.10 shows a corner deflection profile with excessive voids beneath the leave corner (31). Load transfer efficiency at joints and cracks can be calculated as follows(33):

Load Transfer Efficiency (%)= (DUS/DLS) * 100

where: DUS = deflection of the unloaded slab.

DLS = deflection of the loaded slab.

Figure 2.11 illustrates the definition of load transfer across the joints (24). Load transfer restoration is recommended for all transverse joints or cracks that exhibit poor load transfer (e.g., approximately 0 to 50 percent when measured in the early

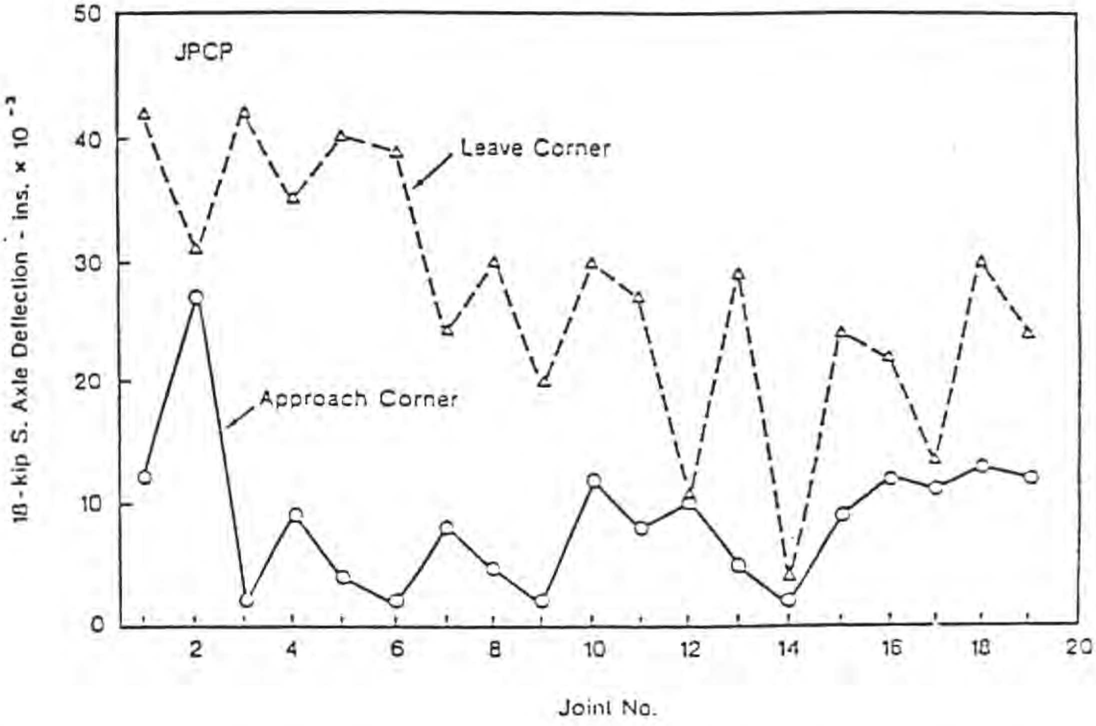


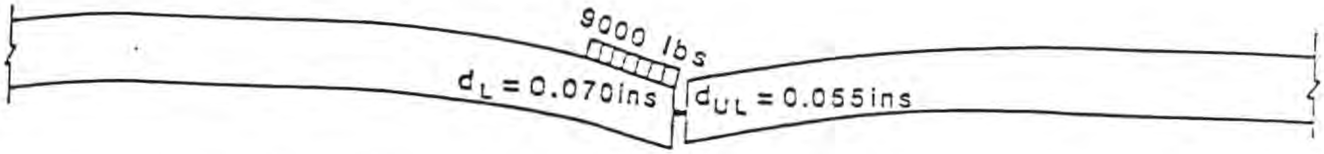
Figure 2.10. A Corner Deflection Profile With Excessive Voids Beneath the Leave Slab (31).

morning or in colder weather (31)).

Locations For Testing

Pumping typically occurs at transverse joints and working transverse cracks. Void detection should be carried out at these locations. Since there is a low probability of having voids in the middle of the slab, mid-slab deflection is to be taken only as an indication of what uniform support should be expected at the

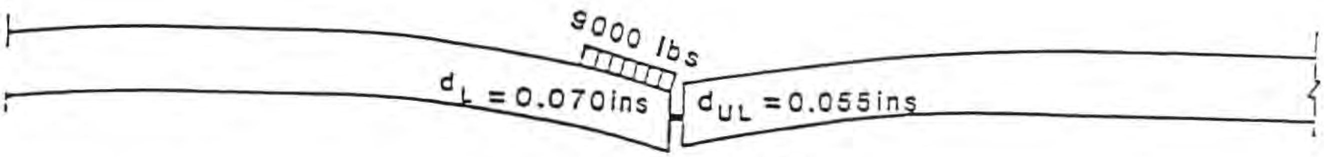
perfect joint. Figure 2.12 shows the locations where deflection measurements should be recorded (31).



DEFLECTION LOAD TRANSFER $\quad LT = \frac{d_{UL}}{d_L} \times 100$

$$LT = \frac{0.056}{0.070} \times 100 = 80\%$$

Figure 1a.



DEFLECTION LOAD TRANSFER $\quad LT = \frac{2\, d_{UL}}{d_L + d_{UL}} \times 100$

$$LT = \frac{2\ (0.055)}{0.070 + 0.055} \times 100 = 88\%$$

Figure 1b.

Figure 2.11. An Illustration of Load Transfer Across the Joints (24).

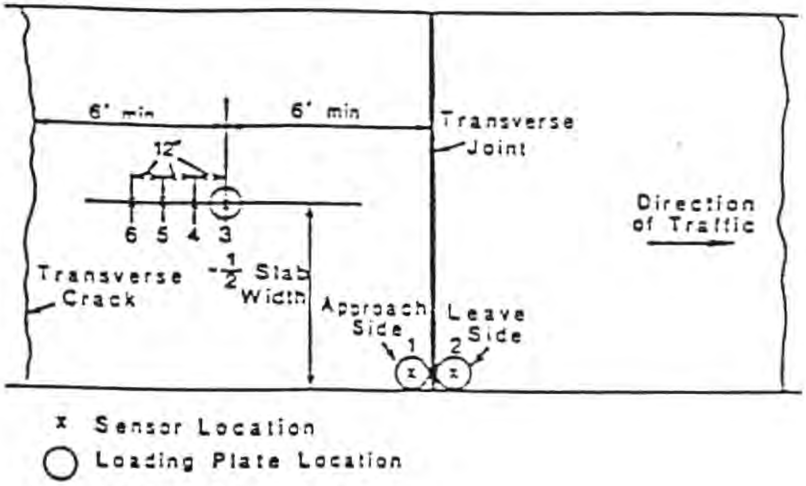


Figure 2.12. Locations of Deflection Measurements (31).

Time and Weather Limitations

Deflection measurements are found to be greatly affected not only by the time of the day but also by the time of the year. Curling of pavement slabs usually happens at night due to the differential change in temperature between the top and the bottom of the slab. This curling leaves the slab without any edge support, which has a great impact on reducing the corner deflection measurements. During the day, the top portion of the pavement can become much warmer than the bottom portion and therefore the edges will curl downward. Figure 2.13 shows the change in deflection in jointed pavement during the day (34). Usually, slab curling takes place between midnight and 10 a.m. However, deflections that should be considered are those that are recorded in the early morning at the time of maximum upward curling of the slab corners and minimum load transfer at the joints (18,27,24,33).

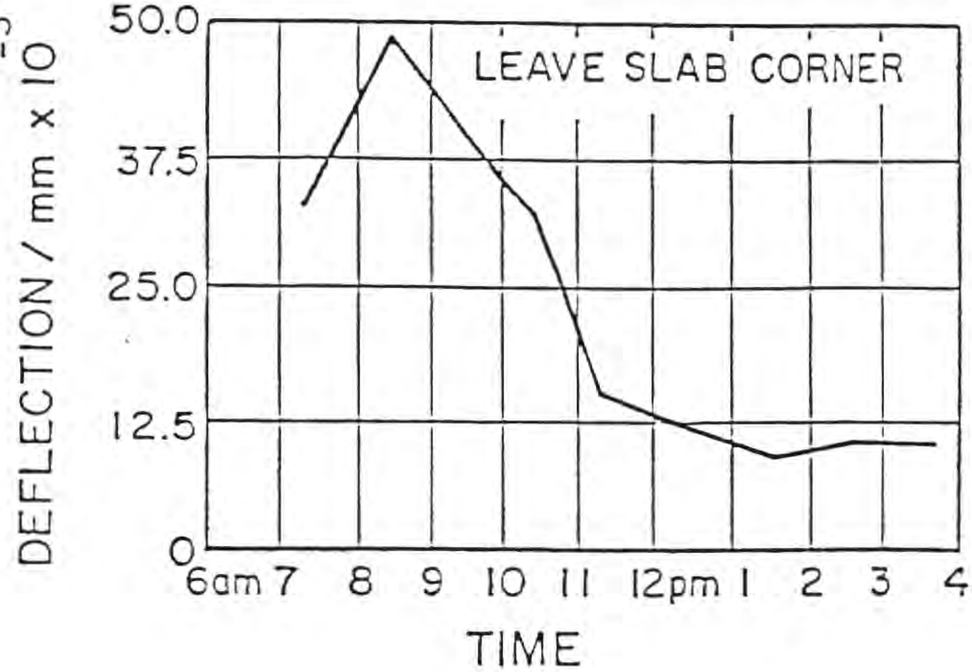


Figure 2.13. Change in Deflection in Jointed Pavement During the Day (34).

The presence of moisture at the bottom of a slab with a dry top causes warping of the pavement and thus loss of support will occur in the middle of the slab. The time of the year was also found to affect deflection measurements. During the cold winter, when the temperature drops below the freezing point, the moisture filling the voids may freeze and deflection measurements will show good support. Figure 2.14 shows the change in deflection during the year (6). Darter and Crovetti (33) recommend taking

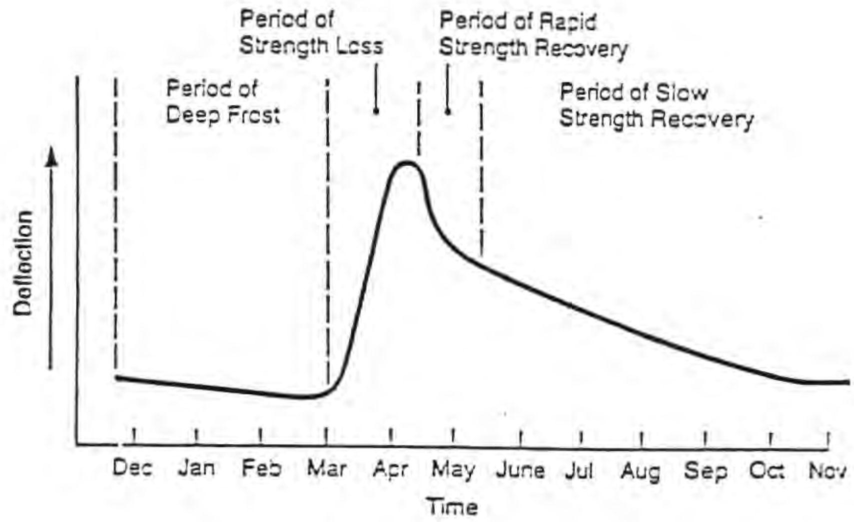


Figure 2.14. Change in Deflection During the Year (6).

deflection measurements when the temperature is between 50° and 80°F.

Deflection Measurement Devices

There are two common methods for measuring slab movement and locating voids beneath the slab. They include: (1) loaded vehicles and (2) deflection devices. The relative desirability of each of these devices must be determined on the basis of several criterions. Among these are (35): (a) safety, (b) initial and operating costs, (c) ease and speed of operation, (d) robustness or dependability of the equipment, and (e) usefulness of the information obtained.

1. Loaded vehicles: they are heavily loaded rubber-tired equipment that profile the pavement to reveal areas of excessive deflection or pumping. Profiling is done at a very low speed, about two

miles per hour (29). When the heavily loaded vehicle passes over joints or cracks, it causes the ejection of water and fines. Areas that deflect above specified minimum values are marked for subsequent undersealing. The Benkelman Beam setup is often used for this type of measurement. Typically, this setup consists of a rear axle wheel load of 18000 pounds. The deflection gauges mounted on the truck must measure the slab movement to 0.001 inch. Slab corners which deflect in excess of 0.025 inch should be undersealed (18).

2. Deflection devices: nondestructive testing devices such as the Falling Weight Deflectometer (FWD), the Dynaflect, the Road Rater, and the Thumper are used to measure the pavement response under a dynamic load. The nondestructive testing equipment utilized for void detection must have the ability to (31):
   - (a) apply a reasonably heavy range of loads,
   - (b) measure deflection directly beneath the center of the load,
   - (c) measure the deflection basin up to 36 inch (and preferably 72 inch) from the center of the plate at 12 inch intervals, and

(d) simultaneously measure slab deflection across joints and cracks for load transfer efficiency calculations.

A 1982 study (35) found that there is no significant difference between the Falling Weight Deflectometer and the Dynaflect. However, the Falling Weight Deflectometer is becoming more popular for use these days by highway agencies. The Falling Weight Deflectometer has a weight which is mounted on a vertical shaft. This weight is hydraulically lifted to a predetermined height and dropped onto a set of rubber springs. The force impulse is transferred from the spring system to a load plate which is resting on a thick rubber pad to help in the distribution of the load evenly over the loaded area. The resulting pavement deflection is measured by seismic deflection transducers. Figure 2.15 shows an illustration of the detection of loss of support beneath the slabs using the Falling Weight Deflectometer (31,33). Deflection readings from the FWD can be used for the back-calculation of the resilient modulus of different layers of the pavement.

Remote Sensing

Remote sensing techniques for void detection and void size measurements have provided excellent results (36). Remote sensing technology is becoming more sophisticated, reliable, and accurate. It is becoming more attractive to public officials for its nondestructive testing of pavement structures.

Devices that are classified under this title include the

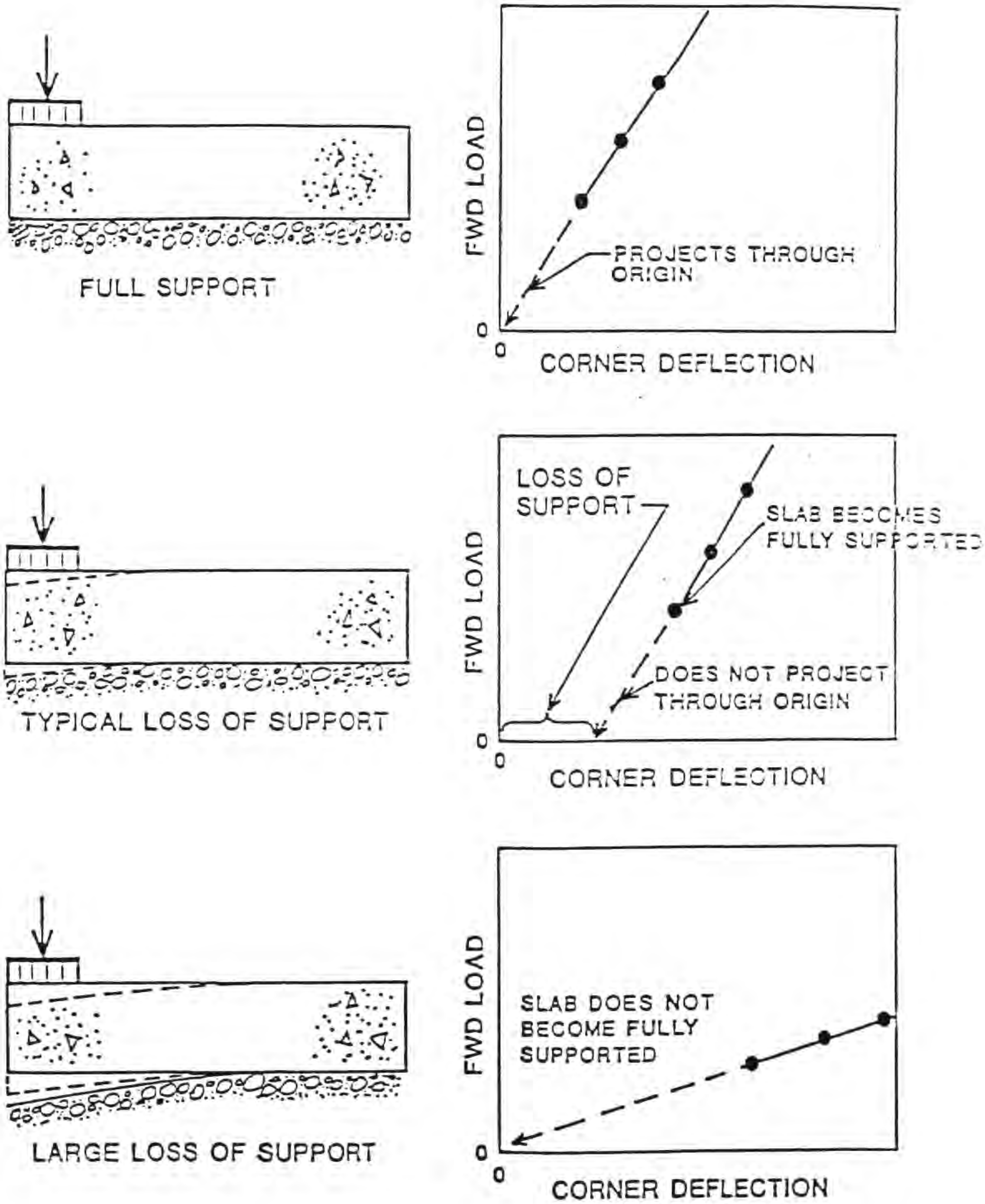


Figure 2.15. An Illustration of the Detection of Loss of Support Beneath Slabs Using the Falling Weight Deflectometer (31,33).

Ground Penetrating Radar (GPR) and the Pulsed Electromagnetic Waves technique.

1. Ground Penetrating Radar (GPR): it is a nondestructive remote sensing system that can be used to rapidly identify and evaluate various pavement structure conditions. This equipment can be used to measure pavement thickness, to identify thin or weakened areas, to locate voids beneath the pavement, and to identify pavement deterioration ("D" cracking) at joints and cracks. A major advantage for using the GPR over FWD is that the GPR is a rapid rolling operation. The GPR system consists of an antenna which is boom mounted at the front of a vehicle. It sends electromagnetic waves into the pavement layer. A portion of the wave energy that is reflected at each pavement layer interface is received by the transducers and processed within the control unit.
2. Pulsed Electromagnetic Waves: this technology has shown to be useful for locating and sizing voids beneath reinforced and non-reinforced concrete pavements. The characteristics of the radar signal response from pavements containing voids will allow signal processing algorithms to recognize small variations related to void detection and identification (34).

2.2.5 Determination of Void Sizes

It is essential to determine void sizes beneath the pavement slab in order to determine the type and the amount of undersealing material required to stabilize the pavement. However, experience has shown that the amount of grout depends upon several other factors, including (33):

(1) the amount of slab lift allowed during grouting,

(2) the base/subbase type and condition,

(3) the subgrade type and the extent of holes or discontinuities,

(4) the shoulder type, and

(5) the available channels for flow.

An average of 2 to 3 ft^3 of grout per joint should be planned for joints where voids exist to ensure adequate coverage of all cavities (33).

The presence of a void does not necessarily imply a large hole beneath the pavement (37). For example, a gap of 0.005 inch between the slab and the subbase will allow sufficient deflection in the pavement to significantly increase the stress. As shown in Figure 2.16, stresses will increase with increasing void size (37). The increase in stresses is relatively lower for a thicker pavement than for a thinner one. As a result of increasing stresses the fatigue life of the pavement is significantly reduced as shown in Figure 2.17 (37).

Void sizes can be determined using nondestructive tests and

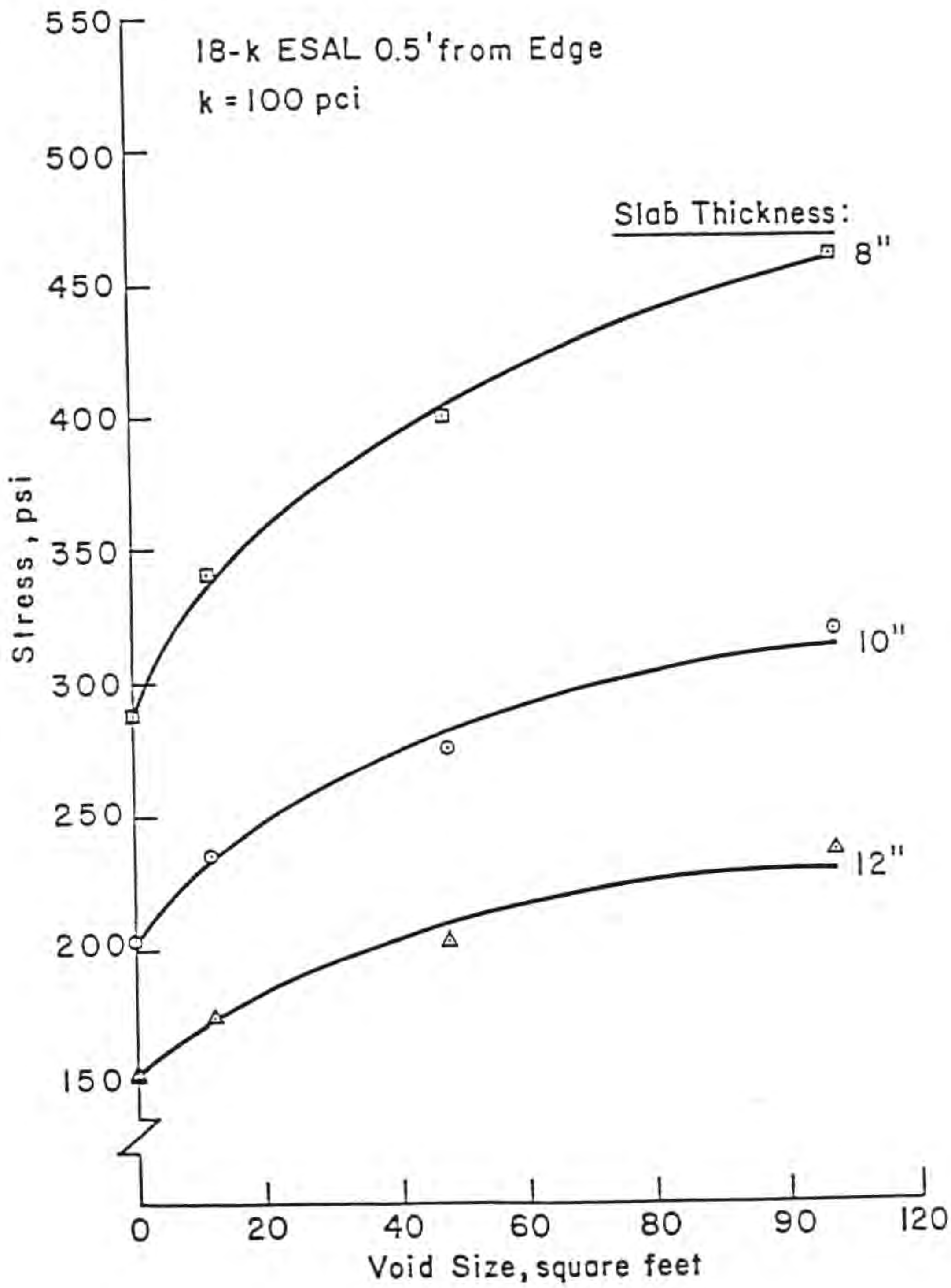


Figure 2.16. The Relationship Between the Void Sizes and Stresses in Concrete Pavements (37).

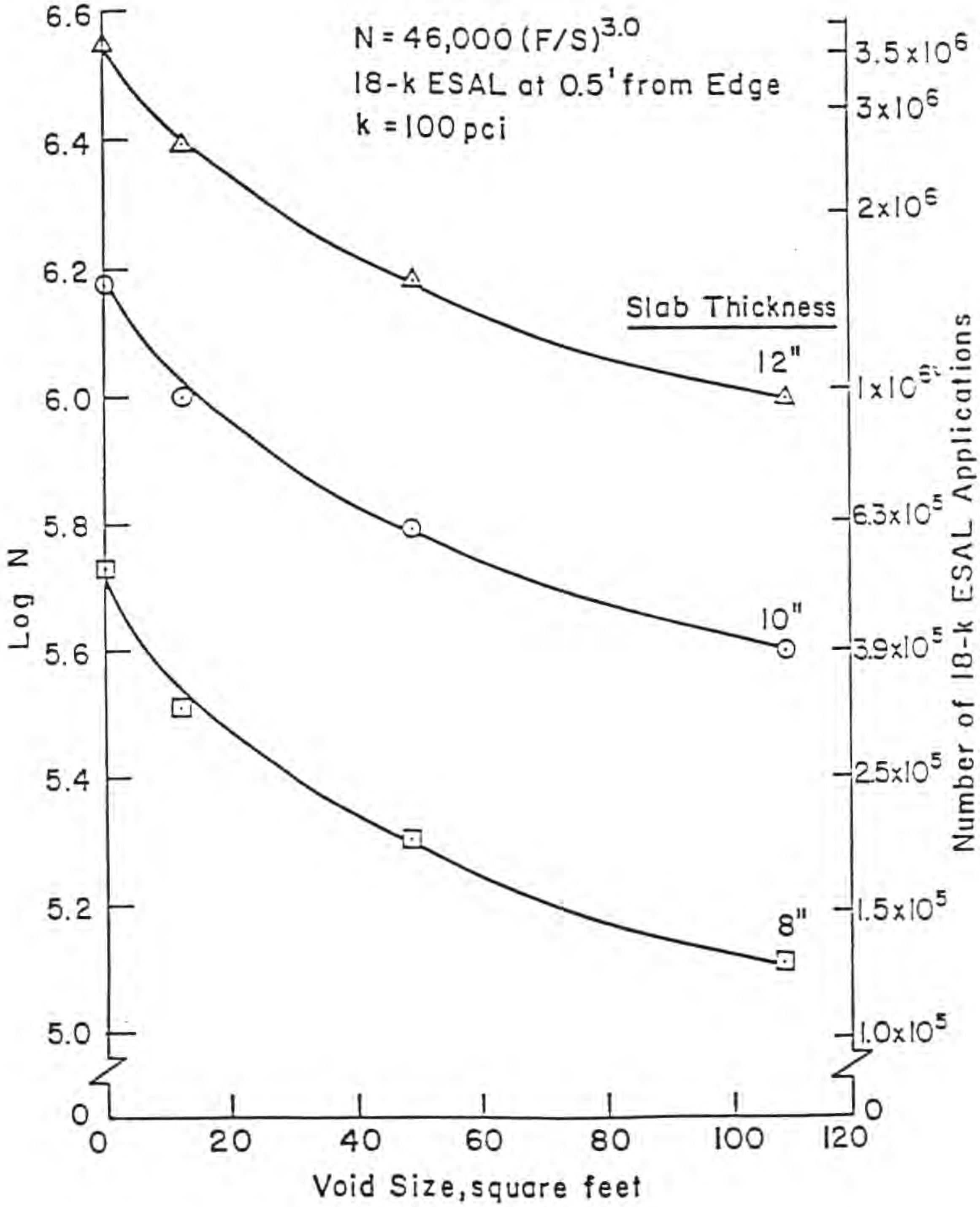


Figure 2.17. The Relationship Between the Fatigue Life of the Pavement and Void Sizes (37).

computer modelling. In computer applications, models were developed to predict the amount of pumped material which can be correlated with the area and the size of voids. However, those models should be used with caution until they are validated with reliable field results.

2.2.6 Determination of the Effectiveness Undersealing.

The primary indication of the effectiveness of an undersealing operation is the performance of the rehabilitated pavement. This can be determined by measuring the movement of the slab corners using the loaded vehicle or deflection devices. High deflection measurements indicate ineffective undersealing work. For example, if the corner deflection under an 18-kip single axle load is in excess of 0.01 inch (SDDOT specification), the slab should be regrouted and retested. No additional grouting should be made after two proper attempts.

Figure 2.18 shows the effect of grouting on the deflection measurements under the leave slab (33). An Illinois study (14) found that if the initial deflections were below the calculated average for a given pavement, undersealing would be ineffective. Furthermore, deflection of the pavement could not be reduced beyond a certain limit. Generally, undersealing when done properly can restore support to the pavement not only in the short term but also in the long term (14).

2.2.7. Assessment of Life Expectancy and Unit Costs Associated With Undersealing

The unit costs associated with an undersealing operation

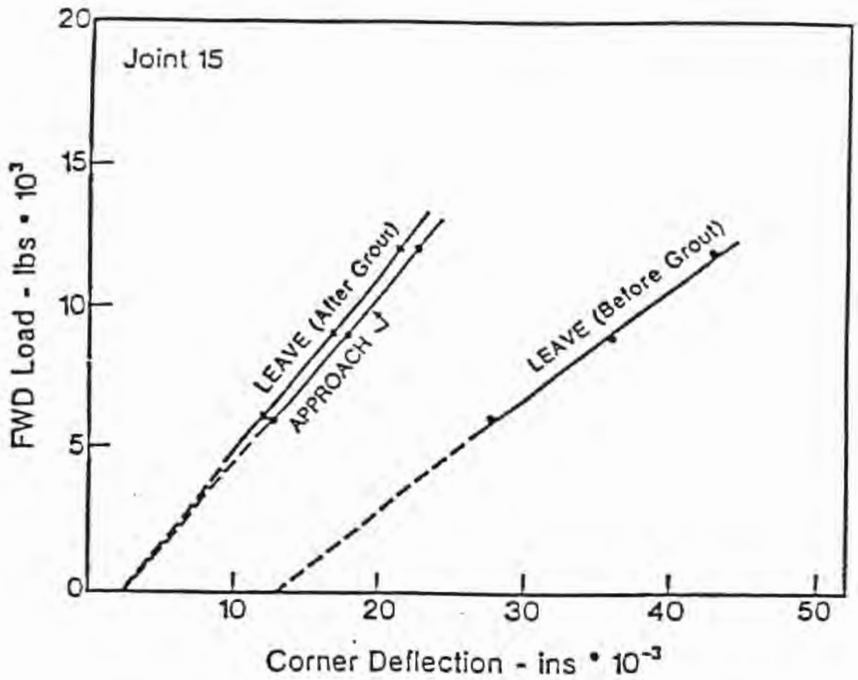


Figure 2.18. FWD Load Versus Corner Deflection (33).

include such items as:

1. Deflection testing for voids detection.
2. Number of holes drilled (this pay item includes drilling, plugging, and hole resealing).
3. Undersealing material cost.
4. Traffic control and mobilization costs.
5. Labor costs.

All these costs will be dictated by the size of the project. It is important to realize that undersealing alone will not be sufficient to prevent future pumping and faulting in concrete pavements. Other pavement restoration works such as resealing joints, sealing cracks, restoring load transfer, concrete patching, installation of edge drains, grinding faults, and placement of concrete or asphalt overlays should be considered in the final analysis. Cost and maintenance alternatives should be developed, and the most effective design to prevent pumping and faulting problems should be adopted.

Most of the literature does not address the long term effectiveness of undersealing. However, several regression models were developed to predict future distresses in PCC pavements. However, these models have great limitations since they are based on limited data collected from specific pavement sections in specific geographical areas of the country. Long term effectiveness of the rehabilitated pavements can be better determined from the long term monitoring of the concrete panels to observe that pumping and faulting will not resume again. This will require periodic deflection measurements and field surveys.

In summary, the design of an undersealing project includes the selection of an acceptable undersealing material, testing of the pavement for voids, estimation of underseal quantities, determination of the optimal time in a pavement's life cycle to perform undersealing work, and adequate construction practices. Furthermore, the success of a pavement undersealing project depends also on other rehabilitation work that should be performed in conjunction with undersealing.

CHAPTER III

SURVEY RESULTS

3.1 General

Pavement undersealing is a major rehabilitation work conducted by some highway agencies to alleviate problems associated with pumping and faulting in concrete pavements. A survey of current highway undersealing practices was sent out to all fifty states and some selected contractors who perform undersealing work. A sample of the survey form is shown in Appendix A. A rigorous review of design and performance experiences, construction methods, and inspection guidelines of undersealing was conducted. The relevant questions on undersealing were:

1. Which agencies implement slab undersealing?
2. What criterion is used to establish those slabs that need undersealing?
3. What are typical materials used in undersealing?
4. What are typical construction methods?
5. What are other pavement restoration works done in conjunction with undersealing?
6. How do you determine the long term effectiveness of undersealing?

3.2 Survey Results

To collect the needed information, two survey techniques were used: the initial information request, and follow-up requests. Thirty three states and two contractors (The Concrete Doctor and May Pressure Grouting) responded to the survey. Figure 3.1 shows the geographical locations of the states who responded to the survey.

Implementation of Undersealing

As shown in Figure 3.1, of the 33 states only 16 are implementing undersealing as a rehabilitation strategy. The other 17 states do not implement undersealing due to the following reasons:

1. The difficulty of locating voids beneath the slab.
2. The difficulty of filling all voids.
3. Overgrouting could cause more damage to the pavement.
4. Undersealing depends on the experience and skills of the contractors.
5. Undersealing is not a cost-effective operation.

However, even some of those states that underseal their PCC pavements are using it on a limited basis due to the reasons mentioned above. Only the State of Indiana is clearly satisfied with its undersealing operation. However, the Indiana DOT is still seeking more information on the means to locate voids beneath the pavement slab.

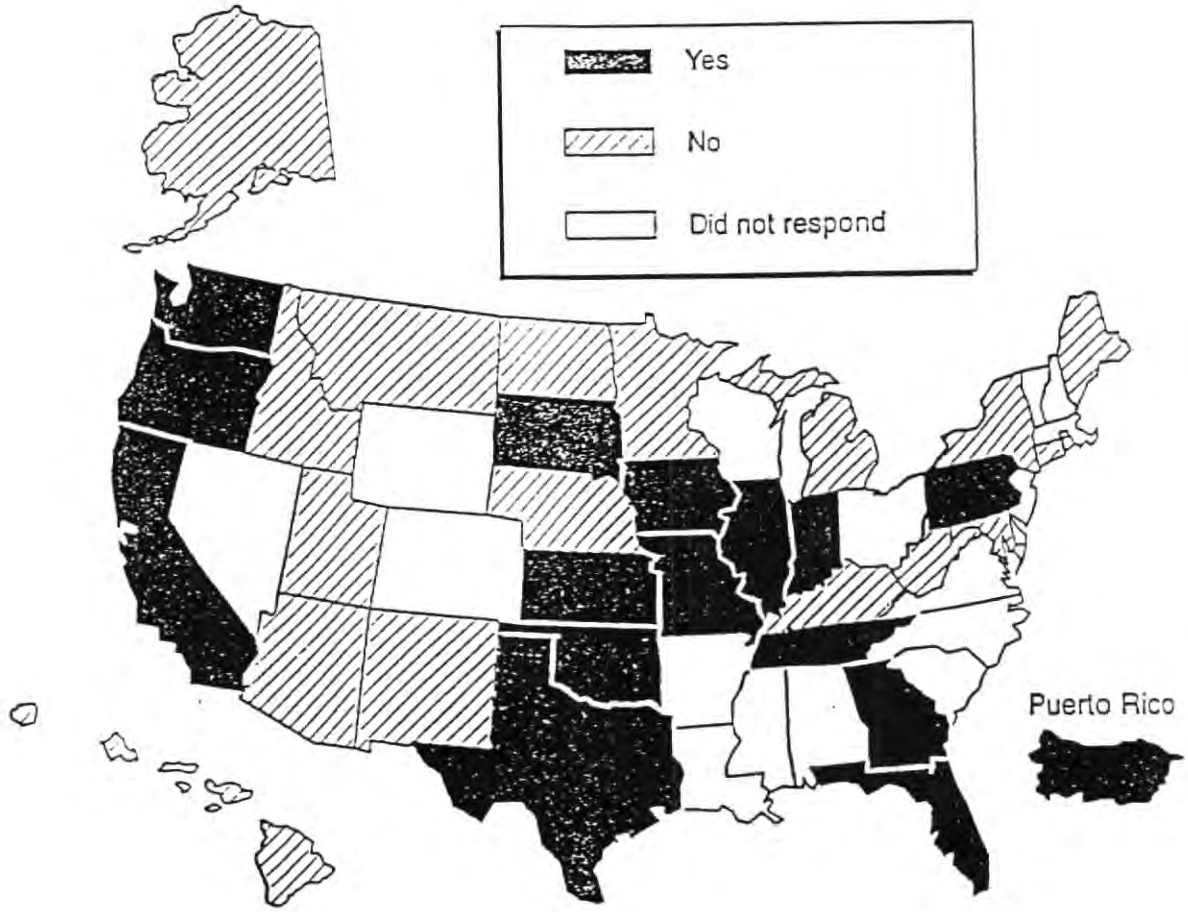


Figure 3.1. Various States Who Responded to the Survey.

Void Detection

Of the 16 states which are using undersealing in pavement restoration work, nine are using visual surveys as indicated by pumping and/or faulting in combination with nondestructive testing to locate voids beneath the slab. The other seven states are relying on non-destructive testing for void detection. The Falling Weight Deflectometer (FWD) and the Benkelman Beam are the most commonly used equipment. States such as Texas, Pennsylvania, Iowa, and Washington have more than one type of device in service. Table 3.1 presents the type of equipment used in determining the locations of slabs that need undersealing work.

Table 3.2 presents the criteria used for determining the slabs that need undersealing. These values are also used for determining the effectiveness of an undersealing operation. Generally, corner deflections greater than .01 inch to greater than .035 inch are adopted. Some states such as Pennsylvania and Missouri use a load transfer efficiency (LTE) value less than 65 percent as a criterion for determining those slabs that need undersealing.

The Concrete Doctor uses visual surveys and the Benkelman Beam while May Pressure Grouting uses the Falling Weight Deflectometer along with visual inspection to detect voids beneath PCC pavements.

Table 3.1. **Techniques Used in Determining the Locations of Slabs That Need Undersealing.**

STATE	Visual	Benkelman	FWD[a]	GPR[b]	Dynaflect	Road Rater
California	X					
Florida	X	X				
Georgia	X	X				
Kansas	X		X			
Illinois	X		X			
Indiana					X	
Iowa	X			X		X
Missouri	X		X			
Oklahoma[c]						
Oregon			X			
Pennsylvania		X	X			
Puerto Rico		X				
South Dakota	X	X	X			
Texas[d]	X		X	X		
Tennessee[e]						
Washington		X	X			

a: Falling Weight Deflectometer.
b: Ground Penetrating Radar.
c: Not specified.
d: The Spectral Analysis of Surface Waves (SASW) technique is also used.
e: A pre-rolling method is used.

Table 3.2. Corner Deflection and Joint Efficiency Values Used in Determining the Locations of Slabs that Need Undersealing.

STATE	> 0.01"	> 0.015"	> 0.02"	> 0.025"	> 0.03"	> 0.035"	JE[a] 65
California[b]							
Florida		X					
Georgia					X		
Kansas[c]							
Illinois[b]							
Indiana[d]							
Iowa[b]							
Missouri[e]							X
Oklahoma[b]							
Oregon[f]				X			
Pennsylvania			X				X
Puerto Rico					X		
South Dakota	X						
Texas			X				
Tennessee					X		
Washington						X	

a: Joint efficiency less than. b: Not specified. c: If corner deflections are greater than a base value established from the interior slab deflection. d: Judgement + Modified Majidzadeh Criteria. e: Corner deflection greater than 0.0175" is also used. f: Darter/Crovetti slope intercept method is also used (a deflection of 10 mils).

Grouting Material

Figure 3.2 shows the types of grouts used in the various states. As depicted in the figure, cement-fly ash grout is the most popular material used in the states. The State of Indiana is the only state that uses asphalt cement with satisfactory results. For cement-fly ash grouts, a seven-day compressive strength of 600 to 800 psi and an average flowability of 10 to 16 seconds are generally recommended and used.

The Concrete Doctor recommends using limestone dust-cement and pozzolan-cement grouts that have a seven-day compressive strength of 600 psi and a flowability of 12 to 18 seconds. May Pressure Grouting recommends using pozzolanic grouts with a seven-day compressive strength of 600 psi and a flowability of 15 seconds.

Pumping Procedures

All sixteen states limit undersealing operations to conditions when the temperature is above 32°F. In cold weather states, undersealing is also not allowed if the subgrade is frozen.

The maximum allowable pumping pressure (excluding the initial pumping pressure) used by highway agencies varies from 20 to 150 psi. Pumping pressures ranging from 50 to 60 psi seemed to be the most preferred values. Table 3.3 presents the maximum allowable pumping pressures specified by state highway agencies.

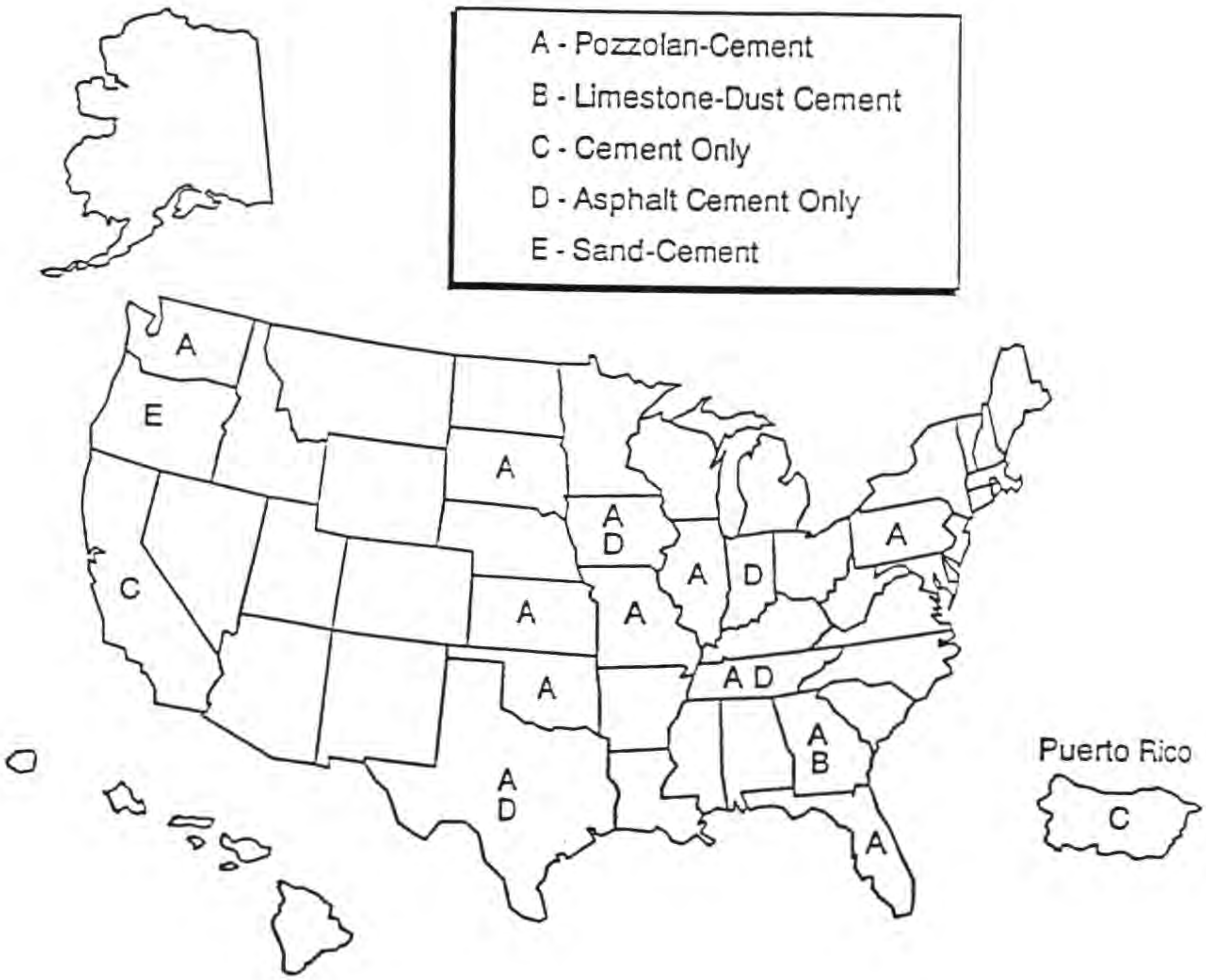


Figure 3.2. Typical Undersealing Materials Used by the Different States.

Table 3.3. **Maximum Allowable Pumping Pressures Specified by the Various Highway Agencies.**

STATE	20[a]	40	50	60	100
California	X				
Florida[b]					
Georgia					X
Kansas				X	
Illinois		X			
Indiana[c]					
Iowa	X				
Missouri			X		
Oklahoma				X	
Oregon[c]					
Pennsylvania[c]					
Puerto Rico[c]					
South Dakota				X	
Texas			X		
Tennessee			X		
Washington[c]					

a: All values are in psi. b: As low as possible.
c: Not specified.

Maximum vertical slab movement allowed during pumping varies from 0.0175 to 0.25 inch. Table 3.4. shows the maximum allowable vertical slab movements specified by the states.

The survey also revealed that pumping should be stopped when one of the following conditions prevails:

1. The maximum allowable pumping pressure has been attained.
2. The maximum allowable vertical slab movement has been measured.
3. Some states recommend setting time limits for pumping grout into the holes.
4. Some states recommend stopping injection when the grout starts to flow from adjacent holes and/or comes out of the joints.

The Concrete Doctor allows a vertical slab movement of up to 0.125 inch and a pumping pressure of 50 psi. Also, May Pressure Grouting allows a vertical slab movement of up to 0.125 inch and a pumping pressure of 60 psi.

Pavement Restoration Work Done in Conjunction with Undersealing

All states adopt some type of pavement restoration work in conjunction with undersealing. This may include:

1. crack sealing,
2. joint resealing,
3. patching,
4. edge drains,
5. grinding,

Table 3.4. Maximum Allowable Vertical Slab Movements Specified by the Various Highway Agencies During Pumping.

STATE	0.035[a]	0.05	0.10	0.125	0.20	0.25
California[b]						
Florida				X		
Georgia		X				
Kansas				X		
Illinois		X				
Indiana						X
Iowa			X			
Missouri					X	
Oklahoma	X					
Oregon[c]						
Pennsylvania		X				
Puerto Rico		X				
South Dakota				X		
Texas				X		
Tennessee			X			
Washington[d]						

a: All values are in inches. b: Zero to profile grade of surface.
c: A deflection of 0.025 inch is specified.
d: a deflection of 0.020 inch is specified.

6. overlays,
7. spall repair,
8. full-depth repair, and
9. dowel retrofit.

Long Term Effectiveness of Undersealing Operations

All sixteen state highway agencies use the corner deflection measurements and joint efficiency values presented in Table 3.2 to establish the acceptance of undersealing work. If a joint fails, the contractor may be required to rework the slab. No guidelines were provided to establish the long term effectiveness of undersealing. All states rely on long term monitoring of the concrete panels to observe that pumping and faulting are not resumed.

CHAPTER IV

RESULTS OF THE MEETING WITH THE SOUTH DAKOTA DOT FIELD ENGINEERS

4.1. General

A meeting was held in Brookings on December 7, 1992 with South Dakota Department of Transportation (SDDOT) field engineers who have worked on undersealing projects. The purpose of the meeting was to facilitate an exchange of information on problems encountered during undersealing operations and to discuss ideas on improving the overall implementation of successful undersealing work in South Dakota. Fourteen SDDOT and four SDSU personnel attended the meeting. No undersealing contractors were present at the meeting. A list of the people attending the meeting is shown in Appendix B.

The items discussed during the meeting included:

1. When and how the SDDOT decides to implement undersealing?
2. What are typical materials and construction methods used in an undersealing operation?
3. How do you determine the effectiveness of an undersealing operation and what are the advantages and disadvantages?

The following sections will discuss each of those individual questions as addressed by SDDOT personnel.

4.2. When and How the SDDOT Decides to Implement Undersealing?

The South Dakota Department of Transportation maintains a five year construction program for grading and reconstruction type work. Resurfacing type work which includes pavement restoration is programmed two years in advance.

The SDDOT has a specific process for planning for, identifying, and prioritizing needs for both reconstruction and resurfacing in the state maintenance system. Pavement rehabilitation is not well addressed as far as prioritizing needs because the parameters are not as measurable. However, the SDDOT has a considerable backlog of fairly obvious needs in pavement restoration and maintenance. By working with the planning, field, and research personnel, the SDDOT is to prioritize the greatest needs and what should come first in pavement restoration and rehabilitation.

When performing rehabilitation on a PCC pavement project, a full scope of activities is to be done. These activities may include: full-depth repair, partial-depth repair, undersealing, grinding, resealing joints, sealing cracks, and any necessary restoration on the shoulders. In some cases, edge drains may also be installed. Any combination of the above activities may be done during a rehabilitation project.

The pavement in the worst condition has a priority to have restoration work. At the point where restoration work is not cost-effective, a crack and seat procedure is generally adopted.

The SDDOT usually decides to implement undersealing on a PCC pavement rehabilitation project when faulting and rideability problems become apparent. This usually correlates with the presence of voids beneath the pavement. Deflection testing is then performed to identify those slabs that have voids.

Deflection of the slab is measured at the corners using an 18-kip single axle load. If the slab deflects more than 0.01 inch, then the joint will be undersealed. Deflection testing is done early in the morning to acquire data before the slab curls. Heat expansion may cause the joints to lock together.

The problems facing the SDDOT are:

1. It is still not clear when or at what point to choose between pavement restoration and crack and seat.
2. The lack of identifying the pavement needs at an early stage. Usually, the corrective action is taken after the problem is visually identified. By the time rehabilitation work is done, the pavement could have reached a worse condition where the job can not be done properly.
3. The parameters used in South Dakota for programming reconstruction and resurfacing fit very well to asphalt concrete pavements, but not as well to PCC pavements.
4. The situation changes so much between projects in different areas, due to different causes of the

problem. This makes it difficult to know what to do and when.

5. When pavement restoration is programmed, undersealing may not be one of the alternatives considered.

4.3 What are Typical Materials and Construction Methods Used in an Undersealing Operation?

In South Dakota, cement-fly ash grouts that have a seven-day compressive strength of 600 psi are being used. The fly ash and cement are usually mixed at a ratio of 3:1 by volume, respectively. No additives are currently being added to the grouts. Lime-cement and sand-cement grouts have been used in the past with unsatisfactory results. The use of these grouts has been discontinued. The quantity of needed material varies and depends on the size of the voids and the experience of the contractor.

Slabs that show corner deflections greater than 0.01 inch by means of the Benkelman Beam must be undersealed. The Ground Penetrating Radar (GPR) has been tried in South Dakota but it did not produce satisfactory results. Coring has also been tried but it could not indicate the extent of the voids.

Generally, the hole pattern used in undersealing is left to the contractor. Typically, one hole at each corner of the slab and one hole at mid-slab are used. An initial pumping pressure of 150 psi is allowed for a few seconds to initiate the grouting process. After that, the maximum allowable pumping pressure is

60 psi. The maximum allowable vertical slab movement is at the discretion of the contractor, but should not be more than 0.125 inch.

On an undersealing project on I-29 South of Watertown, two different contractors worked on the project. Mays Construction worked on the north bound lane and used an allowable maximum slab lift of 0.125 inch. The Concrete Doctor worked on the south bound lane and stopped pumping as soon as the slab started to rise. When the joints were tested three months later, the Concrete Doctor, who allowed no slab movement, failed more often than Mays Construction , who allowed a vertical slab movement of 0.125 inch.

Generally, pumping should be stopped when one of the following conditions is reached:

1. The grout starts to flow between the holes.
2. The maximum allowable pumping pressure is attained.
3. The maximum allowable vertical slab movement has been obtained.

The undersealing job is accepted if the post-testing deflection measurements show readings of less than 0.01 inch. If a joint fails, the contractor may be required to rework the slab. Normally, no more than two proper attempts at undersealing a particular slab should be done.

Other restoration work performed in conjunction with undersealing may include full-depth repair, partial-depth repair, pavement grinding, joint resealing, crack sealing, and the installation of edge drains.

4.4 How Do You Determine the Effectiveness of an Undersealing Operation, and What are the Advantages and Disadvantages?

The effectiveness of undersealing is determined by the long term monitoring of the concrete panels to observe that pumping and faulting are not resumed. Some deflection measurements have also been performed on undersealed sites.

It is expected that pavement life can be prolonged ten to fifteen years due to undersealing. Undersealing improves the structural support but does not restore rideability. Any other rehabilitation works that do not consider restoring the support to the faulted pavement is a waste of money. Also, relying on grinding alone could have a negative impact on the structural integrity of the pavement.

On the average, it costs about $100,000 per two-lane mile for pavement restoration and rehabilitation. This is considerably less than the cost of performing a crack and seat procedure, which costs about $170,000 per two-lane mile. In the long term and for a pavement in very poor condition, a crack and seat technique could be more cost-effective than performing undersealing in conjunction with other rehabilitation work.

CHAPTER V

FIELD EVALUATION

5.1 Scope of Work

An evaluation of selected undersealed and non-undersealed plain jointed concrete pavement sections was conducted during April, May, and September of 1993. Both deflection measurements and visual surveys of the pavement sections were performed. The Dynatest 8000 Falling Weight Deflectometer (FWD) was used to measure deflections. As of 1987, nine projects were undersealed in South Dakota (see Table 5.1). After consulting with Blair Lunde and Ken Marks, the following four projects were selected for further evaluation:

1. F0012 (57) 274, PCEMS 2667. US12 from West of Mina at MRM 275 East 12.6 miles to Aberdeen in Edmunds and Brown counties.
2. F0012 (43) 291 PCEMS B430 & F0281 (27) 184, PCEMS A430. US 281 from US12 South 3.2 miles in Brown county.
3. F0012 (76) 262, PCEMS 3527. US12 from Ipswich East 11.4 miles in Edmunds county.
4. NH 0050 (59) 386, PCEMS 3866. SD50 from East of Yankton at MRM 386.64 East 7.248 miles in Yankton county.

The basis for selecting these projects was the availability of FWD data prior to undersealing, the presence of non-undersealed sections in the highway, as well as the age of the

Table 5.1. A List of Undersealed Projects in South Dakota.

Project	County	Construction Year	Rehab Yr.	No. of Lanes	Surface	Base	ADT (Cars/Truck)	Condition at Time of Construction	Comments
F0012(43) F0281(27)	Brown	1971	1987	4U	7" PCC	4" C.T.	3673/441	Good with Severe Faulting	Part of US 12 was not rehabilitated
F0012(57)	Edmunds & Brown	NA	1989	2,4D	7" PCC	4" L.T.	2100/298 to 9890/731	Marginal for Rehabilitation	Two miles not undersealed, considerable failure
F0212(62)	Codington	1971	1989	4U,4D	8" PCC	3" L.T.	8600/1728 to 4210/1006	Very Marginal for rehabilitation	-
IR29-6(19)	Codington, Hamlin & Deuel	1975/1976	1990	4D	9.5" PCC	3.5" L.T.	5590/1174 to 5060/850	Good with significant faulting & failed joint seals	-
F0037(53)	Beadle & Sanborn	1974	1991	4D	7" PCC	4" L.T.	1800/286 to -/216	Fair with extreme faulting, spalls, failed joint seals	-
F0014(96)	Kingsbury & Brookings	1976 1973	1991 1991	2 4D	8" PCC 7" PCC	4" L.T. 2" ACB	1357/185 2480/268 to 2556/269	Good with extreme faulting & failed joint seals Not good, no underseal	- Grindings, reseal joints
F0012(76) F0045(26)	Edmunds	1974 1977	1992 1992	2 NA(2)	7" PCC 7" PCC	4" L.T. 4" L.T.	2061/293 1746/248 and 3377/145	Good with faulting and failed joint seals Extensive cracked slabs due to utilities and subgrade	- Required full depth repair
F0050(53)	Yankton & Clay	NA	1991	NA	8" PC 7" PCC	3" L.T. 3" L.T.	1085/98 (1989 data)	Good (SHRP test sections) SPS-4	Small amount of slab replacement and spall repair
F0012(61)	Brown & Day	NA	NA	NA	NA	NA	NA	NA (SHRP test sections, SPS-6 undersealing)	Crack, break, seat and overlay

Notes: U = Undivided ; D = Divided ; C.T. = Cement Treated ; L.T. = Lime Treated ; ACB = Asphalt Concrete Base ; NA = Not Available pavement after undersealing.

5.1.1 Time and Condition of Testing

Temperature and moisture gradients in concrete have major influences on deflection measurements. Generally, deflection measurements should be normalized to 77°F so that consistent data comparisons could be done. Raw deflection data was analyzed at their corresponding temperatures. It was important in this research project to show the influence of temperature on joint closure and joint efficiency.

In locating voids beneath concrete slabs, it is important to conduct FWD tests when the slabs are relatively flat. In other words, deflection testing should be performed when the slabs are not experiencing curling and/or joint closure. This condition can be achieved when the ambient temperature is between 50-80°F (33). When a pavement curls up at the corners and edges, this will yield higher deflection measurements and thus will indicate the presence of more voids. When the joints are in a locked-up condition, this will generally give a higher level of load transfer efficiency. Thus, to evaluate load transfer efficiency, the pavement joints and cracks should be tested during the cool of day (between the hours of 7 and 11 a.m in the summer time) or at night. For void detection, the pavement should be tested when the temperature warms up and the slabs are flattened out.

5.1.2 Deflection Measurements Protocol

Three or four 500 ft sections were selected for each project. The slabs were tested at four locations: in the middle of the slab near the transverse joint, at the center of the slab, 2 to 3 ft from the shoulder close to the transverse joint, and 2 to 3 ft from the shoulder in the middle of the slab. The sections tested were east bound (EB), west bound (WB), north bound (NB), or south bound (SB). Figure 5.1 shows a slab layout of the test locations. The "X" in this figure represent the test panel number. Figures 5.2 and 5.3 show representative locations of the deflection measurements taken at the joint (location X.22) and at the mid-slab (location X.10). When joint deflection measurements were performed (i.e. X.31 and X.22), one sensor was placed close to the edge of the leave slab while the other sensor was placed close to the edge of the approach slab. The distance between the two sensors was about 4 inches (see Figure 5.4). At each test location, two levels of loads were applied. One load was close to 8000 lb while the other load was close to 12000 lb. This was true for all of the projects with the exception of SD50 in Yankton where two FWD loads of 9000 lb and 16000 were applied after undersealing the project.

5.1.3 Data Analysis Methodology

In our analysis, the deflection data at the higher load level were selected for evaluation (SD50 data were analyzed at the lower load level). The most basic method of void detection is the use of profile plots of deflections. Approach and leave slab deflections versus slab or joint number graphs can be drawn. Where there is a sudden increase in the leave slab deflections, this will indicate the presence of voids beneath the slab.

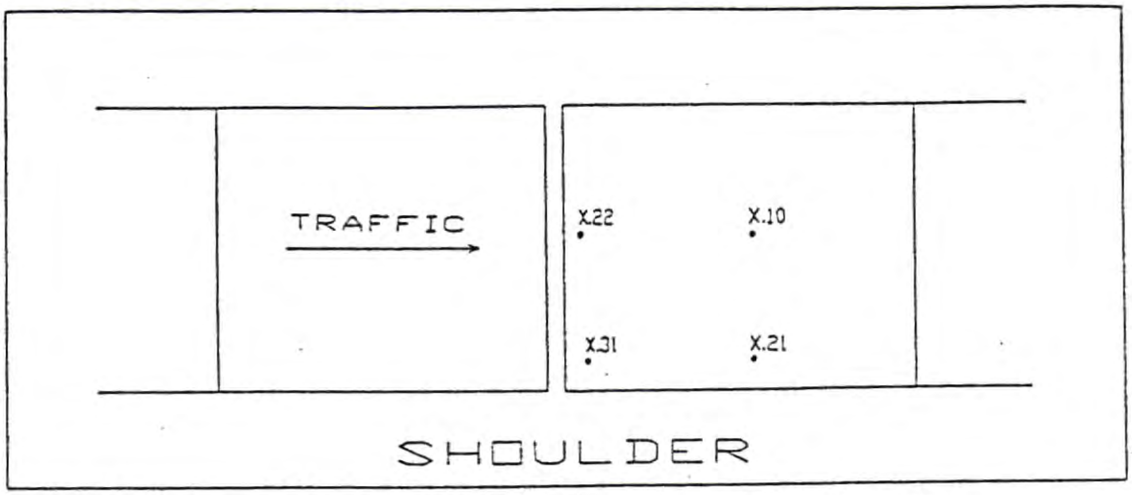


Figure 5.1. Slab Layout Showing Deflection Testing Locations.

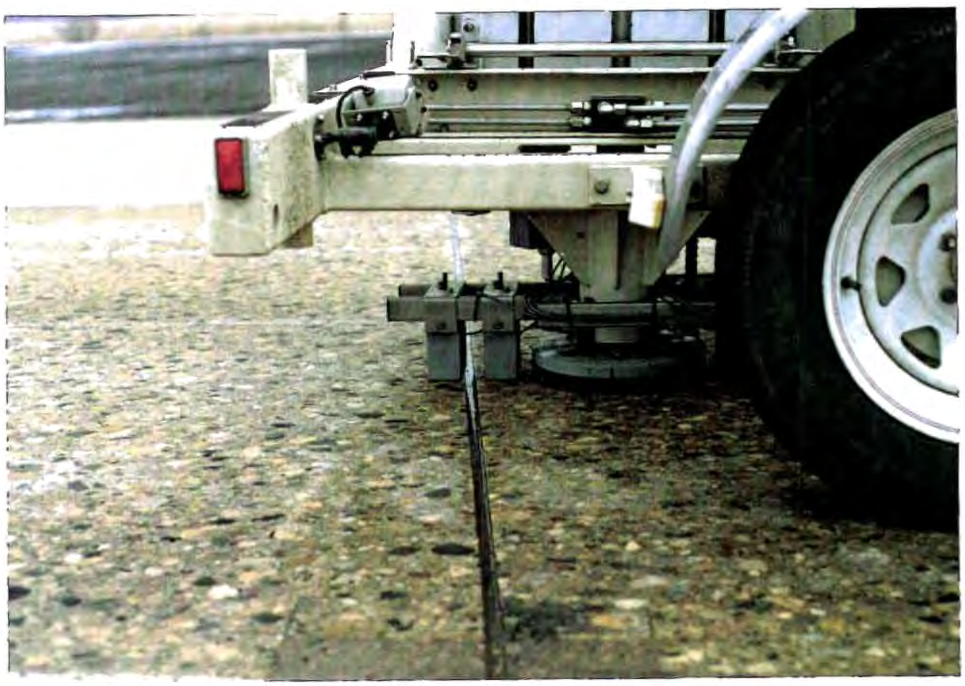


Figure 5.2. Deflection Measurements Taken at Transverse Joints.

Figure 5.3. Mid-Slab Deflection Measurements.

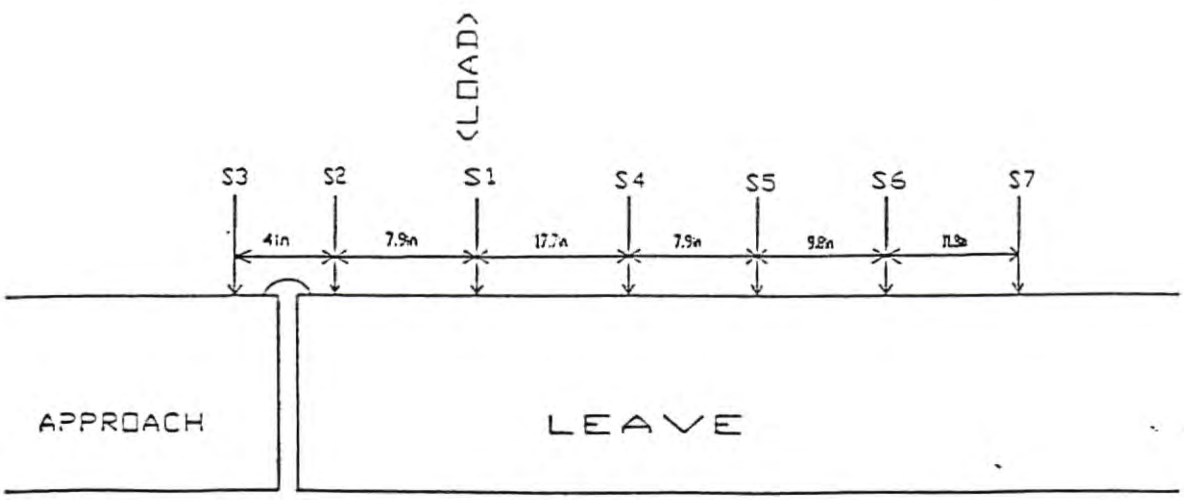


Figure 5.4. Falling Weight Deflectometer Sensors Arrangement.

Also, profile plots of load transfer efficiency (LTE) are good indications of those panels where voids and faulting may be present. Load transfer efficiency can be computed using the following formula:

LTE (%) = (Deflection of Loaded Slab/Deflection of Unloaded Slab)x98.

The FWD manufacturer recommends multiplying by 98 instead of 100 because the gap distance between the two sensors on each side of the transverse joint is considered low (about 4 inches). In our field evaluation, the load was always applied on the leave slab.

A comparison between the load transfer efficiency data obtained at the leave corner (location X.31) and at the mid-slab edge (location X.22) was conducted. The results indicate that the LTE values computed at the leave corners are either marginally smaller or the same as those LTE values obtained at the mid-slab edge. Since most voids due to pumping would probably occur under the corner of the leave joint, only leave corner deflection data will be addressed in our analysis.

A third graphic method of void detection was generated by plotting load versus deflection data. Those slabs that have deflection values of more than the mean deflection plus one standard deviation for the whole subsection were used in drawing the graphs. If the pavement is fully supported and is behaving elastically, the relationship between the load and the deflection should be linear. For pavements which are fully supported, the graphs should intersect the deflection axis at or near zero.

Those graphs which intersect the deflection axis at a point greater than .003 inch indicate that voids may be present beneath the slabs (40).

Finally, a statistical analysis using Duncan grouping with an α value of 0.10 (confidence interval of 90%) was performed to indicate if there are any significant difference in the deflection or load transfer efficiency values.

In the following sections, each project will be analyzed separately using the testing methodology discussed before.

5.2 US12 From West of Mina at MRM 275 East 12.2 Miles to Aberdeen in Edmunds and Brown Counties

The existing pavement structure consists of 7 inch of undoweled plain jointed concrete with 4 inch of lime-treated cushion base. The subgrade soils are generally characterized by low plasticity clays (CL). The highway is either two or four lane divided roadway with asphalt concrete shoulders. Skewed joints were placed at variable distances of 16, 17 and 19 ft. The average daily traffic (ADT) is about 2400 vehicles. The pavement was constructed in 1971 and rehabilitated in 1989. The condition of the pavement at the time of construction was marginal for rehabilitation. Pavement restoration work included joint resealing, crack sealing, concrete repair, undersealing, installing edge drains, shoulder restoration, and grinding. A two mile segment was not undersealed and experienced considerable concrete failure in the full-depth repair areas the first winter after rehabilitation. Four subsections were selected for testing and evaluation. The starting locations for these subsections are as follows:

1. MRM 287 west bound (undersealed).
2. MRM 283 west bound (non-undersealed).
3. MRM 286 east bound (undersealed).
4. MRM 283 east bound (non-undersealed).

5.2.1 MRM 287 West Bound & MRM 283 West Bound

US12, MRM 287, WB is a four lane divided highway that was undersealed and ground in 1989. US12, MRM 283, WB is a two lane highway which is part of the two mile stretch that was not undersealed. However, this subsection was ground to eliminate the presence of faulting. Although the number of lanes is different, the two subsections will be compared as they represent the west bound traffic direction. The hole pattern for grout injection in US12, MRM 287, WB consisted of five holes (see Figure 5.5). Visual surveys and FWD testing of subsection 1 were conducted on April 6, 1993. The pavement surface temperature was about 40°F. US 12, MRM 287, WB was tested again in the afternoon of May 10, 1993 to acquire the locked-up deflection data. The pavement surface temperature during that afternoon was about 83°F. Four years after undersealing was completed on US12, MRM 287, WB, the pavement sections were in good conditions. There were no visible signs of pumping or faulting in the slabs. However, low severity D-cracking and patching were observed in some panels (see Figure 5.6).

US 12, MRM 283, WB was tested at 3:00 p.m. on April 6, 1993.

Figure 5.5. Hole Pattern Arrangement Used on US12 in Aberdeen.

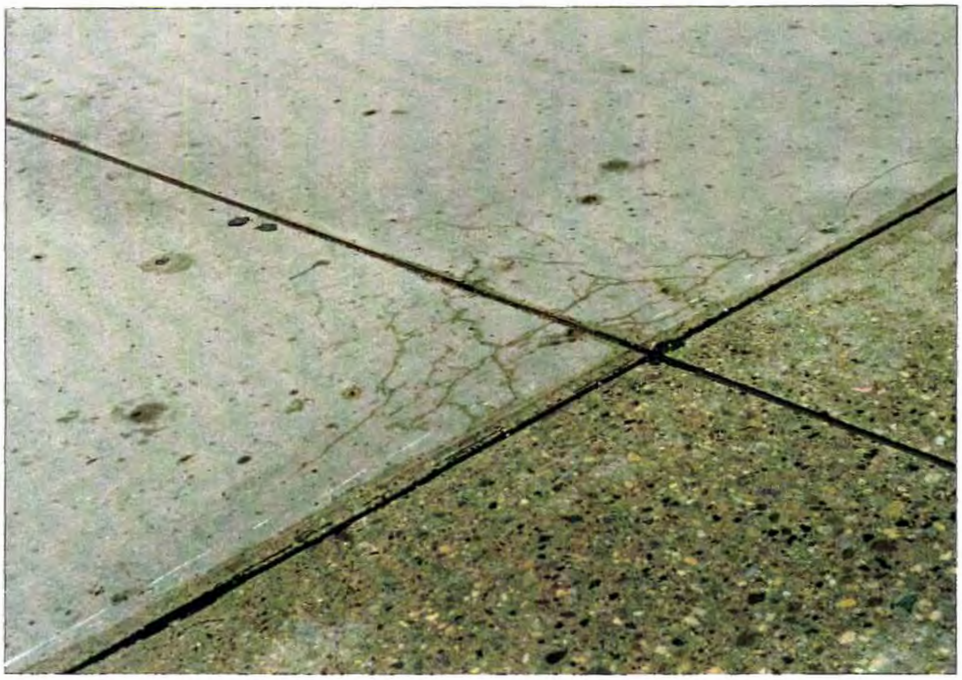


Figure 5.6. Low Severity D-Cracking Observed on US12, MRM 287, WB.

The pavement temperature during testing was about 47°F. Therefore, the joints were in an unlocked condition. No FWD testing was performed when the joints were locked up. While no pumping was observed in US12, MRM 283, WB, 18 out of 26 slabs had an average faulting of 0.125 inch. Also, low severity D-cracking and patching were observed in some panels.

Figures 5.7 and 5.8 show the representative deflection profiles for US12, MRM 287, WB and US12, MRM 283, WB. The average leave slab deflection of those joints that fell above the mean plus one standard deviation US12, MRM 287, WB was about 0.0331 inch. For US12, MRM 283, WB, this average was approximately 0.0528 inch. A 37 percent reduction in deflection was obtained between the undersealed and non-undersealed

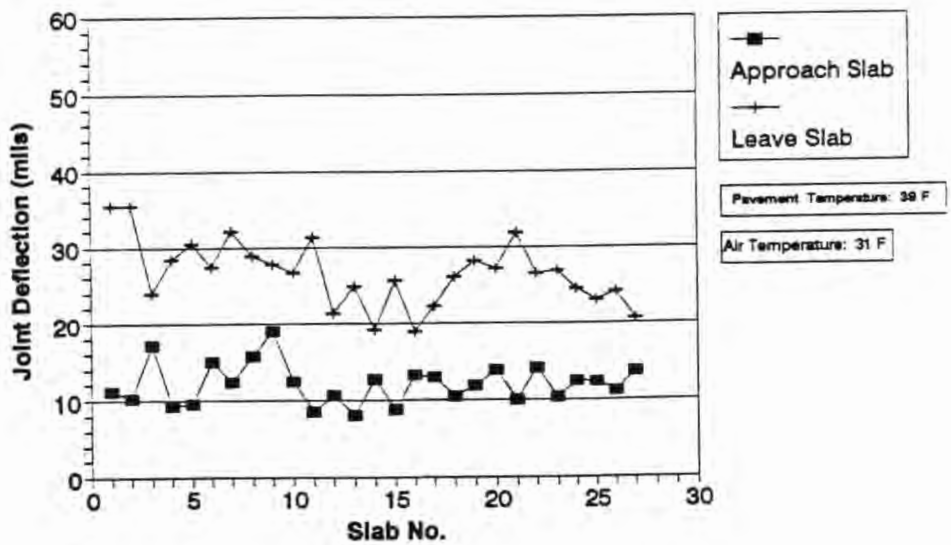


Figure 5.7. Deflection Profile for US12, MRM 287, WB.

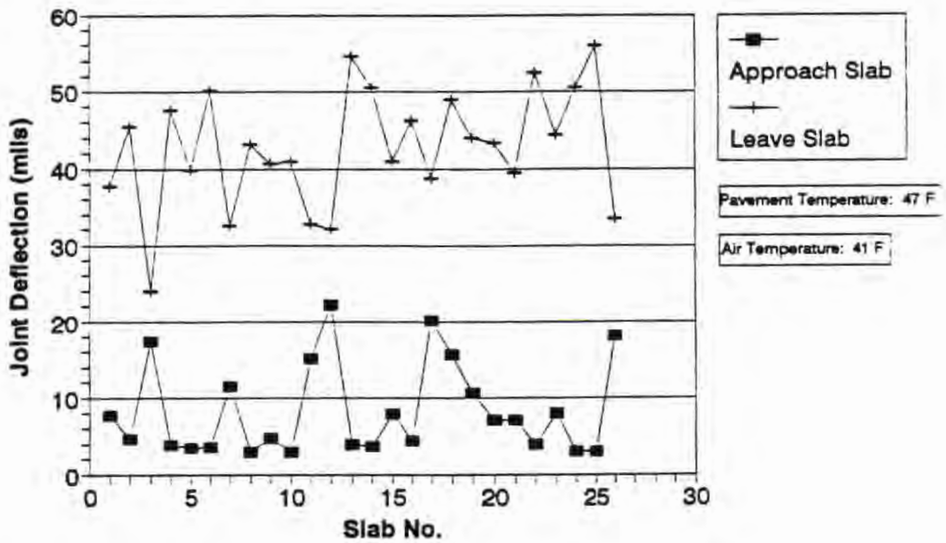


Figure 5.8. Deflection Profile for US12, MRM 283, WB.

subsections. The average load transfer efficiency (LTE) for the undersealed subsection is about 46.1% in comparison with 21.9% for the non-undersealed subsection. A 114 percent increase in LTE was obtained between the undersealed and the non-undersealed subsections (see Figure 5.9). Although there is an improvement in LTE values as a result of undersealing the pavement, values such as 46.8 and 21.9 percent are low enough to indicate that potential faulting and pumping problems may arise in the near future.

Figures 5.10 through 5.14 show plots of load versus deflection for those leave slabs in US12, MRM 287, WB that have deflection values greater than 0.0331 inch. The data shows that these lines intersect the deflection axis at values less than

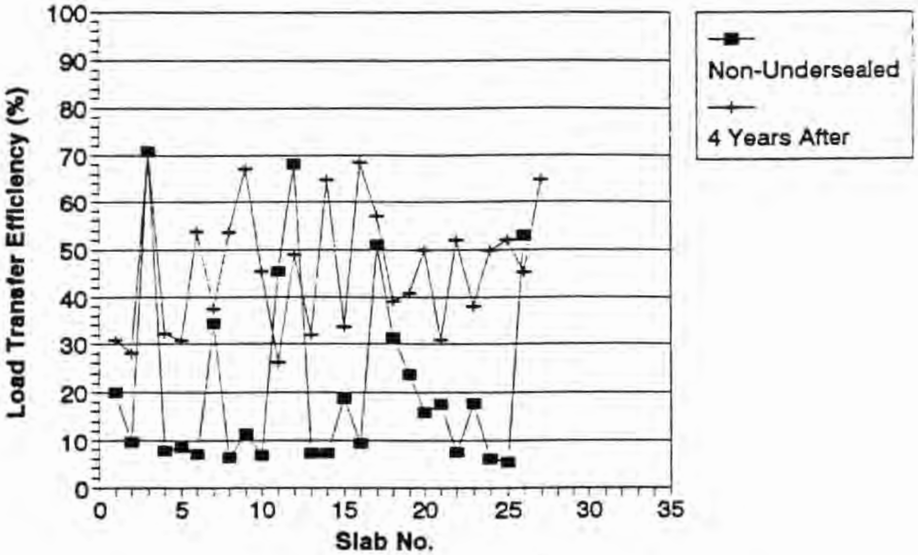


Figure 5.9. Comparison of Load Transfer Efficiency Values Between US12, MRM 287, WB and US12, MRM 283, WB.

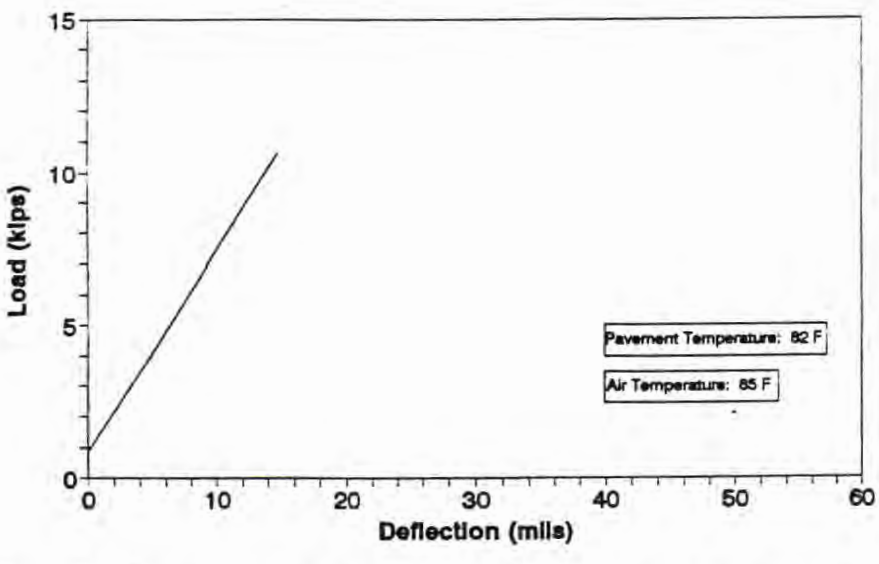


Figure 5.10. Load Versus Deflection for Slab 10, US12, MRM 287, WB.

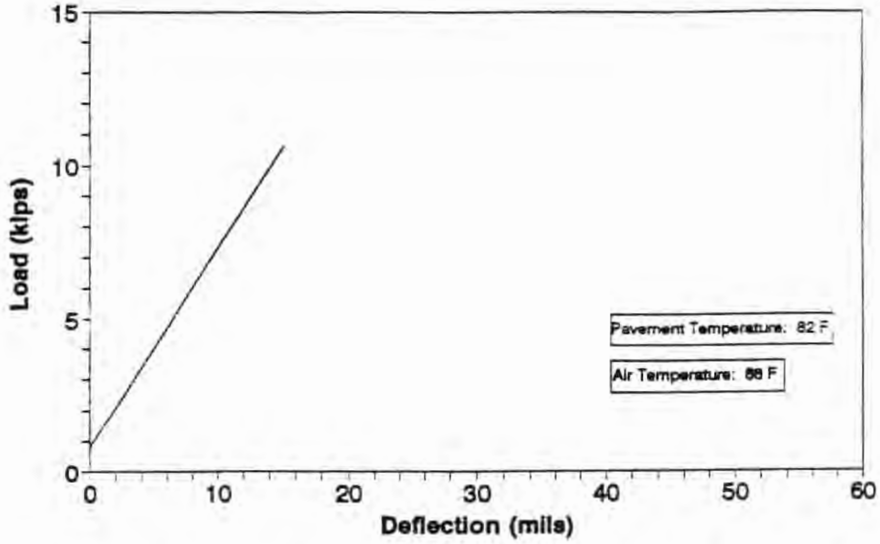


Figure 5.11. Load Versus Deflection for Slab 16, US 12, MRM 287, WB.

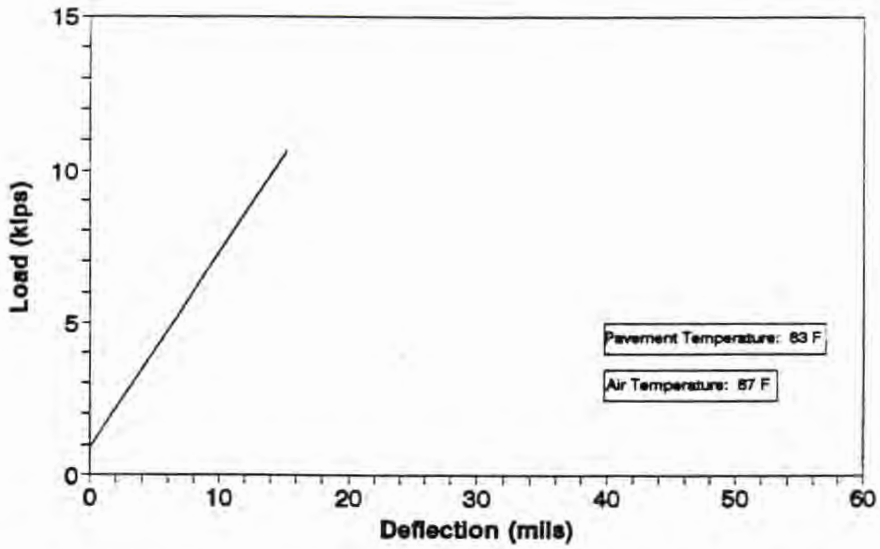


Figure 5.12. Load Versus Deflection for Slab 19, US12, MRM 287, WB.

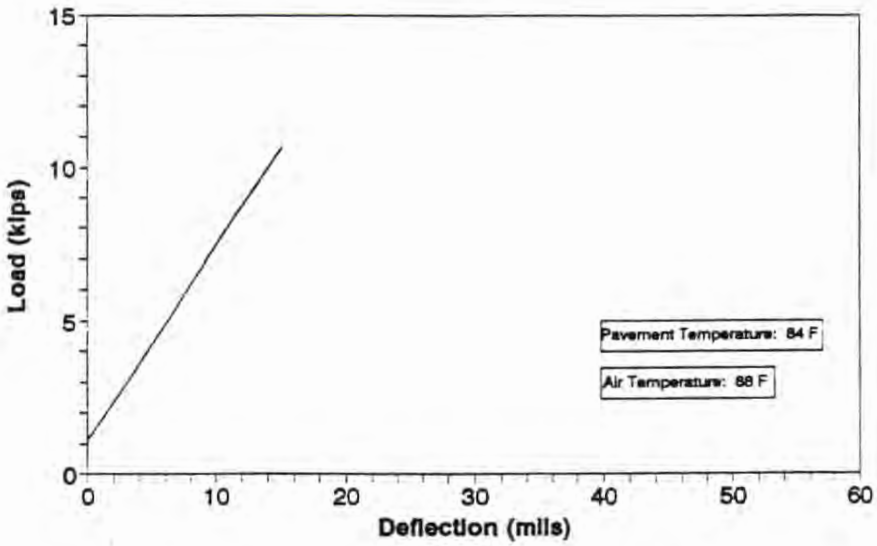


Figure 5.13. Load Versus Deflection for Slab 22, US12, MRM 287, WB.

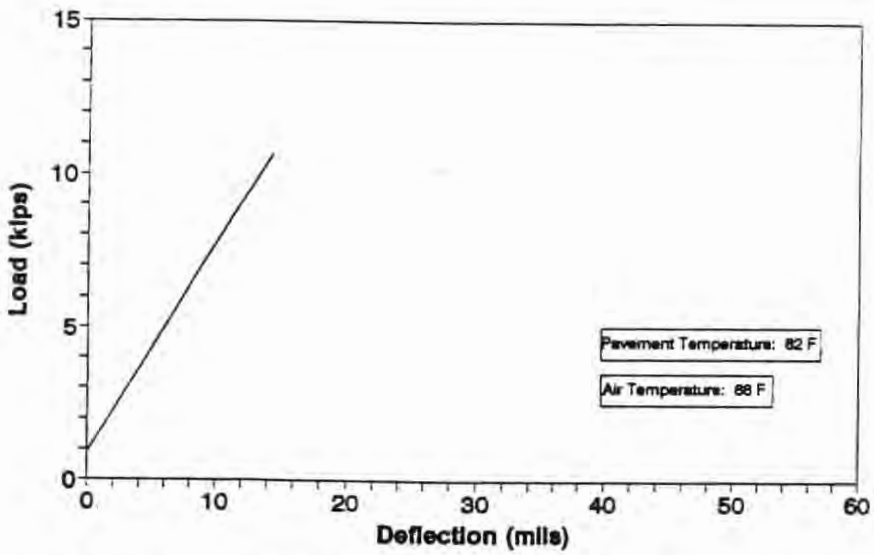


Figure 5.14. Load Versus Deflection for Slab 23, US12, MRM 287, WB.

zero. This indicate that no voids exist below this undersealed pavement (33). No such analysis could be done for US12, MRM 283, WB because the pavement was not tested under locked-up conditions.

5.2.2 MRM 286 East Bound and MRM 283 East Bound

US12, MRM 286, EB was undersealed and ground in 1989 and has the same characteristics as US12, MRM 287, WB. US12, MRM 283, EB was only ground in 1989 and has the same characteristics as US12, MRM 283, WB. US12, MRM 286, EB was tested on April 6, 1993 between 11:30 a.m. and 2.00 p.m. The average pavement temperature was 42°F. No FWD testing was conducted while the pavement edges were in locked-up conditions. Similar to US12, MRM 287, WB, there were no visible signs of pumping or faulting in US12, MRM 286, EB. US12, MRM 283, EB was tested on April 7, 1993. The average pavement temperature was 39°F and the weather was rainy on that day. There were visible signs of clear water being ejected from some joints near the shoulder. However, no significant faulting was measured. Figure 5.15 shows a photograph of low to medium severity cracks that have radiated around the joints in US12, MRM 283, EB. These cracks started from the edge corner and extended to the middle of the lane. Also, this pavement subsection showed low to medium severity patching. US12, MRM 283, EB was tested again in the afternoon of May 12, 1993. The pavement temperature was 89°F.

The deflection profiles for US12, MRM 286, EB and US12, MRM 283, EB are shown in Figures 5.16 and 5.17, respectively.

Figure 5.15. Low to Medium Severity Cracking Observed on US12, MRM 283, EB.

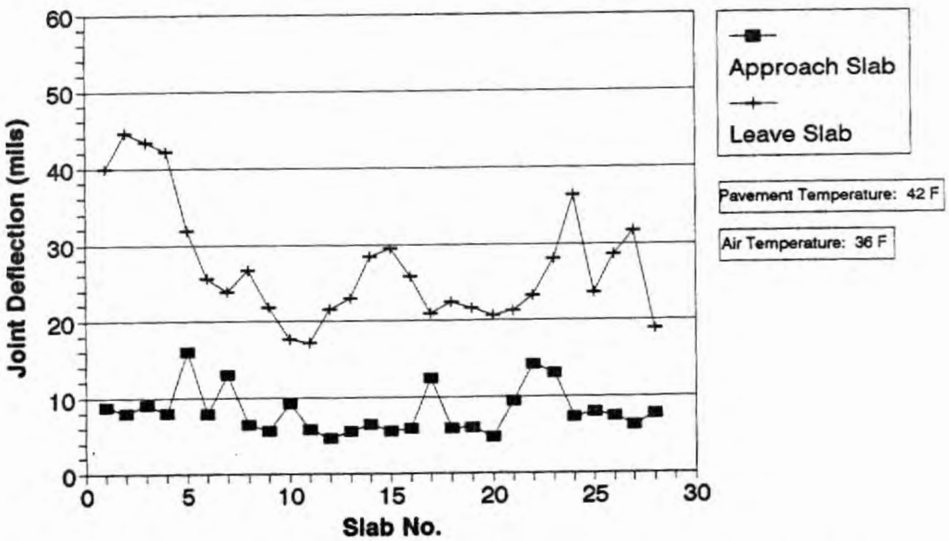


Figure 5.16. Deflection Profile For US12, MRM 286, EB.

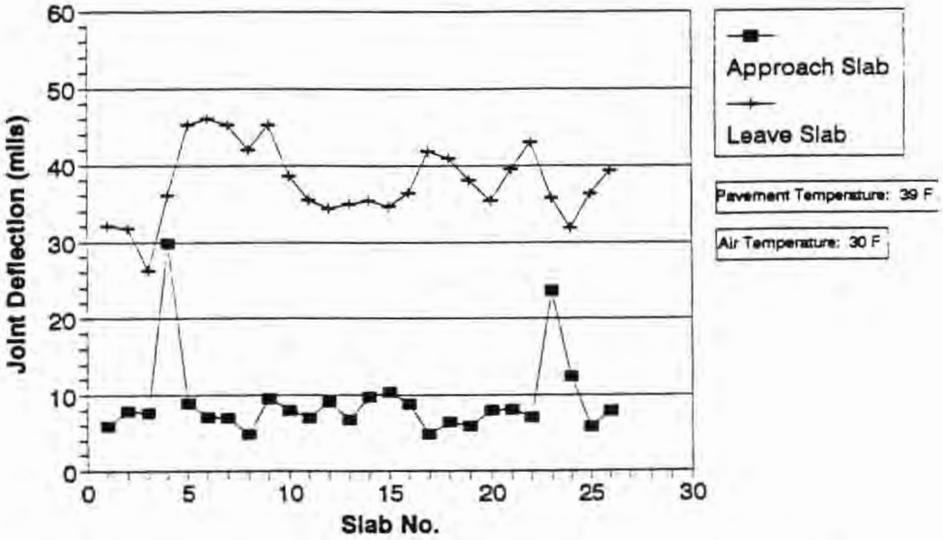


Figure 5.17. Deflection Profile for US12, MRM 283, EB.

Average leave slab deflections of 0.0412 and 0.0420 inch were obtained for US12, MRM 286, EB and US12, MRM 283, EB, respectively. A two percent reduction in deflection was obtained between the undersealed and non-undersealed sections. It should be noted here that in US12, MRM 283, EB, twelve out of 26 slabs yielded deflection values more than the mean plus one standard deviation. Figure 5.18 shows a comparison of load transfer efficiency values obtained for both subsections.

The average joint efficiency value for US12, MRM 286, EB was 31.2% in comparison with 24.5% for US12, MRM 283, EB. A 24 percent increase in LTE was obtained between the undersealed and non-undersealed subsections. However, LTE values of 31.2 and 24.5 percent are low and may indicate undesirable pavement performance in the future.

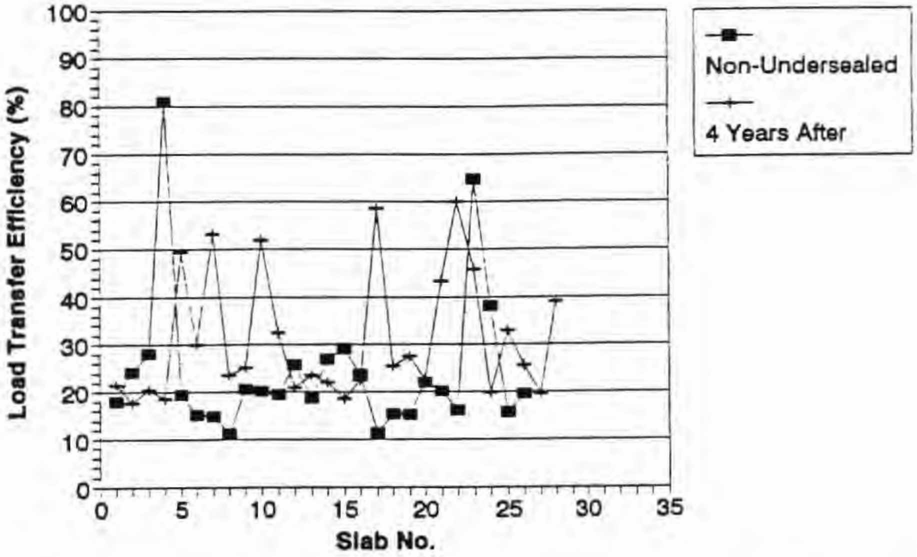


Figure 5.18. Comparison of Load Transfer Efficiency Values Between US12, MRM 286, EB and US12, MRM 283, EB.

Figures 5.19 through 5.23 indicate that although US12, MRM 283, EB was not undersealed, no voids were present below the slabs. Considering that no significant faulting was developed during the last 4 years and that no voids were present below the pavement, there is no clear interpretation for the low load transfer efficiency and high deflections data that were obtained in US12, MRM 283, EB.

5.2.3 Statistical Analysis

The results of the Duncan grouping statistical analysis are presented in Tables 5.2 and 5.3. Table 5.3 indicate that the deflections of the undersealed subsections are the same and they are significantly lower than the deflections of the non-undersealed subsections. Also, the mean load transfer efficiency

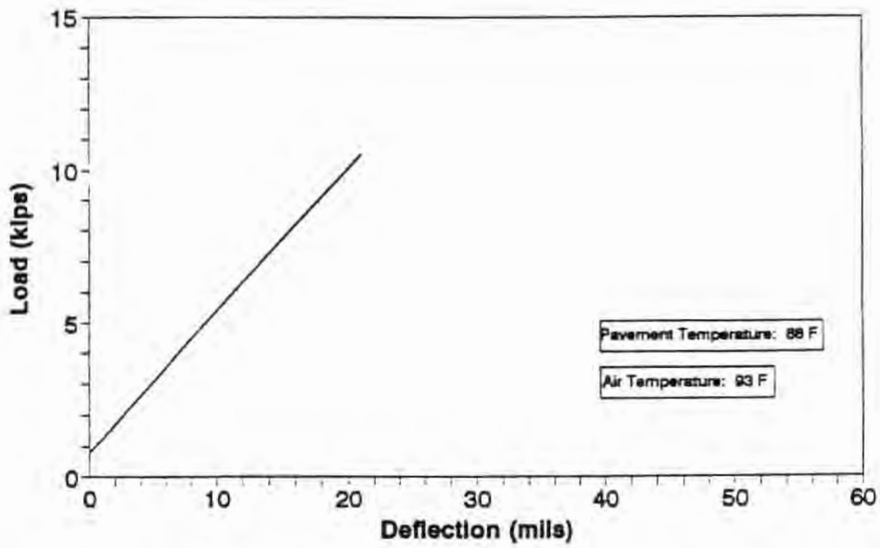


Figure 5.19. Load Versus Deflection for Slab 1, US12, MRM 283, EB.

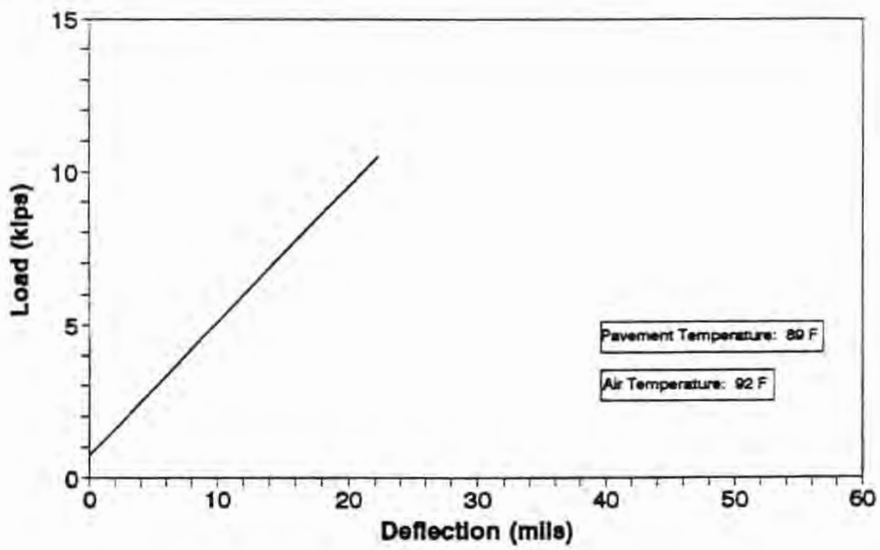


Figure 5.20. Load Versus Deflection for Slab 2, US12, MRM 283, EB.

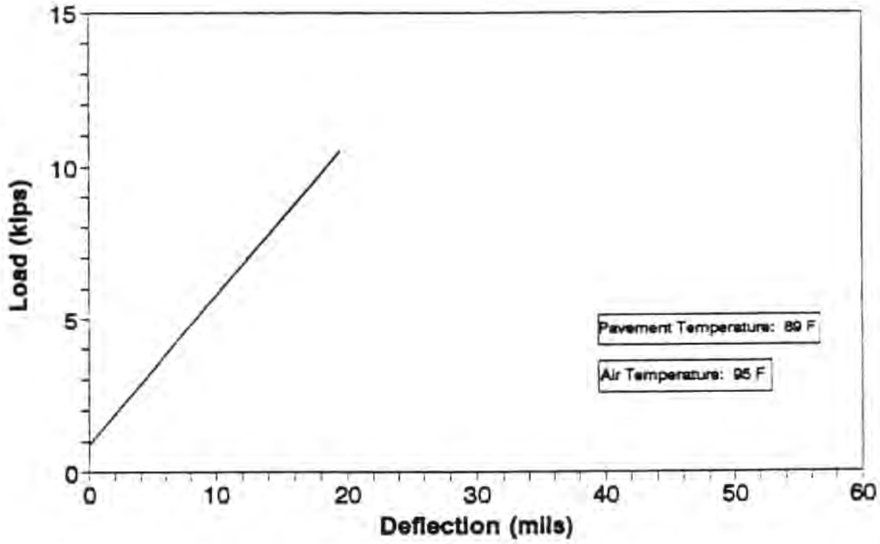


Figure 5.21. Load Versus Deflection for Slab 4, US12, MRM 283, EB.

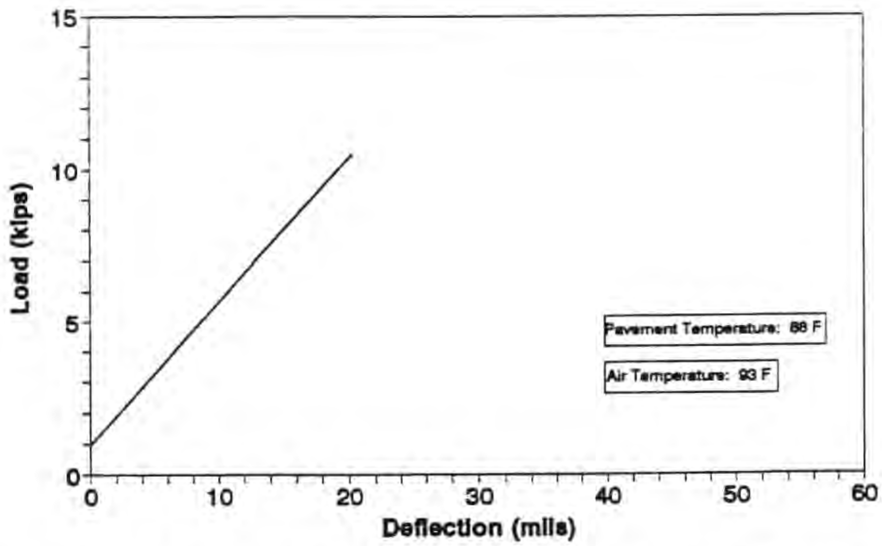


Figure 5.22. Load Versus Deflection for Slab 19, US12, MRM 283, EB.

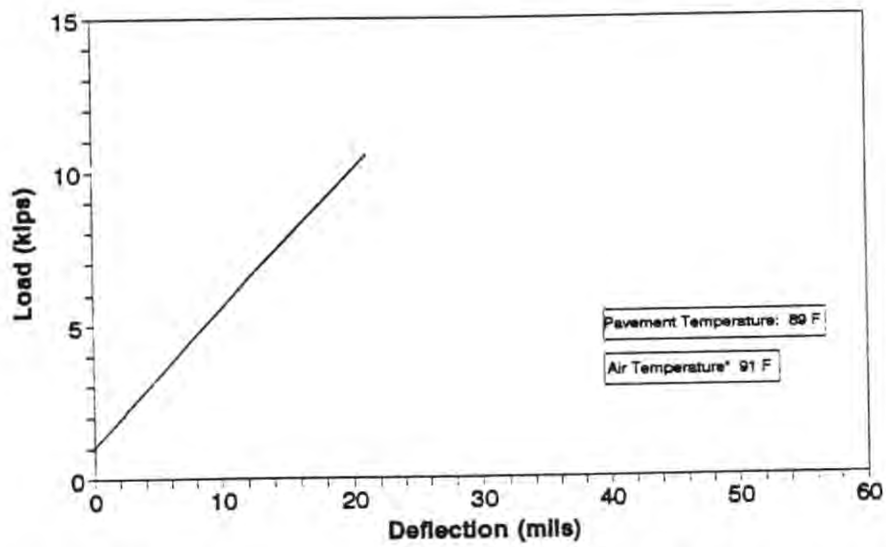


Figure 5.23. Load Versus Deflection for Slab 21, US12, MRM 283, EB.

Table 5.2. Duncan Grouping For Load Transfer Efficiency Values (US12 from West of Mina).

Duncan Grouping		Mean LTE (%)	No. of Slabs	Subsection
	A	46.1	27	287 WB
	B	31.1	28	286 EB
C	B	24.5	26	283 EB
C		21.9	26	283 WB

Note: Means with the same letter are not significantly different.

Table 5.3. Duncan Grouping For FWD Deflections (US12 from West of Mina).

Duncan Grouping	Mean Deflection (inch)	No. of Slabs	Subsection
A	0.0427	26	283 WB
B	0.0377	26	283 EB
C	0.0271	28	286 EB
C	0.0266	27	287 WB

for US12, MRM 287, WB is the highest and is significantly different from the other subsections. US12, MRM 283, WB and US12, MRM 283, EB as well as US12, MRM 286, EB and US12, MRM 283, EB are not significantly different. However, US12, MRM 286, EB is significantly different from US12, MRM 283, WB.

5.2.4. Comparison of Locked and Unlocked Cases

In section 5.1.1, it was stated that when the joints are in the locked-up condition, the load transfer efficiency values will be higher. The locked-up condition also tends to result in lower deflections during FWD testing. Figures 5.24 and 5.25 show comparisons of load transfer efficiency values in both the locked and unlocked conditions for US12, MRM 283, EB and US12, MRM 287, WB, respectively. In both cases, the load transfer efficiency of the locked condition was always much greater than that of the

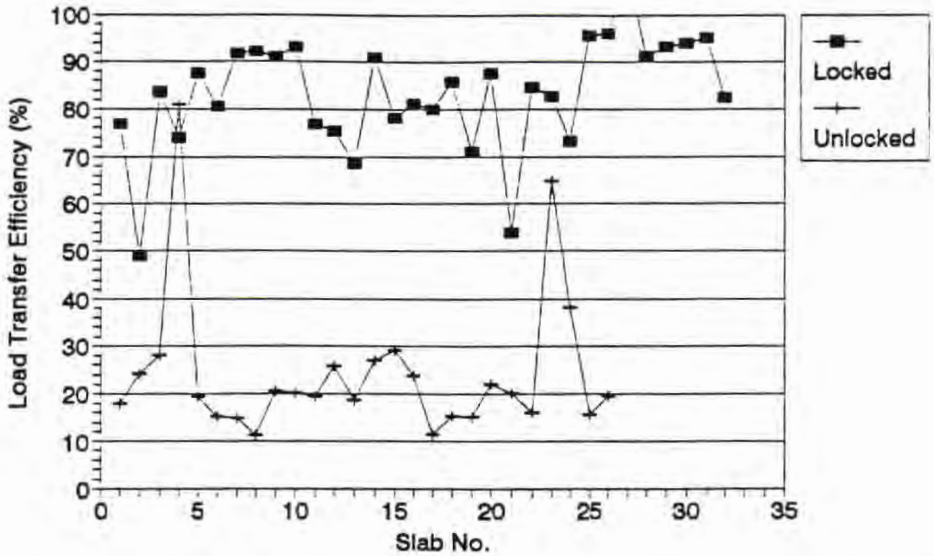


Figure 5.24. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in US12, MRM 283, EB.

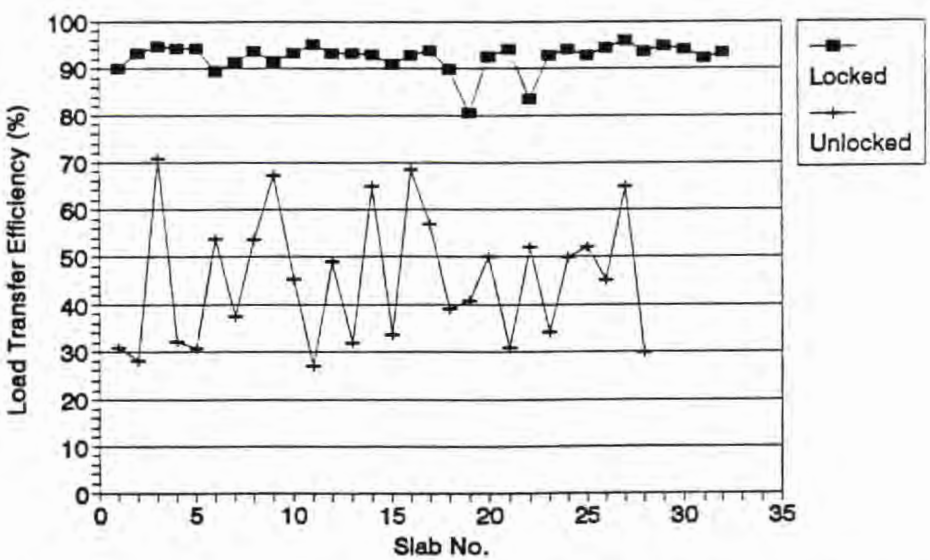


Figure 5.25. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in US12, MRM 287, WB.

unlocked condition. Figures 5.26 and 5.27 show comparisons of leave slab deflection in both the locked and unlocked conditions for US12, MRM 283, EB and US12, MRM 287, WB, respectively. In both cases, the leave slab deflection of the locked condition was always less than that of the unlocked condition.

5.3. US281 from US12 South 3.2 Miles in Brown County

This highway was originally constructed in 1971 with 7 inches of undoweled plain jointed concrete and 4 inches of lime-treated base. Low plasticity clays are the predominant type of soils at the site. This highway is a 4 lane undivided roadway with asphalt concrete shoulders and skewed joints spaced at 20 ft. The current average daily traffic (ADT) is about 4100 vehicles. In 1987, the highway was undersealed and ground. A

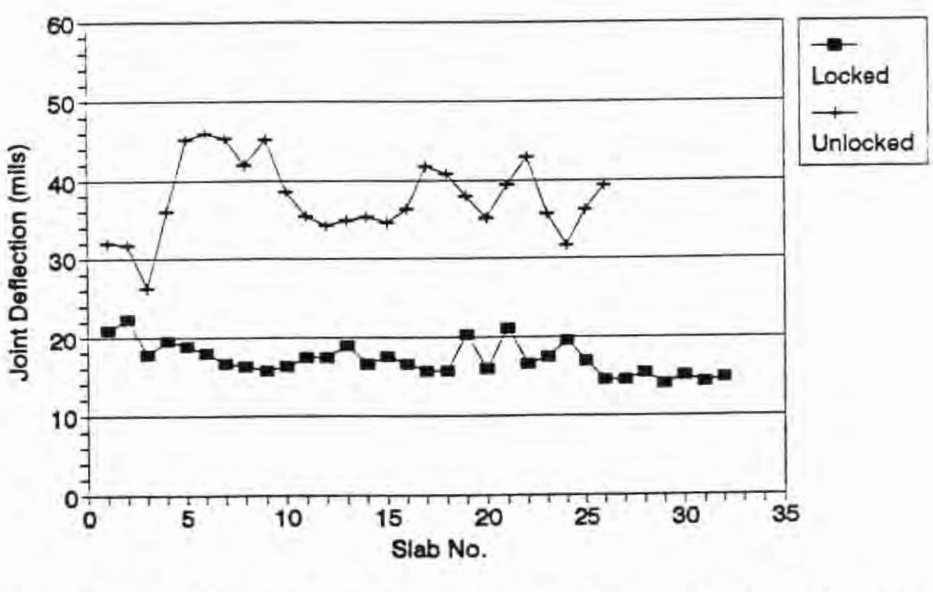


Figure 5.26. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in US12, MRM 283, EB.

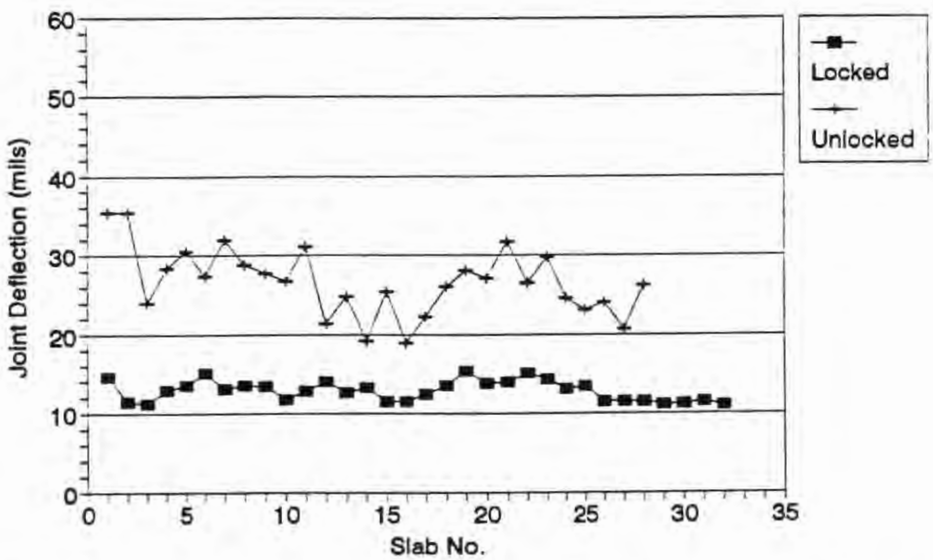


Figure 5.27. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in US12, MRM 287, WB.

three hole pattern arrangement was used for grout injection. The condition of the concrete at the time of rehabilitation was good but with severe faulting. A 500 ft pavement section on the north bound was ground but not undersealed. Three subsections were selected for FWD testing and evaluation and they are as follows:

1. MRM 192 south bound (undersealed).
2. MRM 194 south bound (undersealed).
3. Melgaard road north bound (non-undersealed).

Only the driving lane in US281, MRM 192, SB was undersealed while both lanes in US281, MRM 194, SB were undersealed. US281, MRM 192, SB was tested on April 8, 1993 in the morning period. The average pavement temperature was 39°F. In the driving lane in US281, MRM 192, SB; 16 out of 25 slabs experienced an average faulting of 0.09375 inch. The other non-undersealed lane had an average faulting of 0.1875 inch. Generally, US281, MRM 192, SB was in good condition with no visible patching. US281, MRM 194, SB was tested on April 7, 1993. The weather was rainy and the average pavement temperature was 37°F. Both lanes in US281, MRM 194, SB were undersealed but only the driving lane was ground. During the application of the FWD load near the joints, clear water was being ejected near the shoulder. An average faulting of 0.10625 inch was measured in this subsection. Low severity patching was also observed in some panels. US281, Melgaard Road, NB was tested during the afternoon of April 7, 1993. The average pavement temperature was 41°F. Similar to US281, MRM 194, SB; this subsection was experiencing pumping problems when FWD loads were applied at some joints. The average faulting was 0.25 inch in the driving lane and 0.125 inch in the passing lane. Furthermore, low severity patching was observed in US281, Melgaard Road, NB.

Figures 5.28 through 5.30 show the deflection profile plots for US281, MRM 192, SB; US281, MRM 194 SB; and US281, Melgaard Road, NB; respectively. The average leave slab deflections for US281, MRM 192, SB; US281, MRM 194 SB; and US281, Melgaard Road, NB; were 0.0236, 0.0284 and 0.0288 inch, respectively. Eighteen percent reduction in deflection was obtained between US281, MRM 192, SB and US281, Melgaard Road, NB. However, US281, MRM 194, SB, (undersealed) has almost the same deflection value as US281, Melgaard Road, NB (non-undersealed). The average load transfer

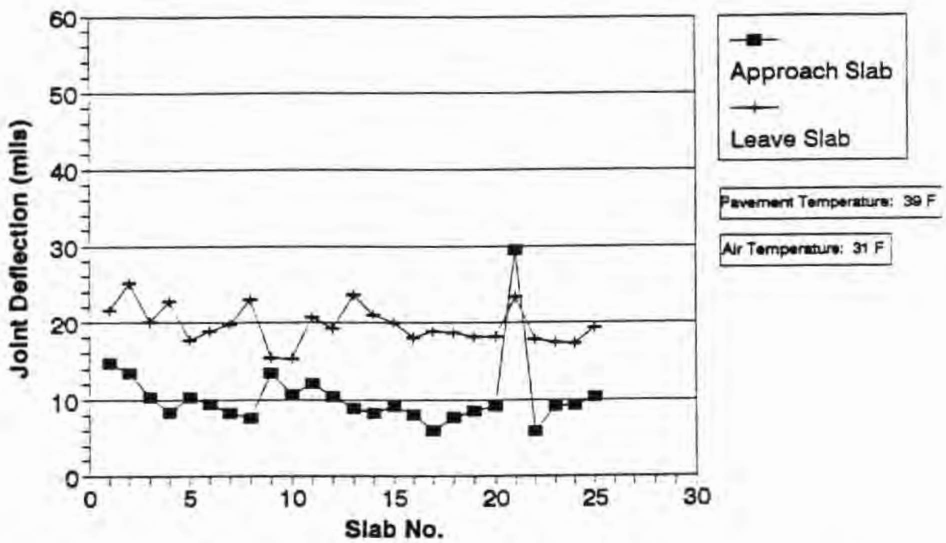


Figure 5.28. Deflection Profile for US281, MRM 192, SB.

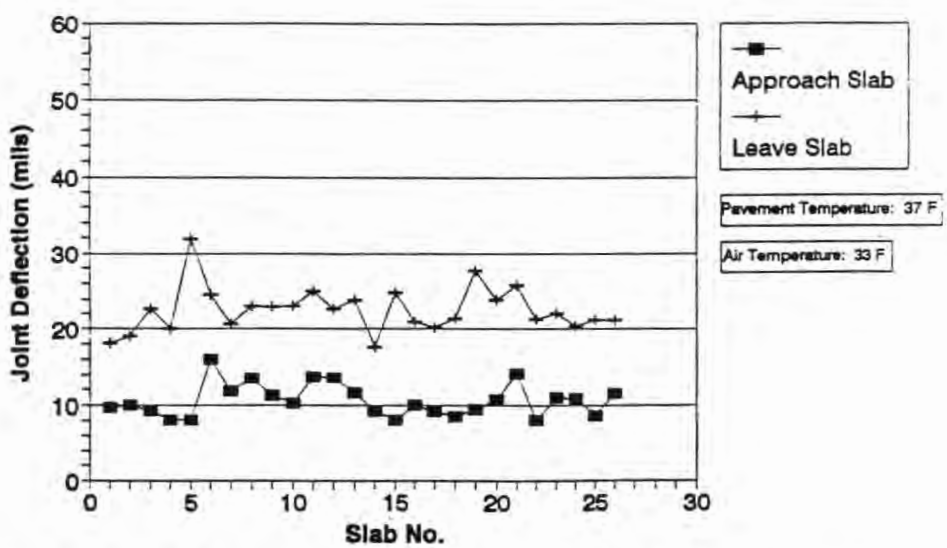


Figure 5.29. Deflection Profile for US281, MRM 194, SB.

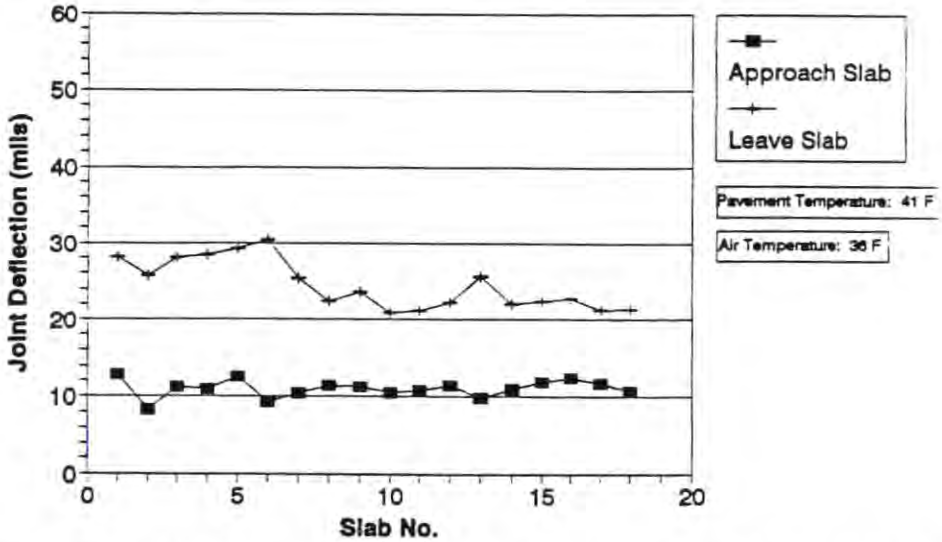


Figure 5.30. Deflection Profile for US281, Melgaard Road, NB.

efficiency values were 51.3, 46.7, and 44.6 percent for US281, MRM 192, SB; US281, MRM 194, SB; and US281, Melgaard Road, NB; respectively. A graphical comparison of the joint efficiency data is shown in Figure 5.31. Ten and five percent increases in LTE values were obtained between US281, Melgaard Road, NB and each of US281, MRM 192, SB and US281, MRM 194, SB; respectively. Load versus deflection graphs were not generated because the pavement sections were not tested during locked-up conditions.

5.3.1. Statistical Analysis

Tables 5.4 and 5.5 present the results of the statistical analysis. The mean LTE values indicate that US281, MRM 192, SB and US281, MRM 194, SB are similar. US281, MRM 192, SB is significantly different from US281, Melgaard Road, NB. However, US281, MRM 194, SB and US281, Melgaard Road, NB are not

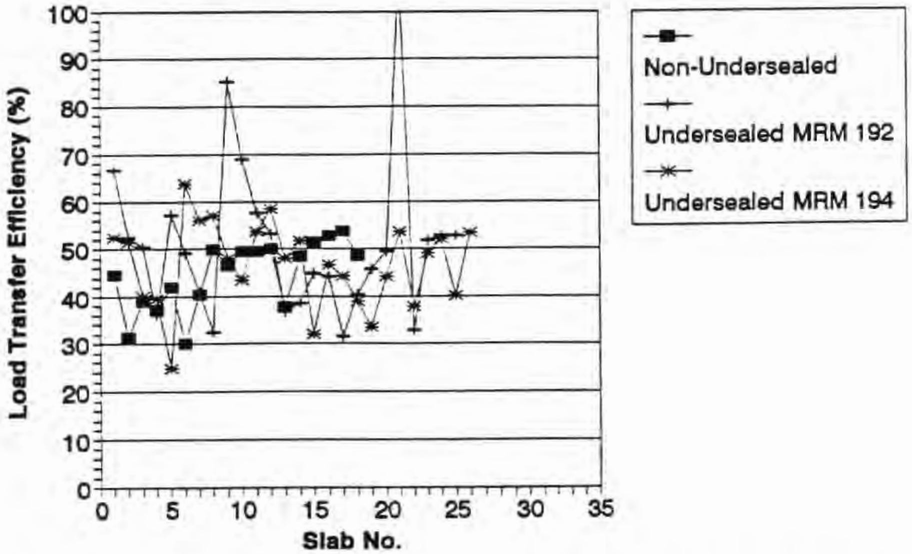


Figure 5.31. A Comparison of Load Transfer Efficiency Values for US281, MRM 192, SB; US281, MRM 194 SB; and US281, Melgaard Road, NB.

Table 5.4. Duncan Grouping For The Load Transfer Efficiency Values (US281).

Duncan Grouping		Mean LTE (%)	No. of Slabs	Subsection
	A	51.3	25	MRM 192 SB
B	A	46.7	26	MRM 194 SB
B		44.6	18	Melgaard RD. NB

Table 5.5. **Duncan Grouping For FWD Deflections (US281).**

Duncan Grouping	Mean Deflection (inch)	No. of Slabs	Subsection
A	0.0245	18	Melgaard RD. NB
B	0.0225	26	MRM 194 SB
C	0.0196	25	MRM 192 SB

significantly different. The mean deflection data indicate that all subsections are significantly different.

5.3.2. Comparison of Locked and Unlocked Conditions

No comparisons between locked and unlocked data were made because this site was not tested during locked-up conditions.

5.4 US12 From Ipswich East 11.4 Miles in Edmunds County

The pavement was constructed in 1974 and it consists of 7 inches of undoweled plain jointed concrete over 4 inches of lime-treated cushion base. The subgrade soils are characterized by low plasticity clays (CL). The highway is a 2 lane roadway with concrete shoulders and skewed joints that are placed at 14, 16, 17, and 19 ft. The current average daily traffic (ADT) is about 2350 vehicles. Prior to undersealing in June 1992, the pavement experienced faulting and failed joint seals problems. Pavement restoration works included concrete repair, undersealing, grinding, resealing joints, and sealing cracks. Four different subsections were selected for testing. The starting locations for these subsections are as follows:

1. MRM 268.4 west bound.
2. MRM 272.17 east bound.
3. MRM 275.03 + 100 west bound.
4. MRM 279.

5.4.1 MRM 268.4 West Bound

US 12, MRM 268.4, WB was tested twice. The first time was in 1992, 28 days after undersealing. The second time was on May 11, 1993, 11 months after undersealing. In 1993, pavement sections were in an excellent condition with no visible signs of distresses. The average leave slab deflections were 0.0223 and 0.0224 inch, 28 days and 11 months after undersealing, respectively. The deflection profile plots for both testing periods are shown in Figures 5.32 and 5.33.

The average joint efficiency was 81.6%, 28 days after undersealing. The average LTE was reduced to 62.7% after 11 months. A 23% reduction in LTE occurred in a 10 months period. Figure 5.34 shows a comparison of the load transfer efficiency values for both periods.

Figures 5.35 through 5.41 show plots of load versus deflection during both periods of testing. The data indicate that the pavement is fully supported 28 days and 11 months after undersealing was completed in this subsection.

5.4.2 MRM 272.17 East Bound

This subsection was also tested twice. Once in 1992, 3 days

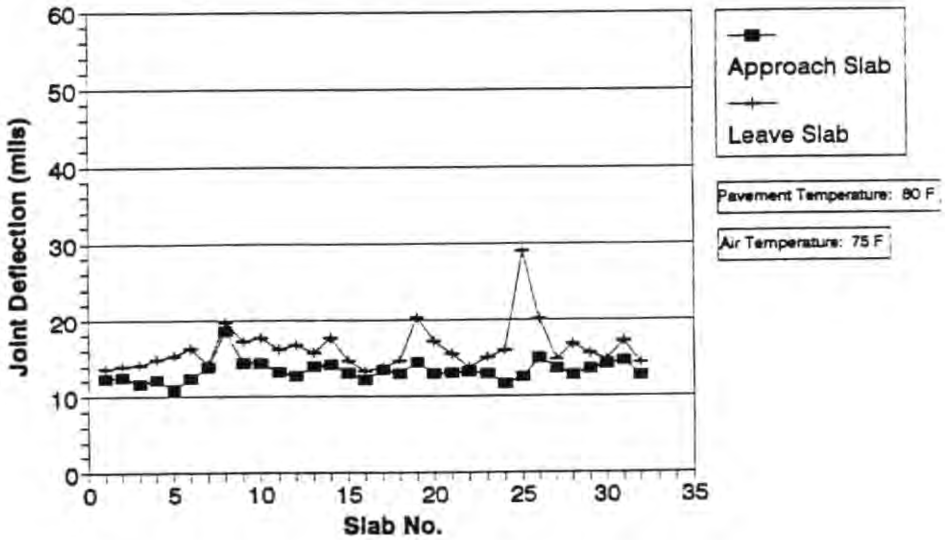


Figure 5.32. Deflection Profile for US12, MRM 268.4, WB, 28 Days After Undersealing.

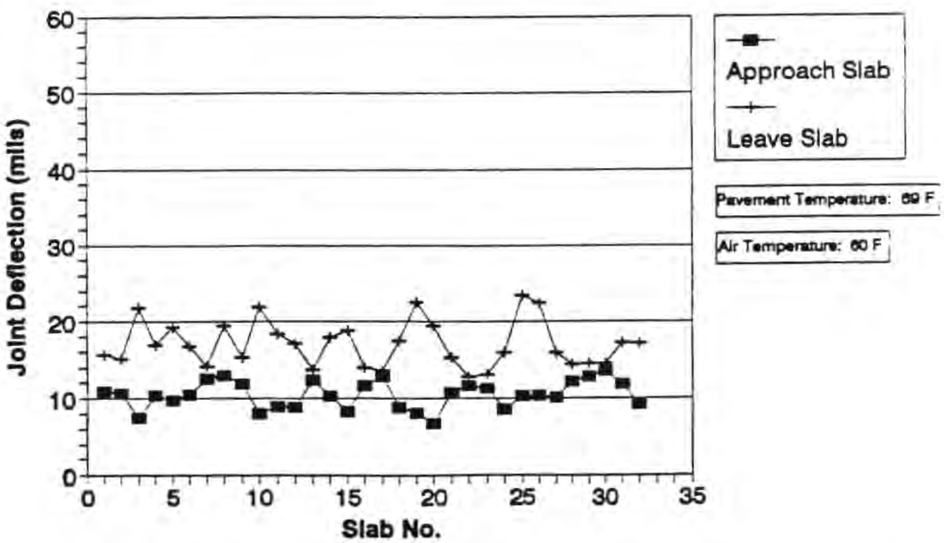


Figure 5.33. Deflection Profile for US12, MRM 268.4, WB, 11 Months After Undersealing.

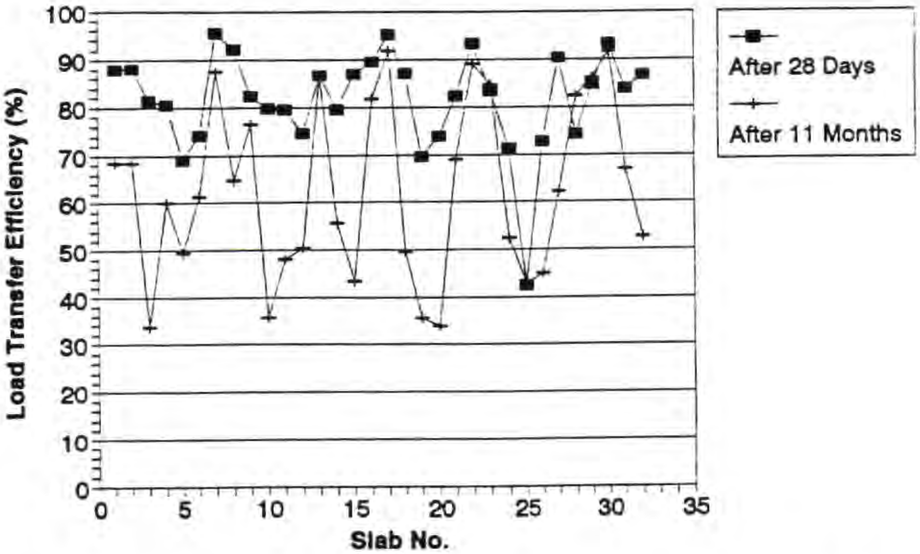


Figure 5.34. Comparison of Load Transfer Efficiency Values in US12, MRM 268.4, WB, 28 Days and 11 Months After Undersealing.

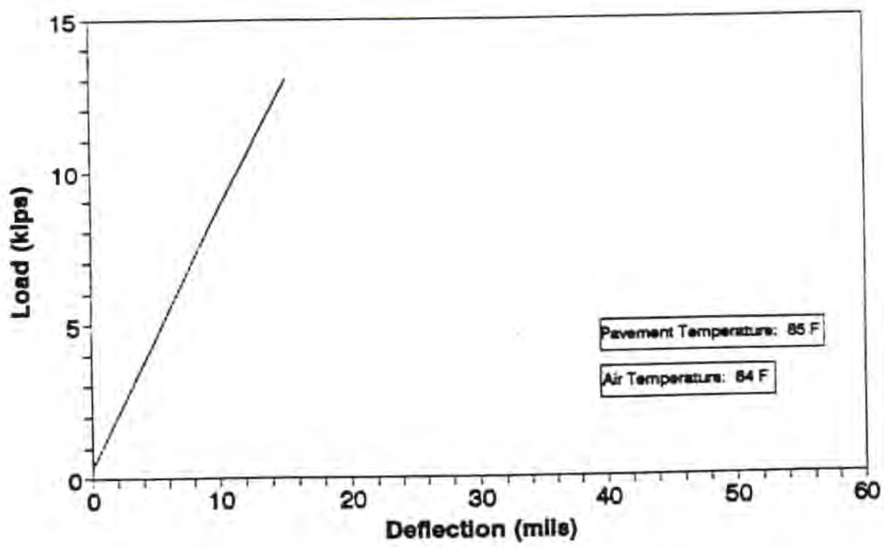


Figure 5.35. Load Versus Deflection for Slab 8, US12, MRM 268.4, WB, 28 Days After Undersealing.

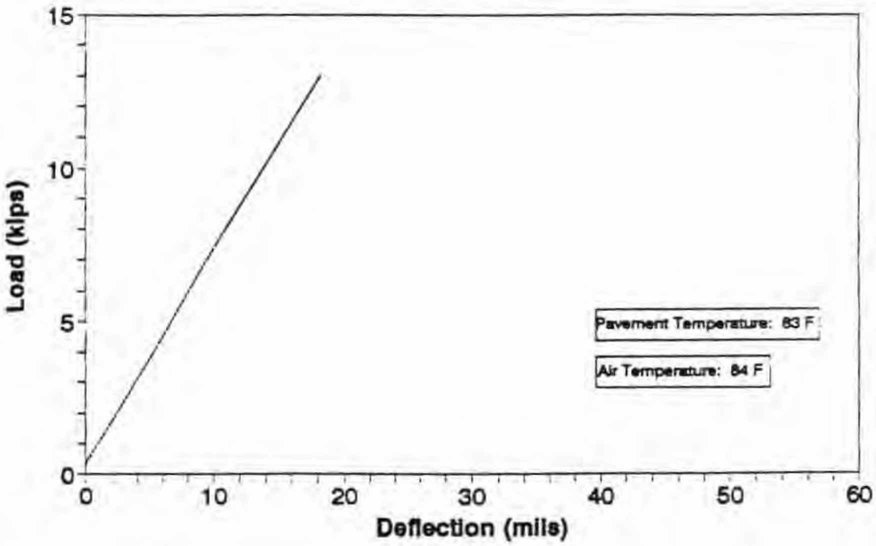


Figure 5.36. Load Versus Deflection for Slab 19, US12, MRM 268.4, WB, 28 Days After Undersealing.

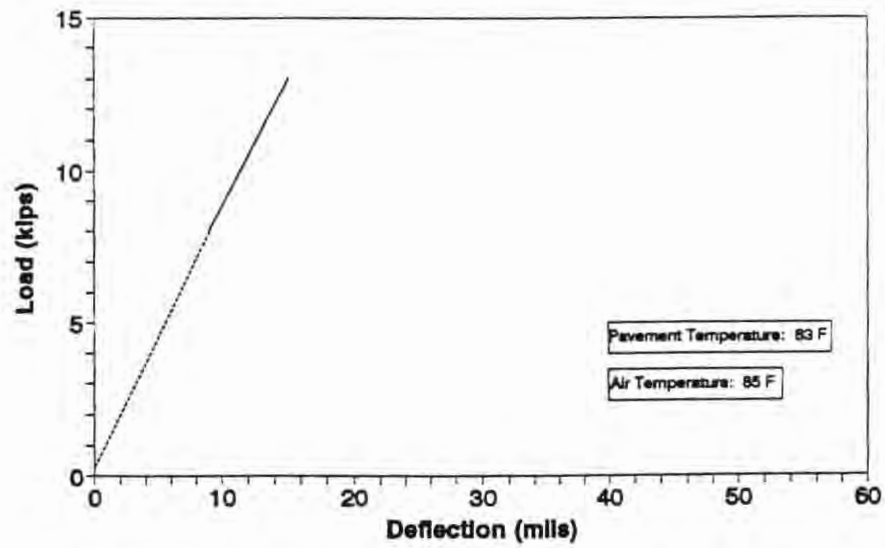


Figure 5.37. Load Versus Deflection for Slab 21, US12, MRM 268.4, WB, 28 Days After Undersealing.

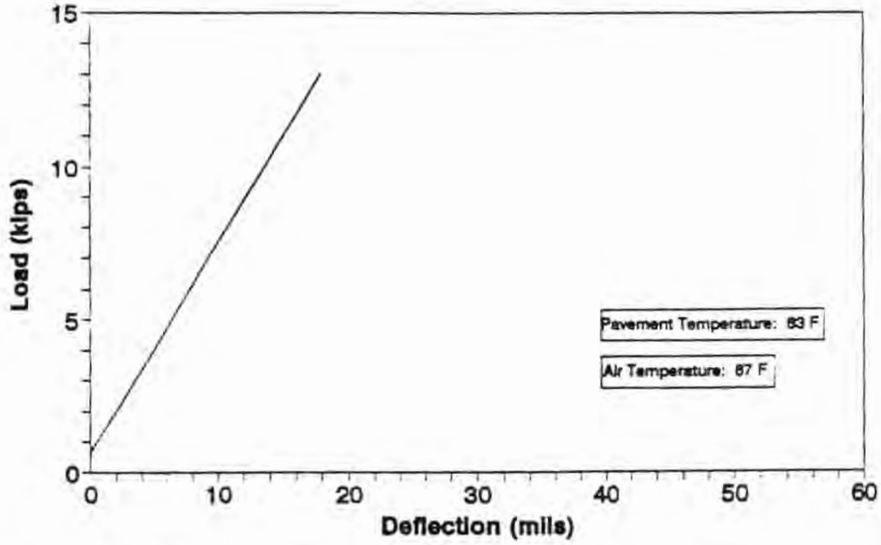


Figure 5.38. Load Versus Deflection for Slab 25, US12, MRM 268.4, WB, 28 Days After Undersealing.

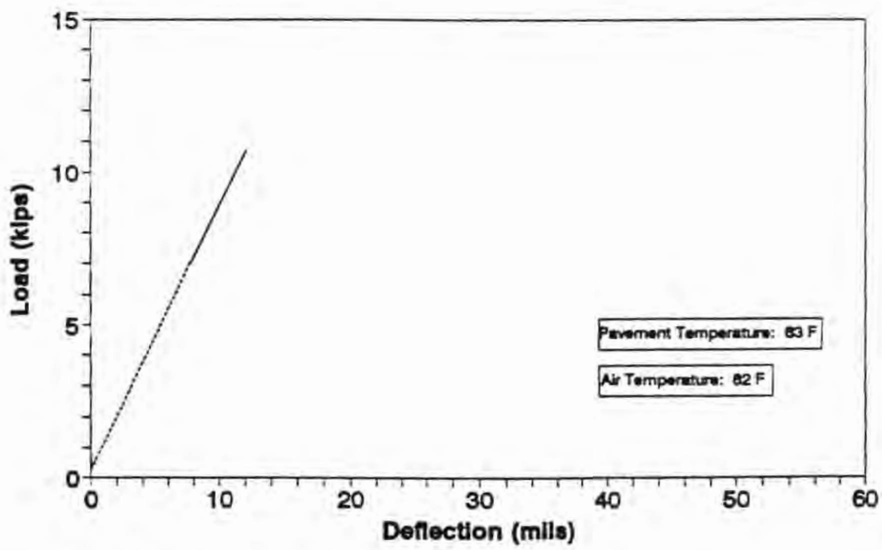


Figure 5.39. Load Versus Deflection for Slab 31, US12, MRM 268.4, WB, 11 Months After Undersealing.

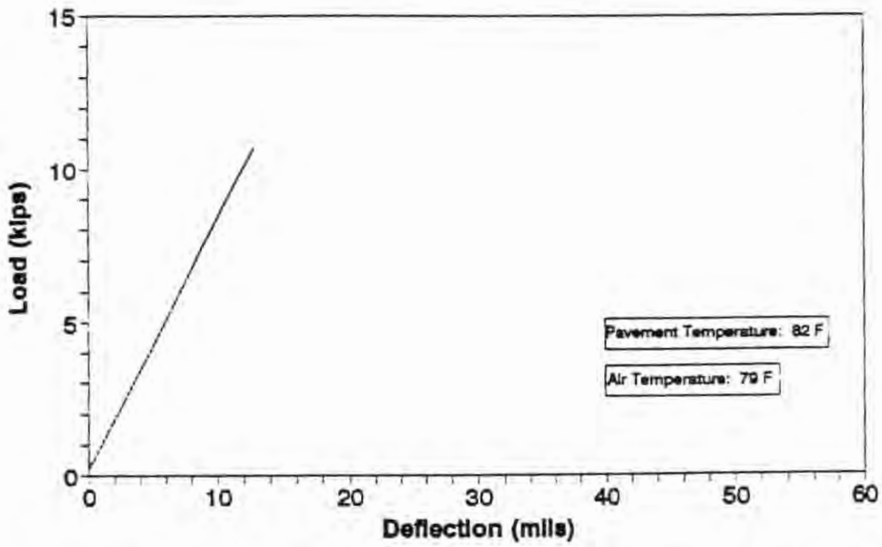


Figure 5.40. Load Versus Deflection for Slab 8, US12, MRM 268.4, WB, 11 Months After Undersealing.

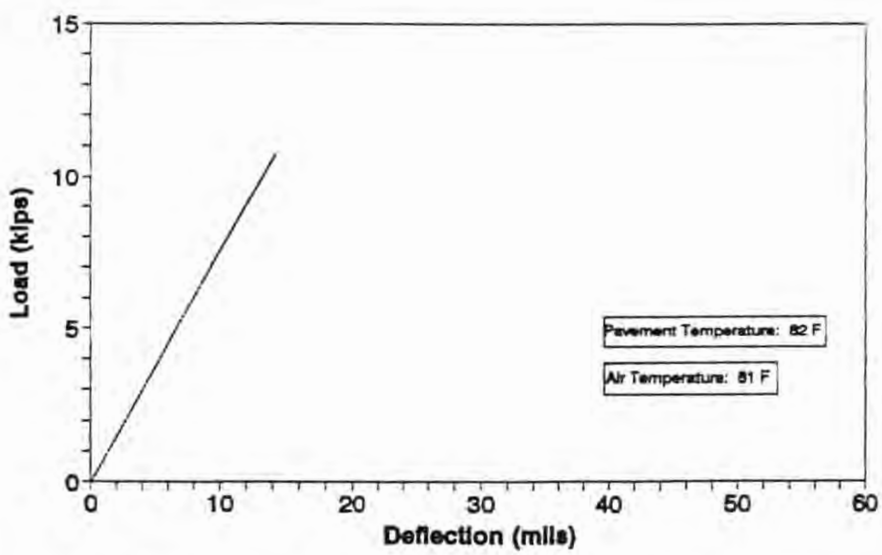


Figure 5.41. Load Versus Deflection for Slab 19, US12, MRM 268.4, WB, 11 Months After Undersealing.

after undersealing and the second time in May 12, 1993, 11 months later. No visible signs of pumping, faulting, patching, or spalling were apparent in this subsection. Figures 5.42 and 5.43 show the deflection profiles for both testing periods. The average leave slab deflections were 0.0263 and 0.0281 inch, 3 days and 11 months after undersealing. A 7% increase in deflection was obtained after 11 months.

The average load transfer efficiency values were 91.4 and 85.7 percent, 3 days and 11 months after undersealing. A 6% reduction in LTE occurred after 11 months. Figure 5.44 shows the load transfer efficiency data for both periods.

Figures 5.45 through 5.47 indicate the presence of voids, 11 months after undersealing was completed in this subsection (an

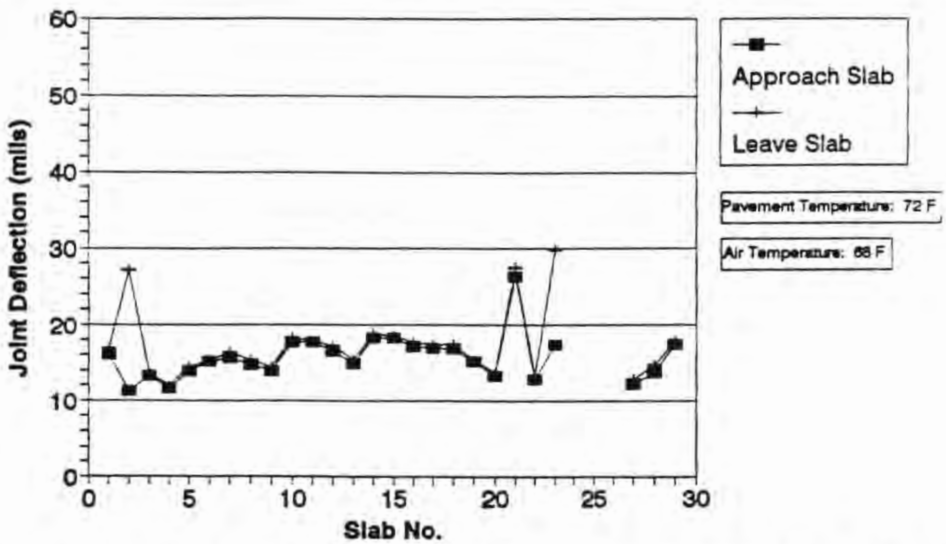


Figure 5.42. Deflection Profile for US12, MRM 272.17, EB, 3 Days After Undersealing.

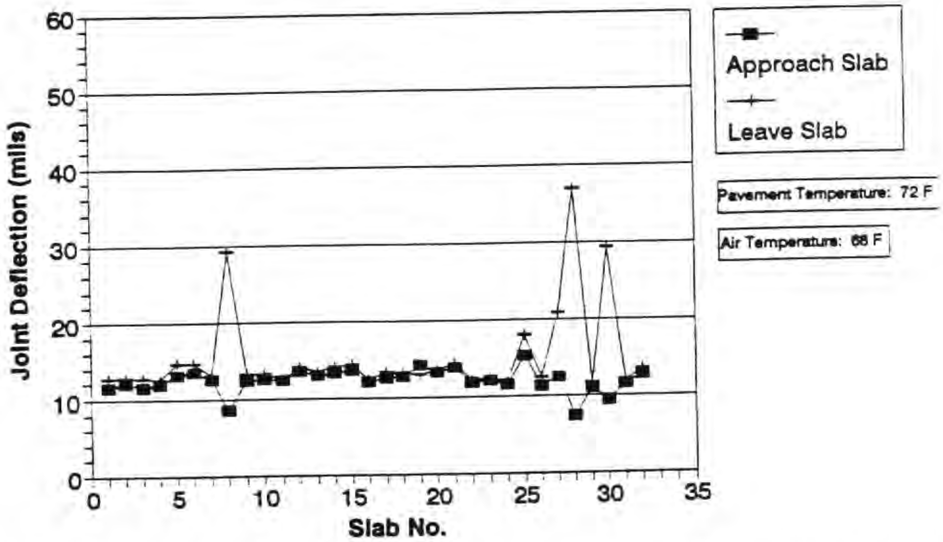


Figure 5.43. Deflection Profile for US12, MRM 272.17, EB, 11 Months After Undersealing.

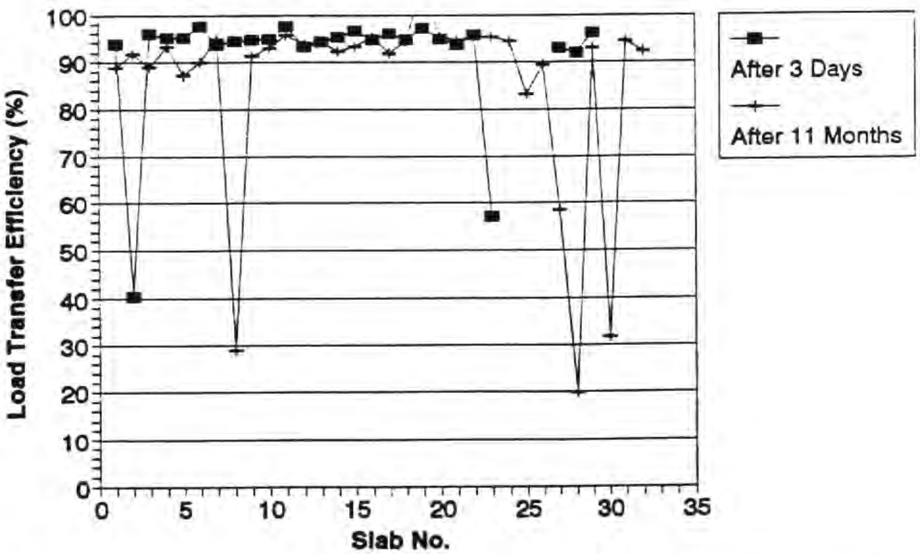


Figure 5.44. Comparison of Load Transfer Efficiency Values for US12, MRM 272.17, EB, 3 Days and 11 Months After Undersealing.

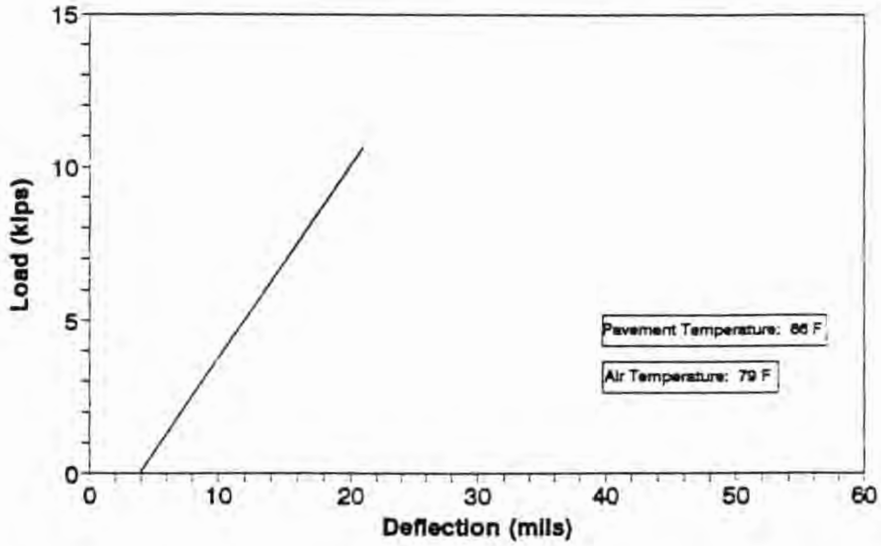


Figure 5.45. Load Versus Deflection for Slab 7, US12, MRM 272.17, EB, 11 Months After Undersealing.

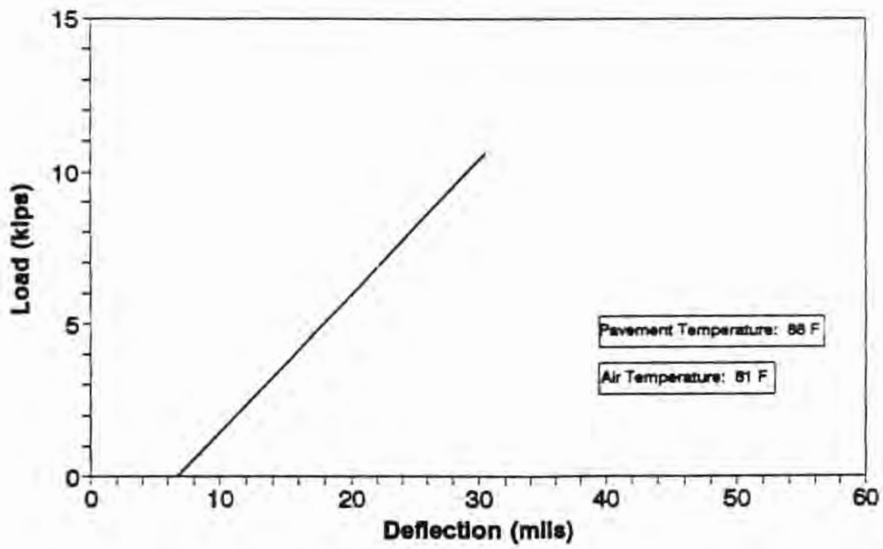


Figure 5.46. Load Versus Deflection for Slab 27, US12, MRM 272.17, EB, 11 Months After Undersealing.

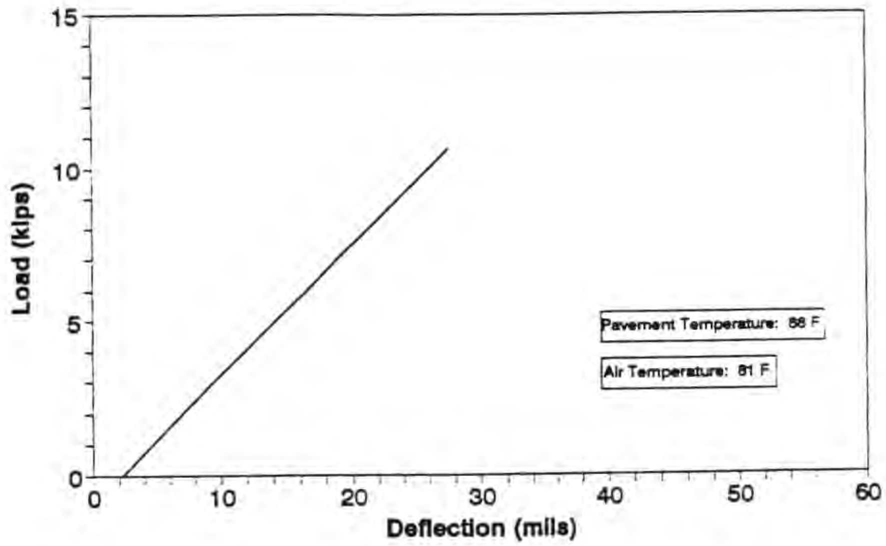


Figure 5.47. Load Versus Deflection for Slab 29, US12, MRM 272.17, EB, 11 Months After Undersealing.

average deflection intercept value of 0.0045 inch was determined). Such deflection can be considered high under the existing pavement temperature.

5.4.3. MRM 275.03 + 100 West Bound

This subsection was tested in 1992 prior to undersealing and was tested again in May 13, 1993, 11 months after undersealing. Similar to the other pavement subsections at this site, no distresses were observed in this subsection. Prior to undersealing, the average leave slab deflection was 0.0428 inch. This deflection was reduced to 0.0170 inch, 11 months after undersealing (60% reduction). Figures 5.48 and 5.49 show the deflection profile plots prior to and after undersealing, respectively.

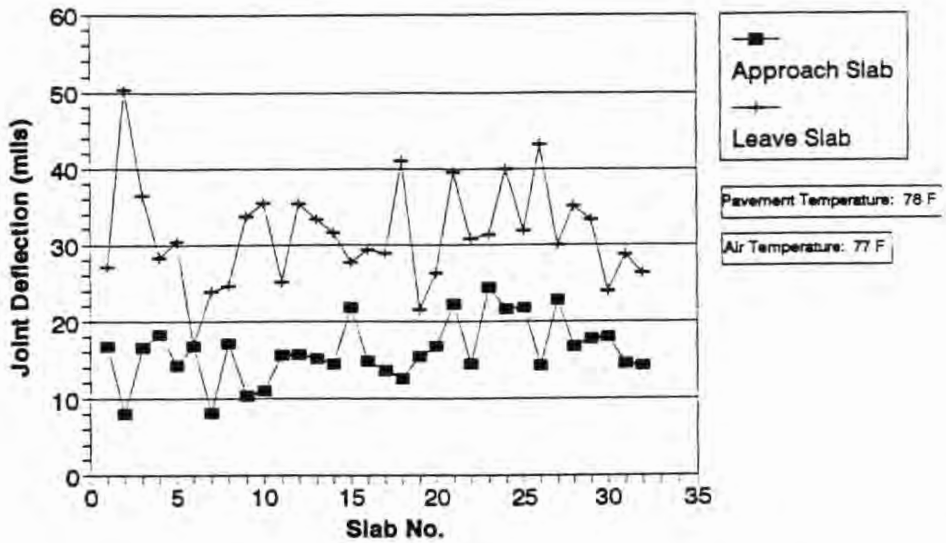


Figure 5.48. Deflection Profile for US12, MRM 275.03 + 100, WB, Prior to Undersealing.

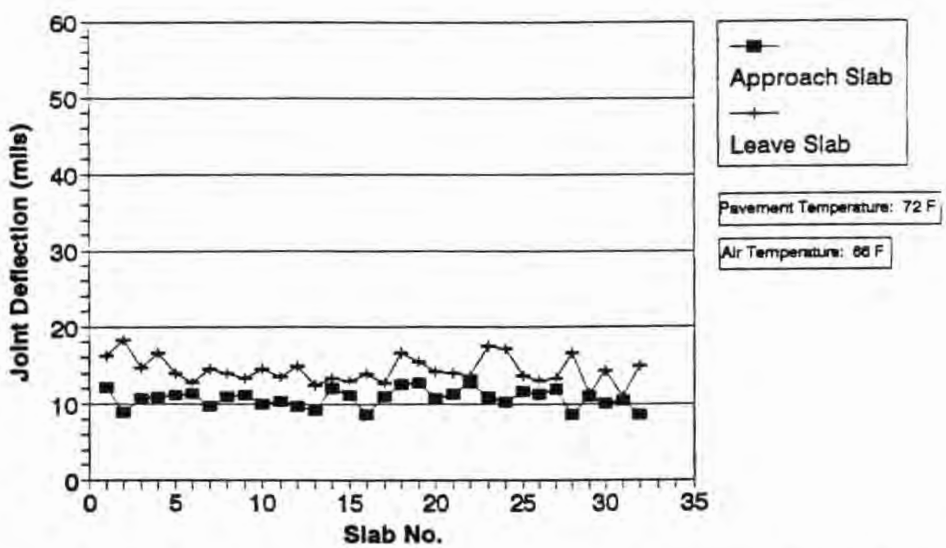


Figure 5.49. Deflection Profile for US12, MRM 275.03 + 100, WB, 11 Months After Undersealing.

Figure 5.50 shows a comparison of joint efficiency values during both periods of testing. An average LTE value of 53% was obtained before undersealing. The average LTE was 74.7%, 11 months later. A 41% increase in LTE occurred after 11 months.

Figures 5.51 through 5.58 indicate that voids with an average depth of 0.002 inch are present beneath the pavement, 11 months after undersealing. The average depth of voids was 0.006 inch prior to undersealing in 1992.

5.4.4. MRM 279

This subsection was only tested prior to undersealing in 1992. No visual survey of the pavement sections is available. The temperature encountered during testing was 85°F which is relatively high for load transfer efficiency and deflection analysis. The load versus deflection plots shown in Figures 5.59 through 5.61. The data indicates that voids with an average depth of 0.0022 inch are present beneath the pavement.

5.4.5 Statistical Analysis

The statistical analysis indicate that prior to undersealing, US12, MRM 275.03 + 100, WB had the highest mean deflection and the lowest mean joint efficiency. Both values were dramatically improved 11 months after undersealing. US12, MRM 272.17, EB has the highest mean load transfer efficiency values and they are not significantly different (3 days and 11 months after undersealing). The mean joint efficiency values for US12, MRM 268.4, WB are significantly different (28 days and 11 months after undersealing data). All the undersealed subsections

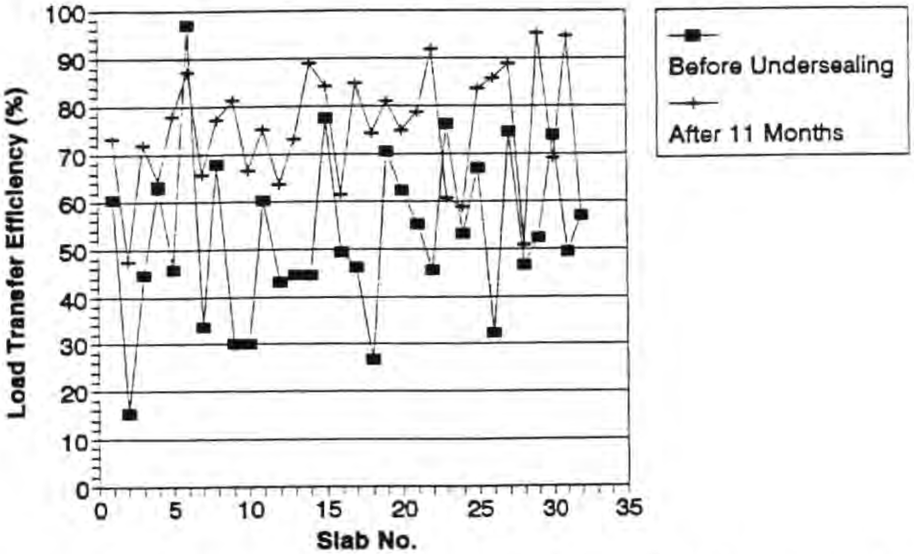


Figure 5.50. Comparison of Load Transfer Efficiency Values for US12, MRM 275.03 + 100, WB, Prior to and 11 Months After Undersealing.

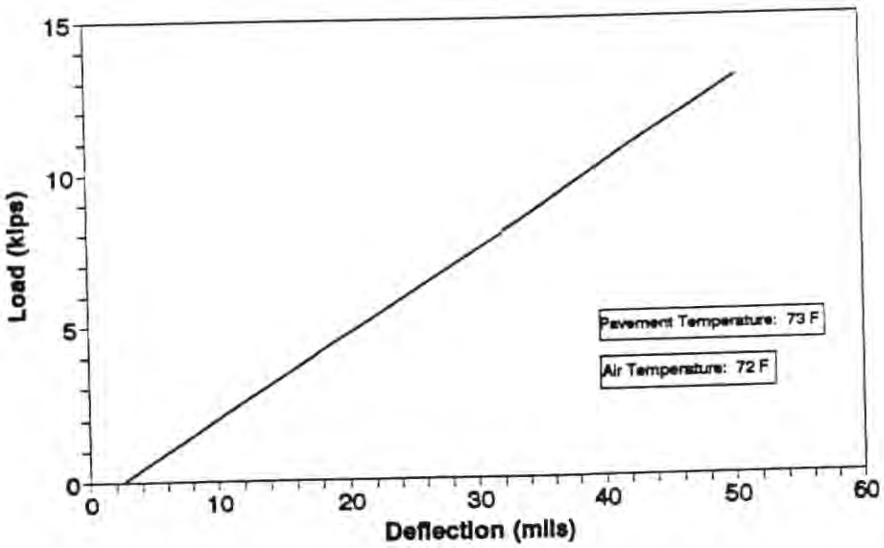


Figure 5.51. Load Versus Deflection for Slab 2, US12, MRM 275.03 + 100, WB, Prior to Undersealing.

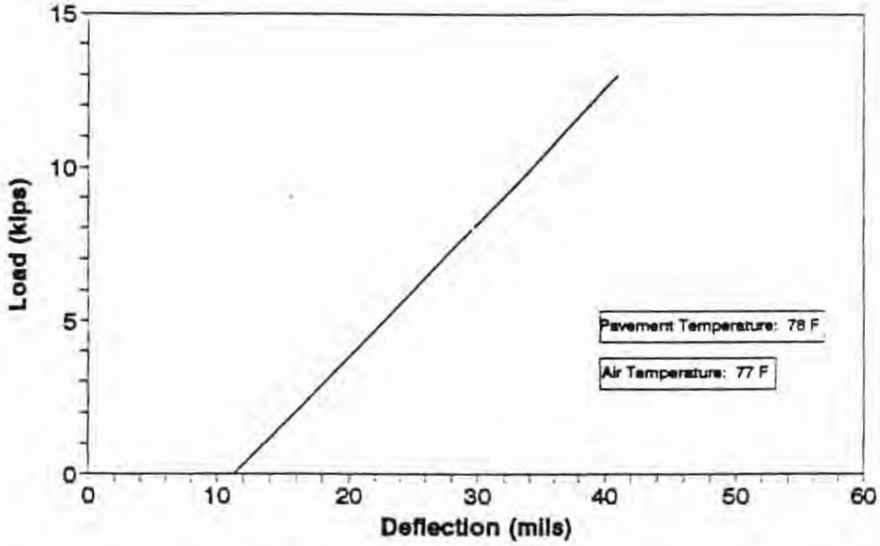


Figure 5.52. Load Versus Deflection for Slab 18, US12, MRM 275.03 + 100, WB, Prior to Undersealing.

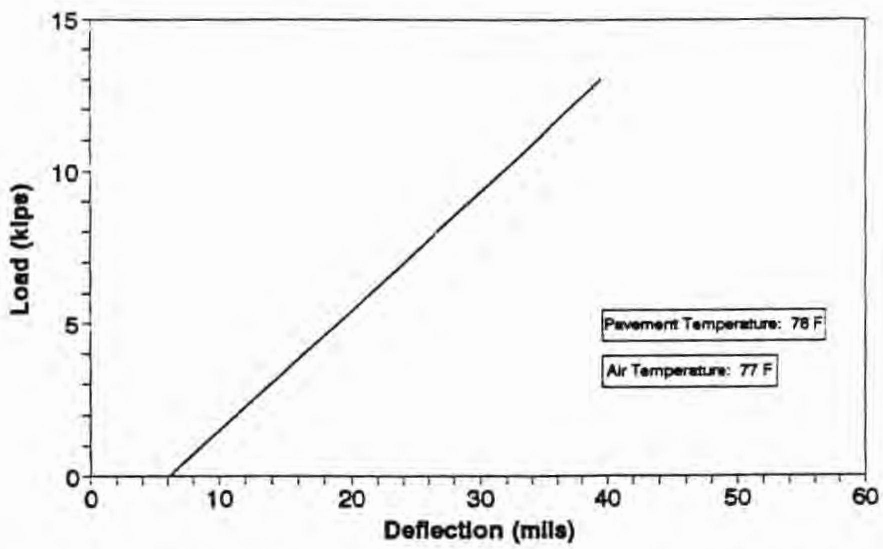


Figure 5.53. Load Versus Deflection for Slab 21, US12, MRM 275.03 + 100, WB, Prior to Undersealing.

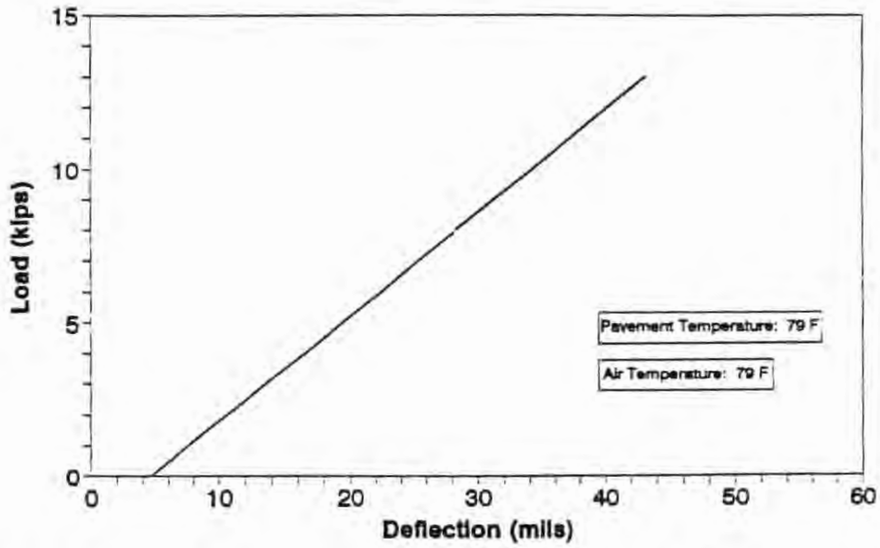


Figure 5.54. Load Versus Deflection for Slab 26, US12, MRM 275.03 + 100, WB, Prior to Undersealing.

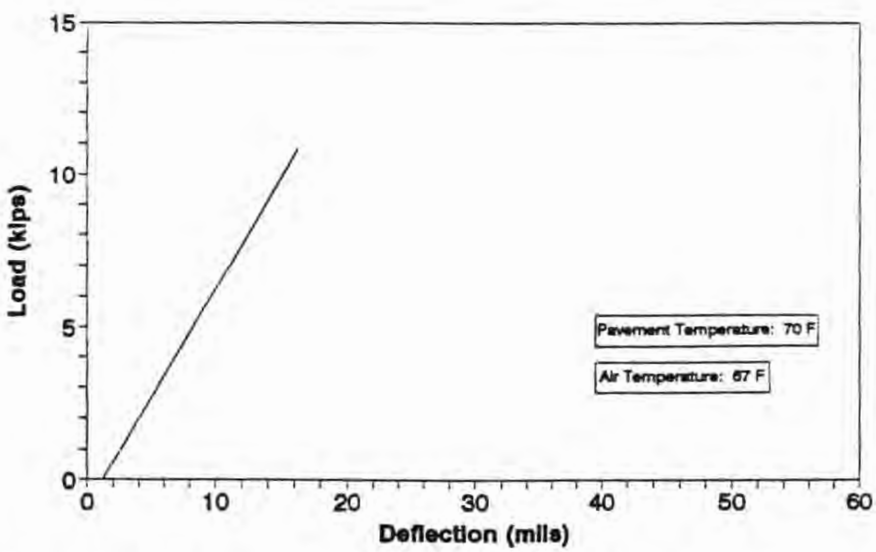


Figure 5.55. Load Versus Deflection for Slab 1, US12, MRM 275.03 + 100, WB, 11 Months After Undersealing.

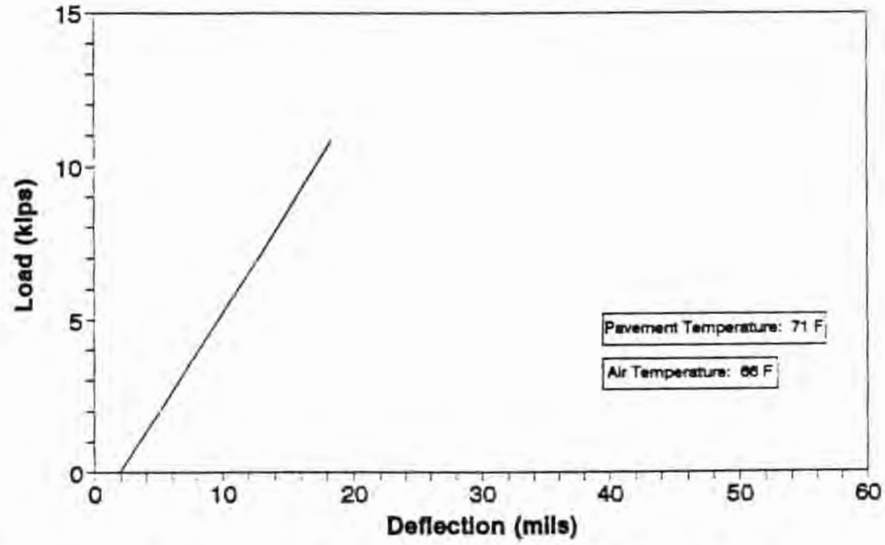


Figure 5.56. Load Versus Deflection for Slab 2, US12, MRM 275.03 + 100, WB, 11 Months After Undersealing.

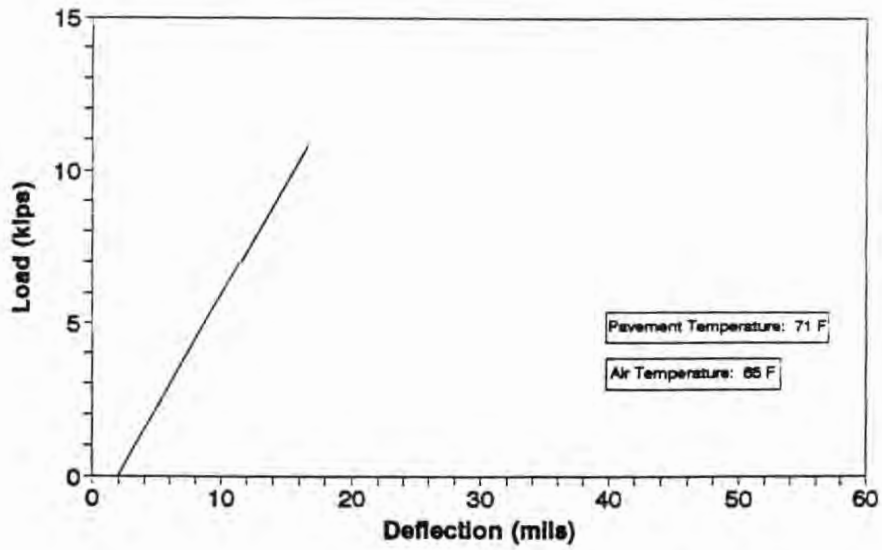


Figure 5.57. Load Versus Deflection for Slab 4, US12, MRM 275.03 + 100, WB, 11 Months After Undersealing.

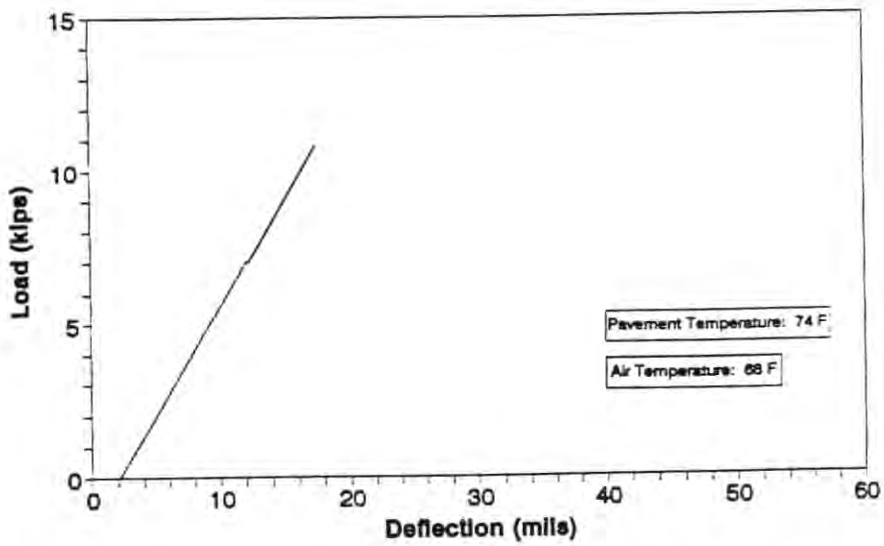


Figure 5.58. Load Versus Deflection for Slab 23, US12, MRM 275.03 + 100, WB, 11 Months After Undersealing.

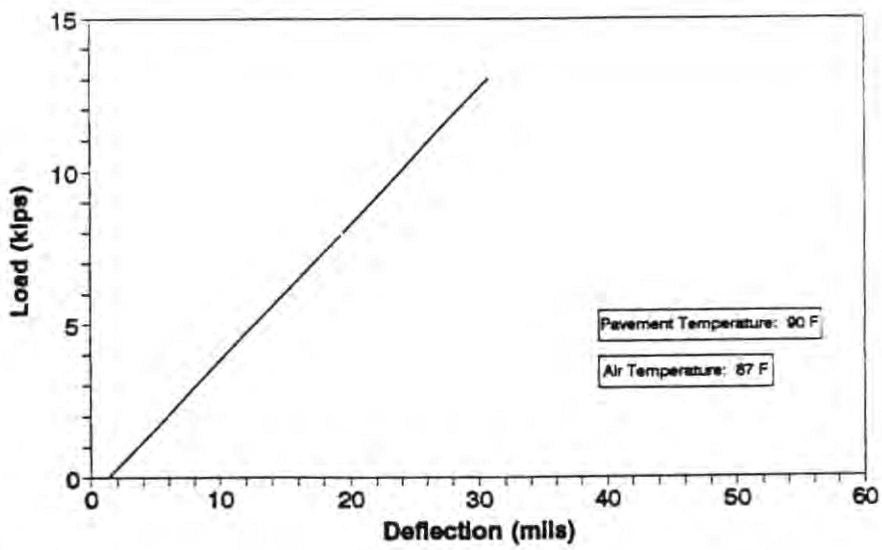


Figure 5.59. Load Versus Deflection Prior to Undersealing for US12, MRM 279, Slab 2.

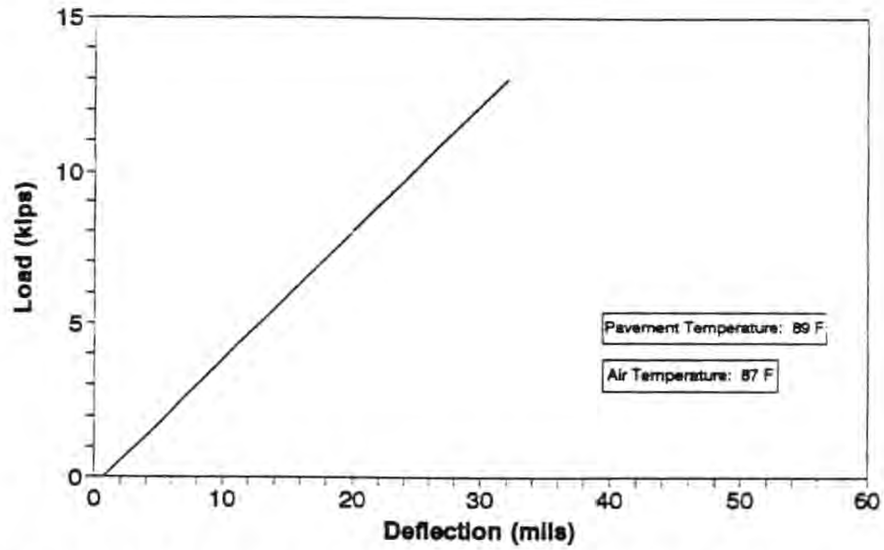


Figure 5.60. Load Versus Deflection Prior to Undersealing for US12, MRM 279, Slab 6.

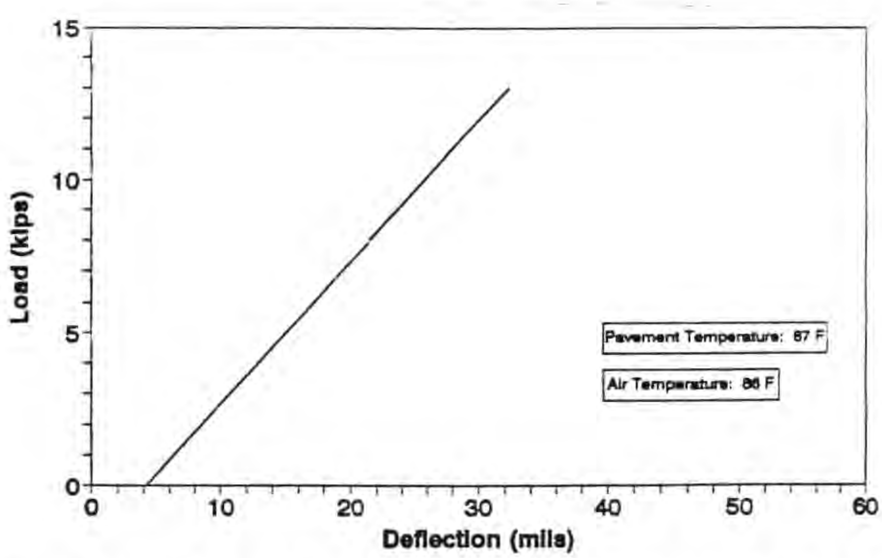


Figure 5.61. Load Versus Deflection Prior to Undersealing for US12, MRM 279, Slab 11.

in this project seems to perform well 11 months after undersealing was completed. The Duncan grouping statistical analysis for the Ipswich project are presented in Tables 5.6 and 5.7.

5.4.6. Comparison of Locked and Unlocked Conditions

Figures 5.62 through 5.66 show comparisons of load transfer efficiency in both the locked and unlocked conditions. In all cases, the load transfer efficiency values of the locked condition were always greater than that of the unlocked condition. Figures 5.67 through 5.71 show comparisons of leave slab deflections in both the locked and unlocked conditions. In all cases, the leave slab deflections of the locked condition were always less than that of unlocked condition.

5.5. SD50 in Yankton County

Rehabilitation work on this project started in June 1993. Prior to undersealing, three subsections were tested and evaluated in April, 1993. The starting locations for these subsections are as follows:

1. MRM 389 east bound.
2. MRM 390 west bound.
3. MRM 393 + 2700 ft west bound.

Undersealing was completed in July 1993 on this project. After undersealing, the same sections were tested in September 1993.

5.5.1 MRM 389 East Bound

FWD testing and visual surveys were conducted on April 26,

Table 5.6. Duncan Grouping For Load Transfer Efficiency Values (US12, Ipswich).

Duncan Grouping		Mean LTE (%)	No. of Slabs	Subsection
	A	91.4	26	272.17 (3 Days)
B	A	85.7	32	272.17 (11 Months)
B	C	81.6	32	268.4 (28 Days)
	C	74.7	32	275.03 (11 Months)
	D	62.7	32	268.4 (11 Months)
	E	53.0	32	275.03 (Before)

Table 5.7. Duncan Grouping For FWD Deflections (US12, Ipswich).

Duncan Grouping		Mean Deflection (inch)	No. of Slabs	Subsection
	A	0.03119	32	275.03 (Before)
	B	0.01721	26	272.17 (3 Days)
	B	0.01704	32	268.4 (11 Months)
C	B	0.01624	32	268.4 (28 Days)
C	B	0.01521	32	272.17 (11 Months)
C		0.01430	32	275.03 (11 Months)

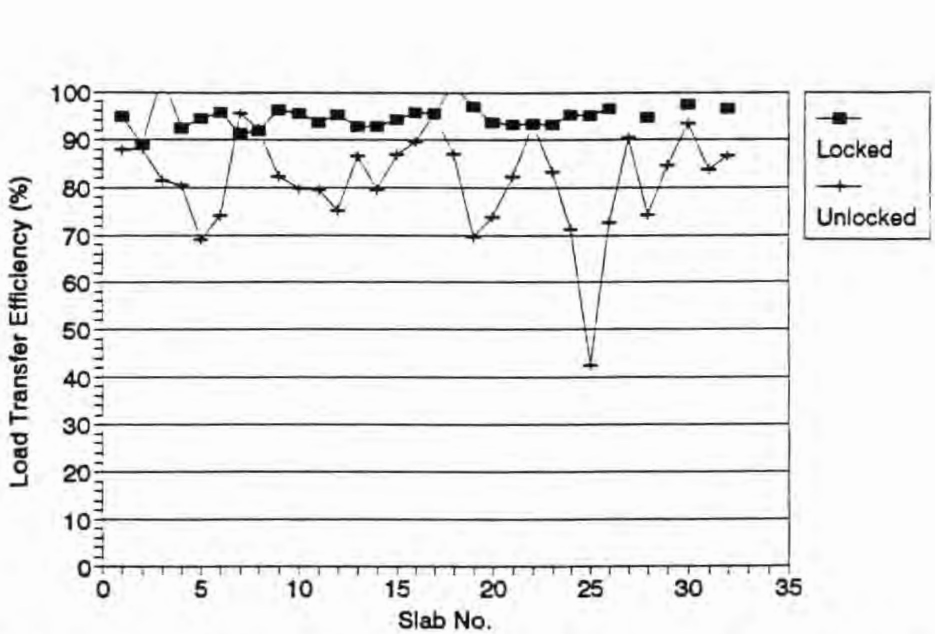


Figure 5.62. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in US12, MRM 268.4, WB, 28 Days After Undersealing.

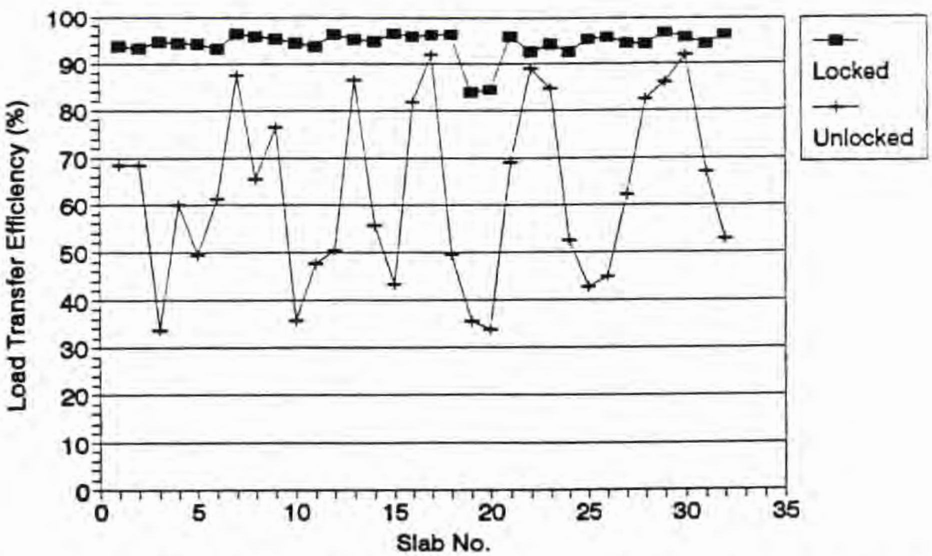


Figure 5.63. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in US12, MRM 268.4, WB, 11 Months After Undersealing.

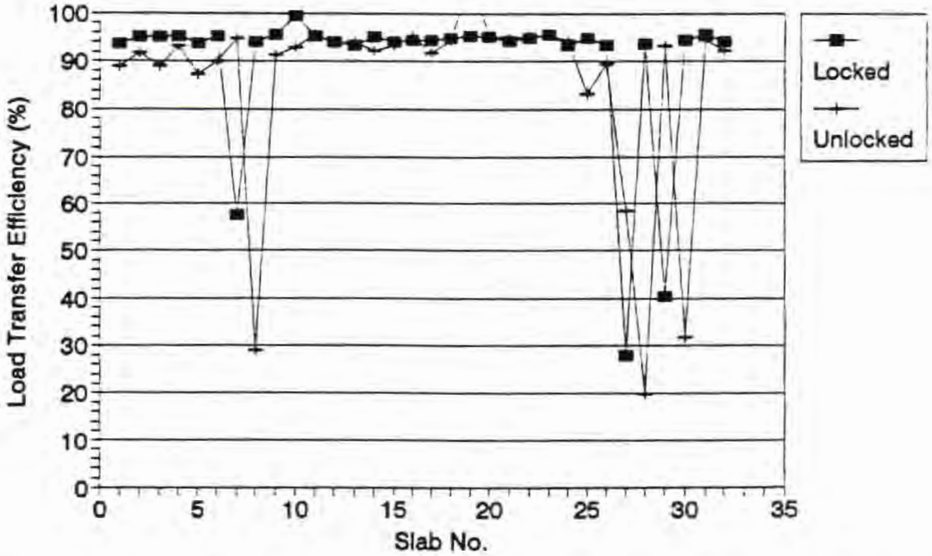


Figure 5.64. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in US12, MRM 272.17, EB, 11 Months After Undersealing.

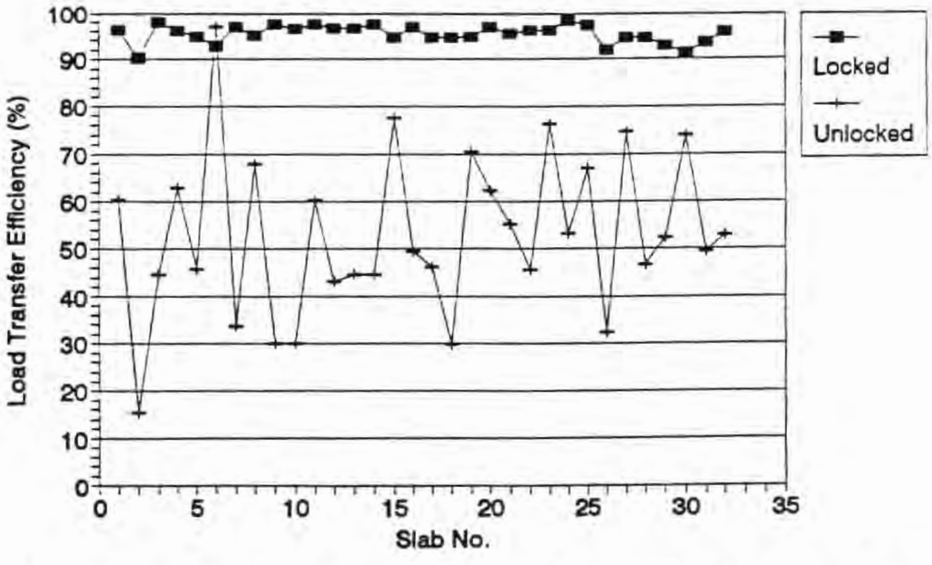


Figure 5.65. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in US12, MRM 275.03 + 100, WB, Prior to Undersealing.

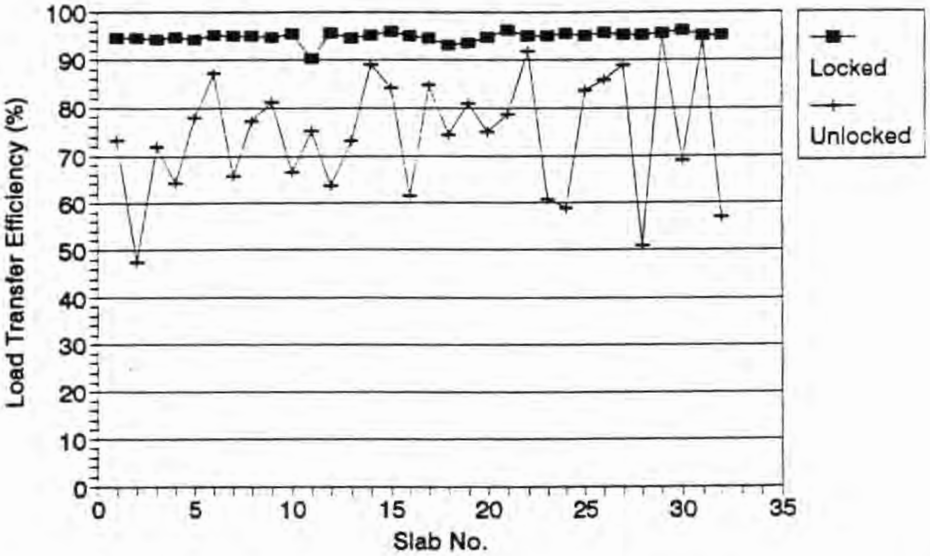


Figure 5.66. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in US12, MRM 275.03 + 100, WB, 11 Months After Undersealing.

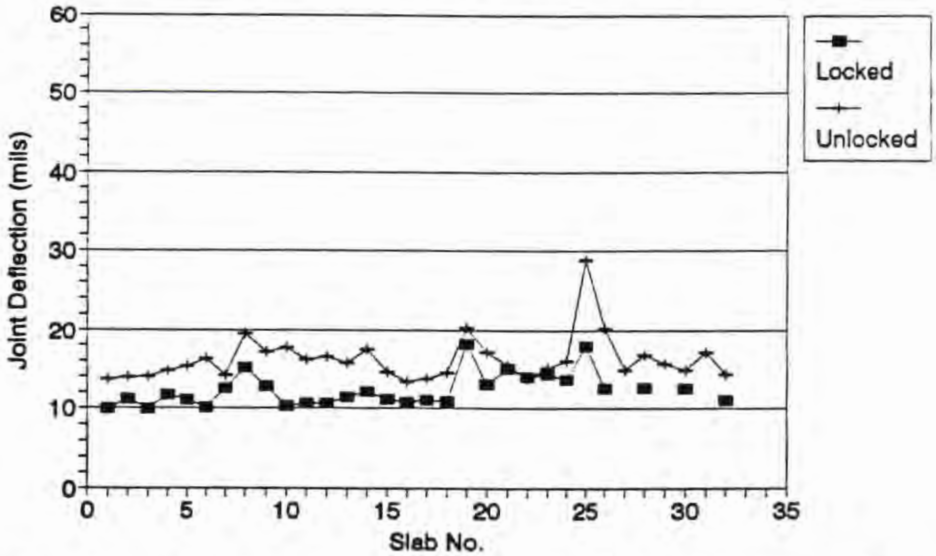


Figure 5.67. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in US12, MRM 268.4, WB, 28 Days After Undersealing.

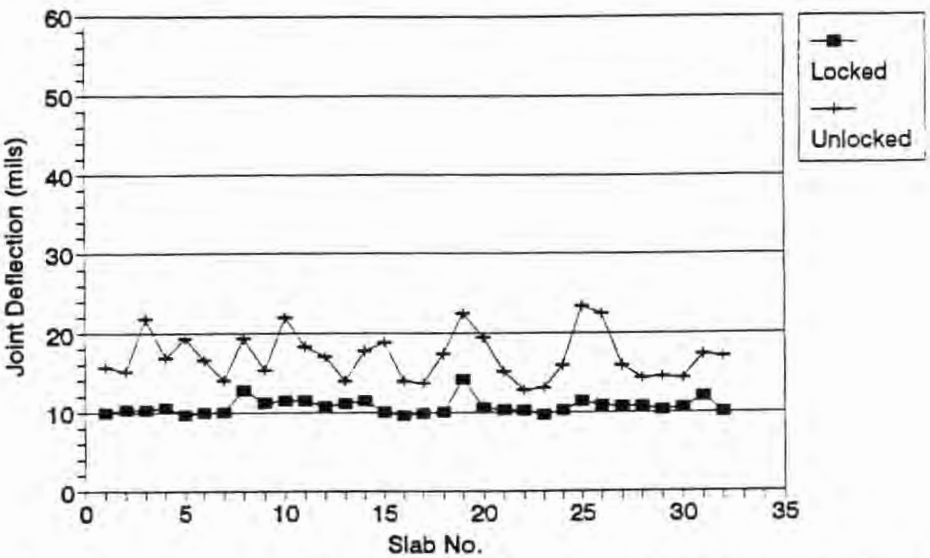


Figure 5.68. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in US12, MRM 268.4, WB, 11 Months After Undersealing.

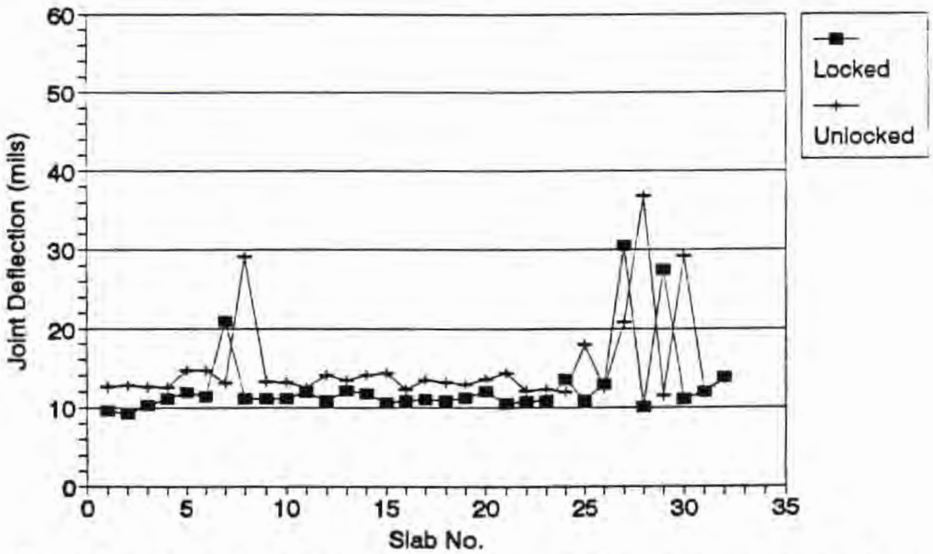


Figure 5.69. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in US12, MRM 272.17, EB, 11 Months After Undersealing.

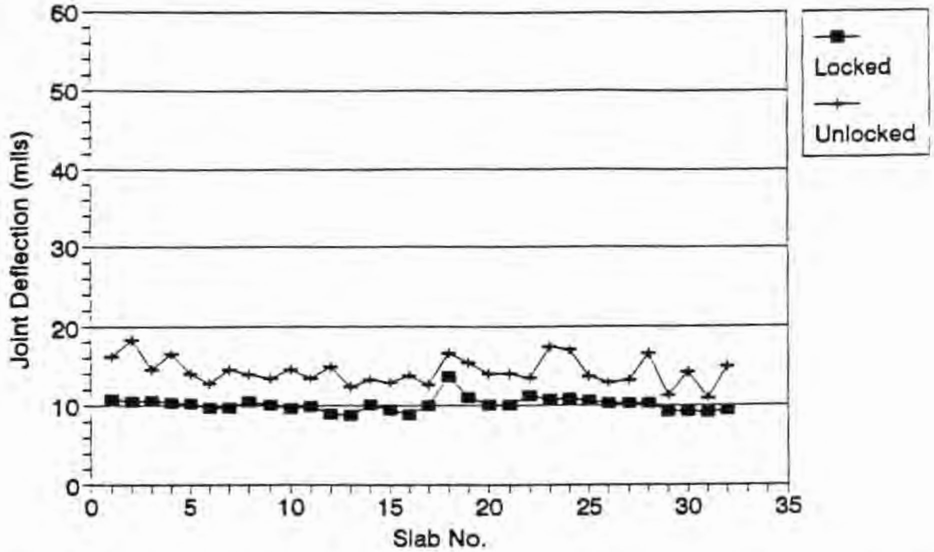


Figure 5.70. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in US12, MRM 275.03 + 100, WB, 11 Months After Undersealing.

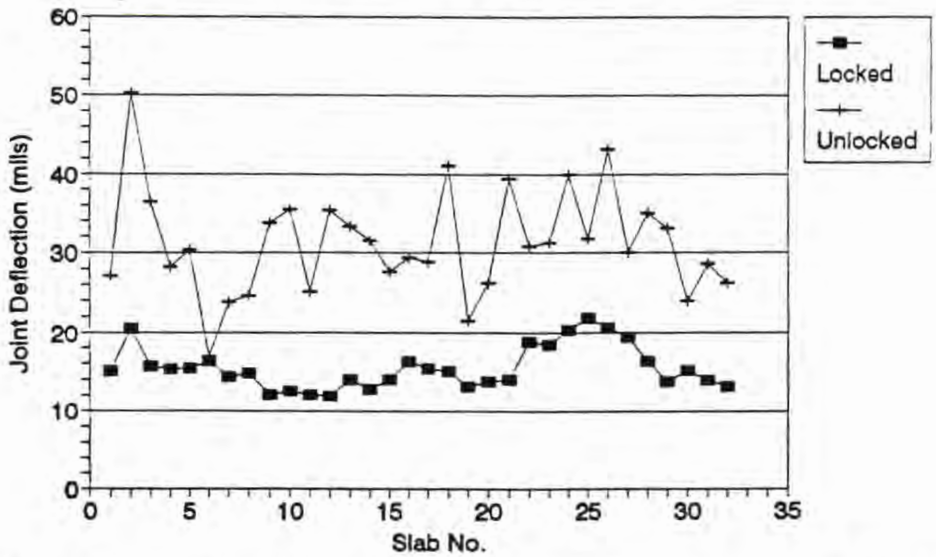


Figure 5.71. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in US12, MRM 275.03 + 100, WB, Prior To Undersealing.

1993. The average pavement surface temperature was 67°F. An average faulting of 0.2625 inch was measured in SD50, MRM 389, EB. Figure 5.72 shows the extent of faulting in this subsection. No cracks or pumping were observed in the pavement section. Figures 5.73 and 5.74 show representative deflection and load transfer efficiency profiles obtained for SD50, MRM 389, EB. The average leave slab deflection was 0.0383 inch and the average joint efficiency was about 25%. Figures 5.75 through 5.78 indicate that the average depth of the voids beneath the pavement is about 0.003 inch.

This section was tested after undersealing on September 21, 1993. The average pavement temperature was 62°F. The SDDOT tested only the first six slabs of this subsection, compared to

Figure 5.72. High Severity Faulting Observed in SD50, MRM 389, EB.

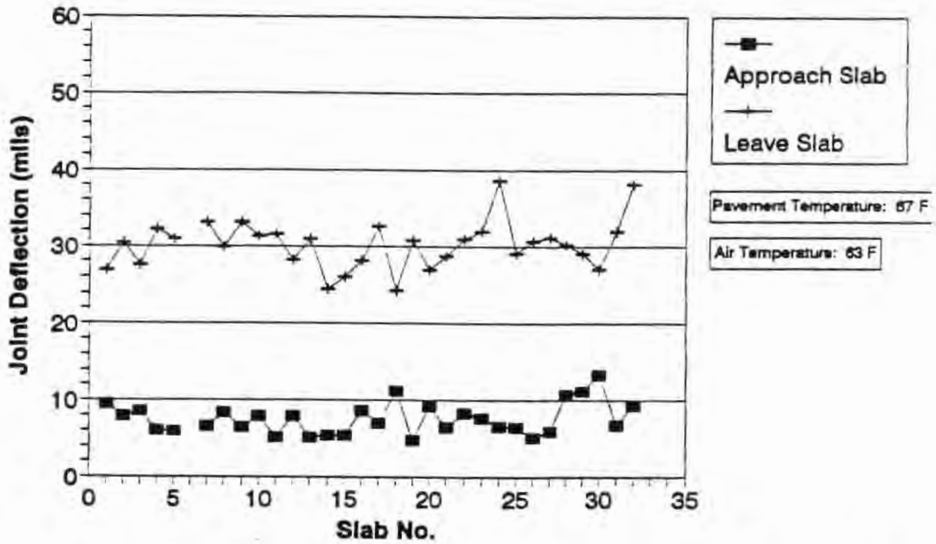


Figure 5.73. Deflection Profile for SD50, MRM 389, EB, Prior to Undersealing.

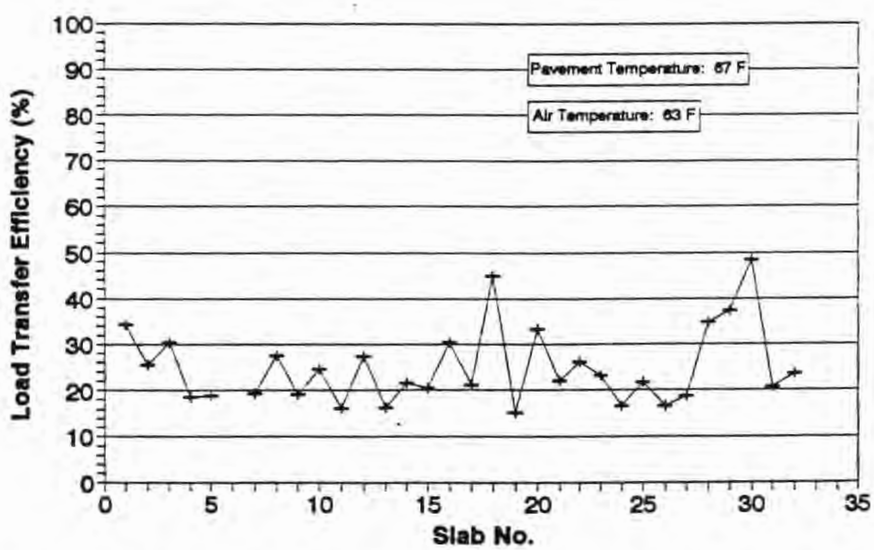


Figure 5.74. Load Transfer Efficiency Profile for SD50, MRM 389, EB, Prior to Undersealing.

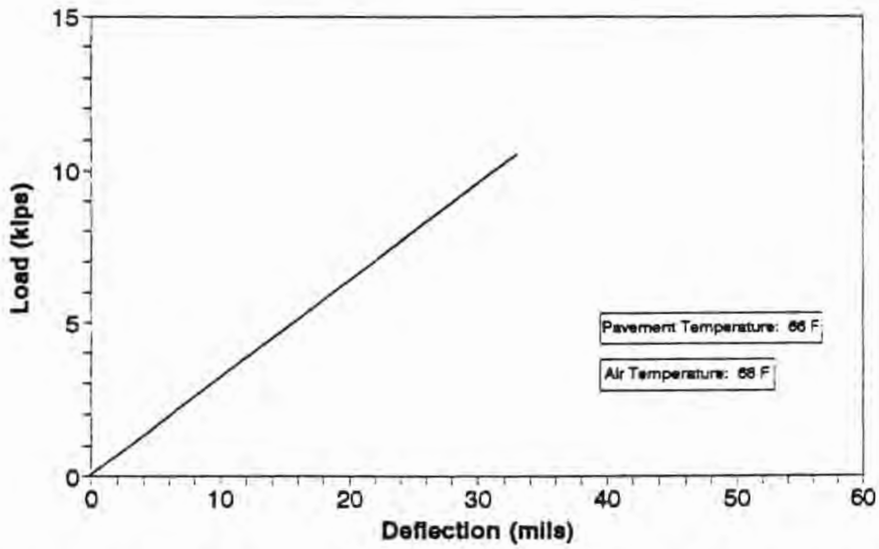


Figure 5.75. Load Versus Deflection for Slab 6, SD50, MRM 389, EB, Prior to Undersealing.

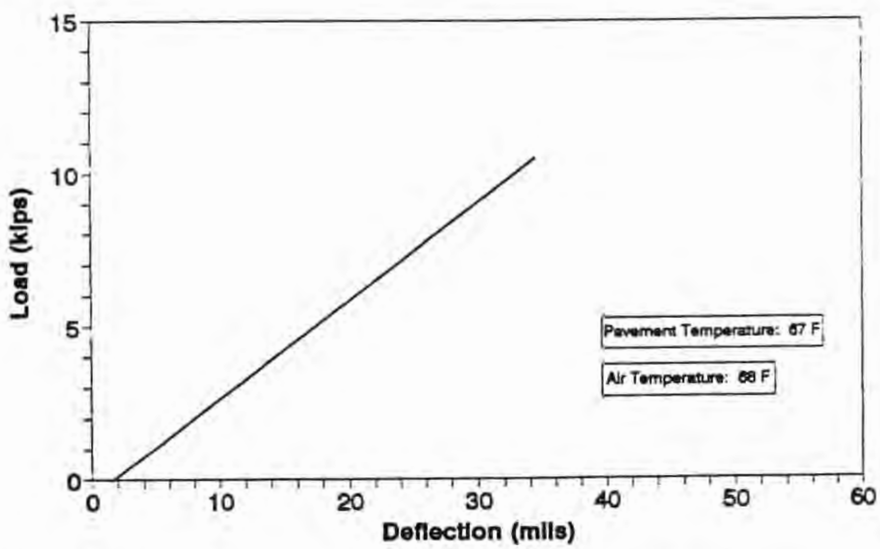


Figure 5.76. Load Versus Deflection for Slab 24, SD50, MRM 389, EB, Prior to Undersealing.

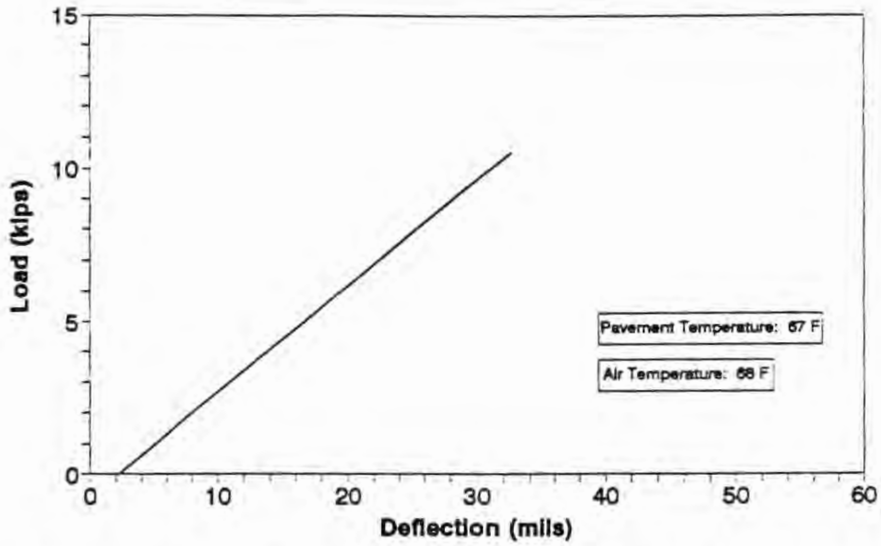


Figure 5.77. Load Versus Deflection for Slab 26, SD50, MRM 389, EB, Prior to Undersealing.

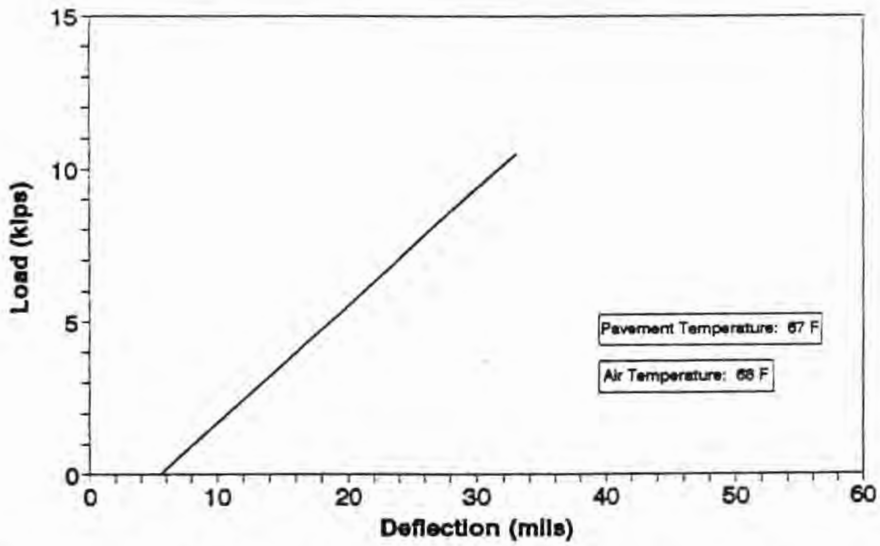


Figure 5.78. Load Versus Deflection for Slab 32, SD50, MRM 389, EB, Prior to Undersealing.

32 slabs that were tested on this subsection before undersealing. Due to the difference in the number of observations before and after undersealing, this subsection will not be included in the statistical analysis. The average leave slab deflection was 0.0133 inch, and the average joint efficiency was about 75%. These values represent a 65% reduction in deflection and a 200% increase in load transfer. Due to the limited number of readings obtained after undersealing, these numbers may not accurately represent what happened over the rest of the subsection. Figures 5.79 and 5.80 show the deflection and load transfer profiles for this subsection after undersealing. Figure 5.81 shows the comparison of the load transfer efficiency values of this section before and after undersealing.

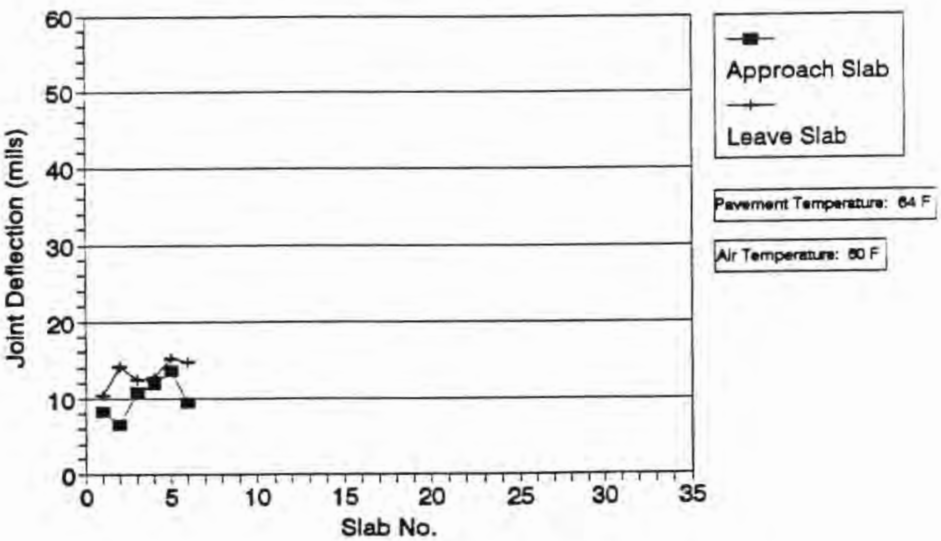


Figure 5.79. Deflection Profile for SD50, MRM 389, EB After Undersealing.

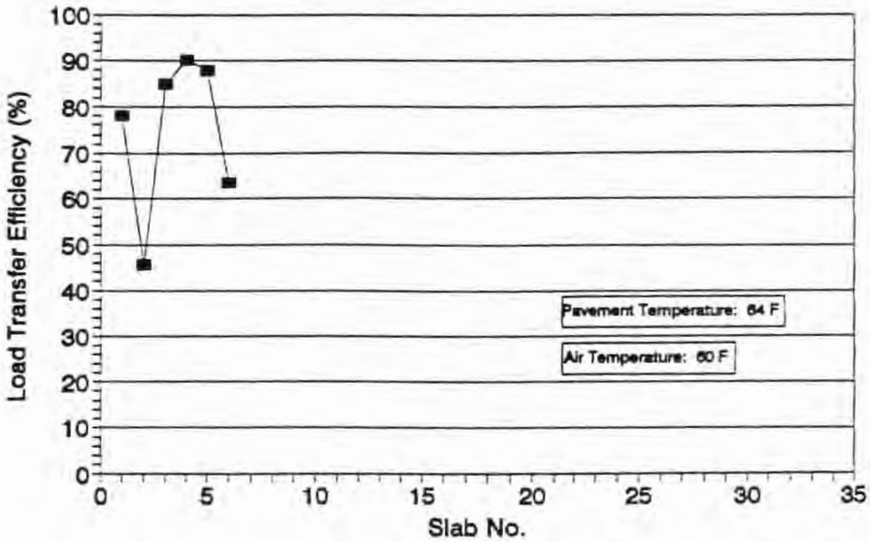


Figure 5.80. Load Transfer Efficiency Profile for SD50, MRM 389, EB, After Undersealing.

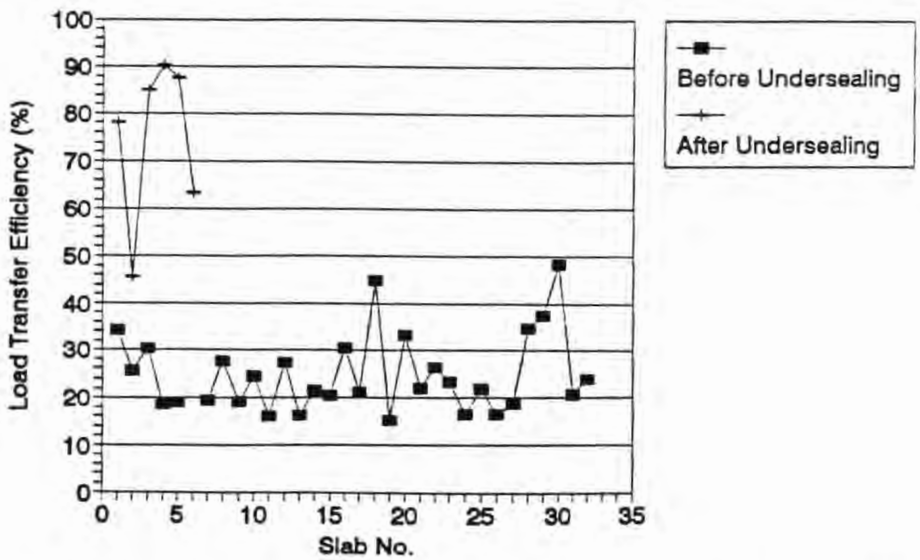


Figure 5.81. Load Transfer Efficiency Comparison for SD50, MRM 389, EB.

5.5.2 MRM 390 West Bound

This subsection was tested on April 27, 1993 and was tested again the next day to acquire the locked-up deflection data. An average faulting of 0.125 inch was measured in SD50, MRM 390, WB. Prior to undersealing, the average leave slab deflection was 0.027 inch and the average load transfer efficiency was 37.8%. The average void depth below SD50, MRM 390, WB was about 0.0043 inch. The deflection and load transfer efficiency profiles are shown in Figures 5.82 and 5.83, respectively. The Load versus deflection plots are depicted in Figures 5.84 through 5.87.

This section was tested after undersealing on September 22, 1993. The average pavement temperature was 65°F. Testing was done only on the unlocked condition. The average leave slab

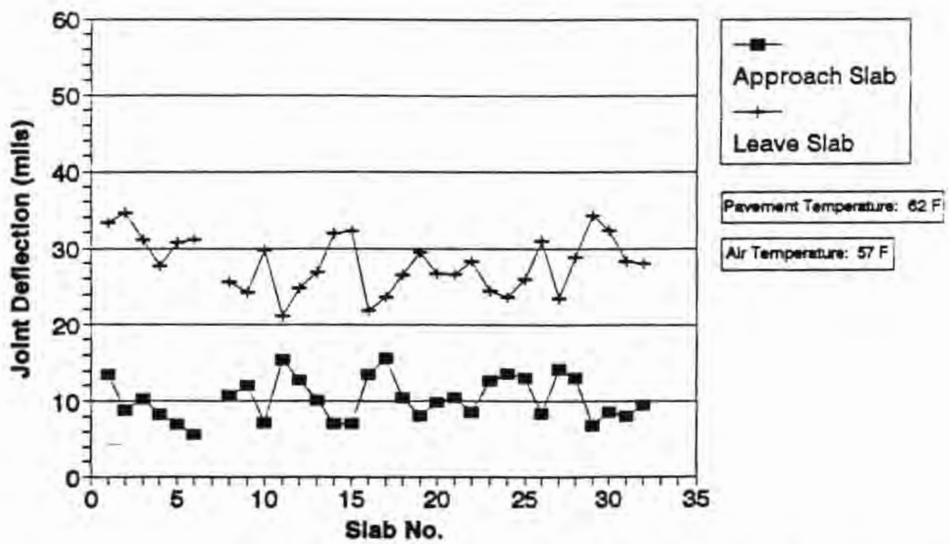


Figure 5.82. Deflection Profile for SD50, MRM 390, WB, Prior to Undersealing.

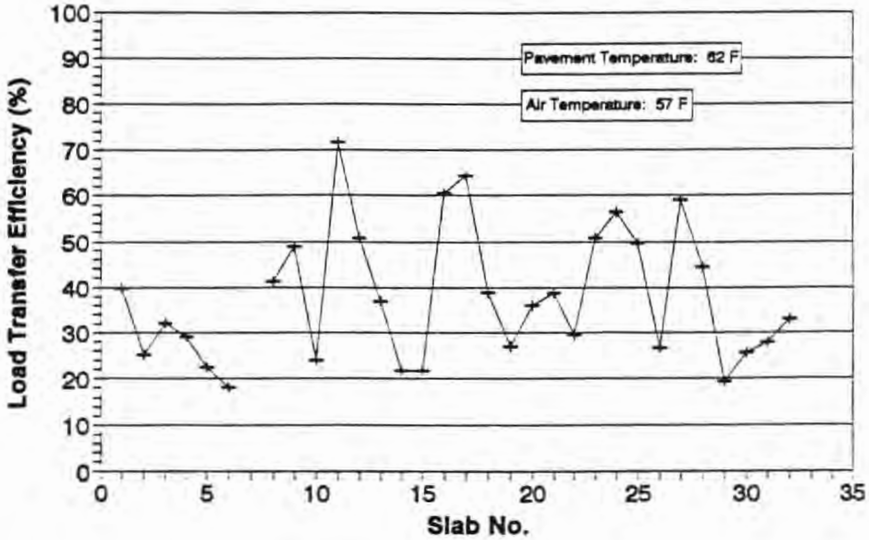


Figure 5.83. Load Transfer Efficiency for SD50, MRM 390, WB, Prior to Undersealing.

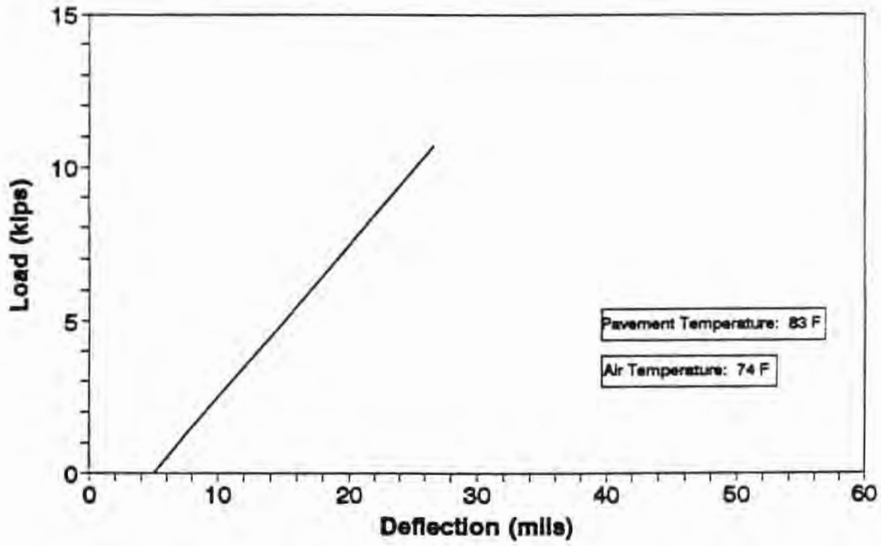


Figure 5.84. Load Versus Deflection for Slab 1, SD50, MRM 390, WB, Prior to Undersealing.

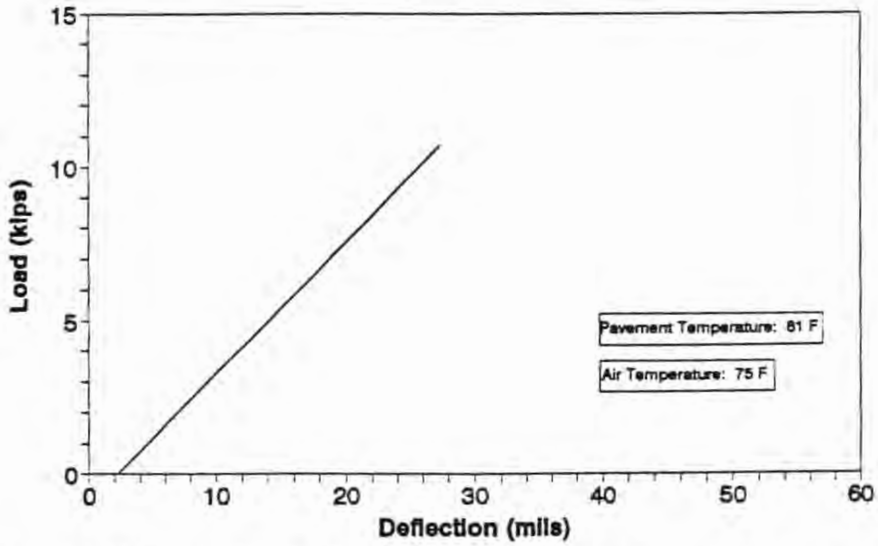


Figure 5.85. Load Versus Deflection for Slab 5, SD50, MRM 390, WB, Prior to Undersealing.

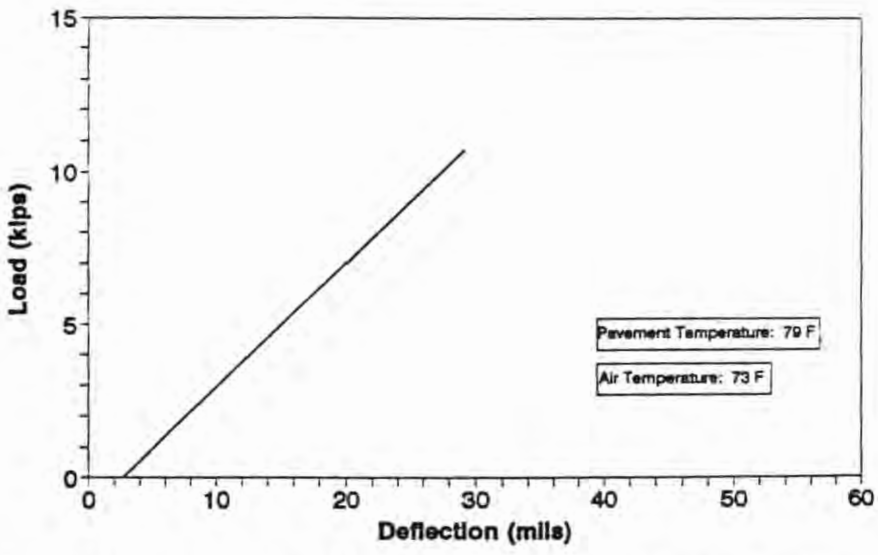


Figure 5.86. Load Versus Deflection for Slab 14, SD50, MRM 390, WB, Prior to Undersealing.

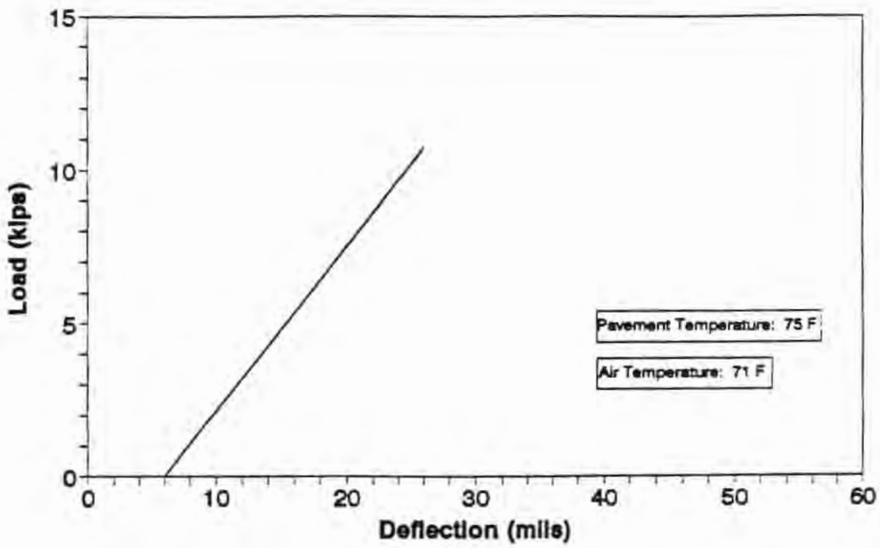


Figure 5.87. Load Versus Deflection for Slab 30, SD50, MRM 390, WB, Prior to Undersealing.

deflection was 0.0101 inch, and the average load transfer efficiency was 90.7%. These values represent a 62.6% decrease in deflection and a 140% increase in load transfer efficiency, respectively. The deflection profile for this section after undersealing is shown in Figure 5.88. Figures 5.89 and 5.90 show the load transfer profile and load transfer comparison for SD50, MRM 390, WB, respectively.

5.5.3 MRM 393 + 2700 ft

Testing on SD50, MRM 393 + 2700', WB was conducted in the morning and the afternoon of April 28, 1993. The average faulting in the subsection was about 0.15625 inch. Figures 5.91 and 5.92 show the deflection and the joint efficiency profiles for SD50, MRM 393 + 2700', WB. The average leave slab deflection

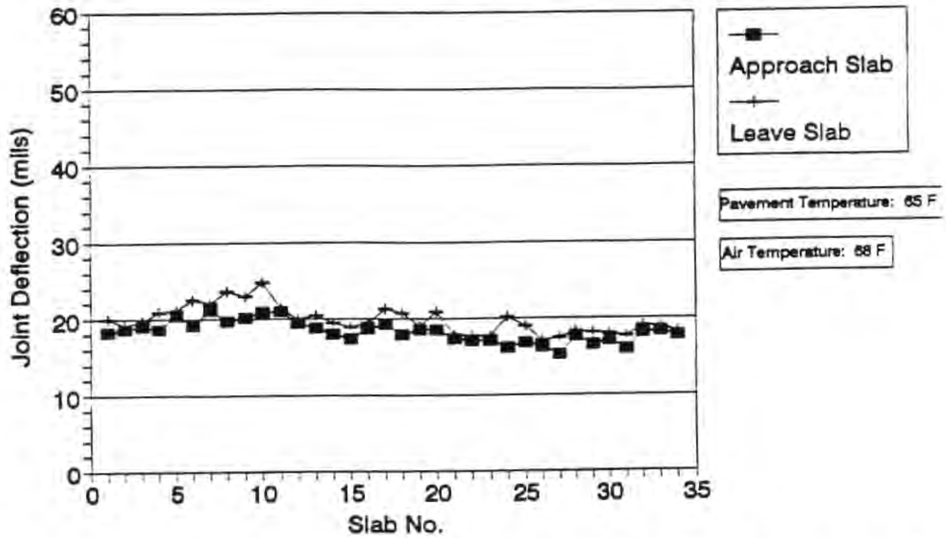


Figure 5.88. Deflection Profile for SD50, MRM 390, WB, After Undersealing.

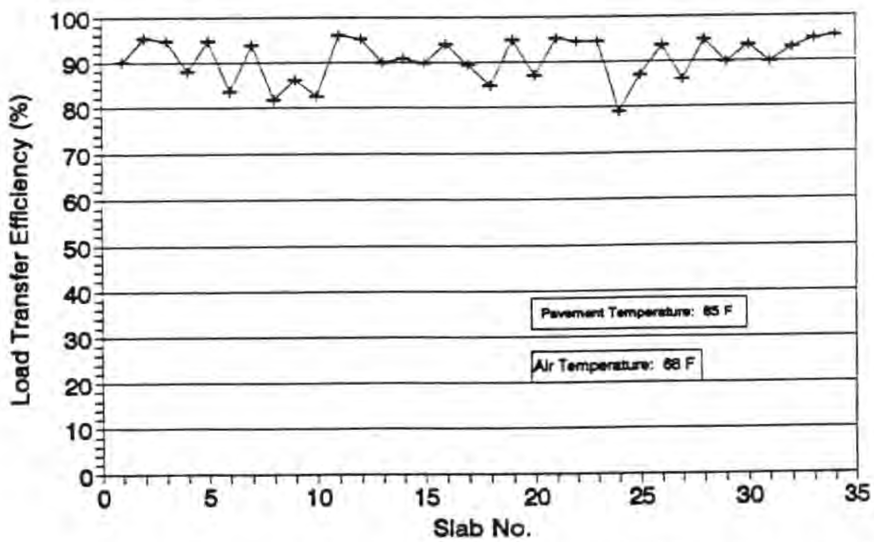


Figure 5.89. Load Transfer Efficiency Profile for SD50, MRM 390, WB, After Undersealing.

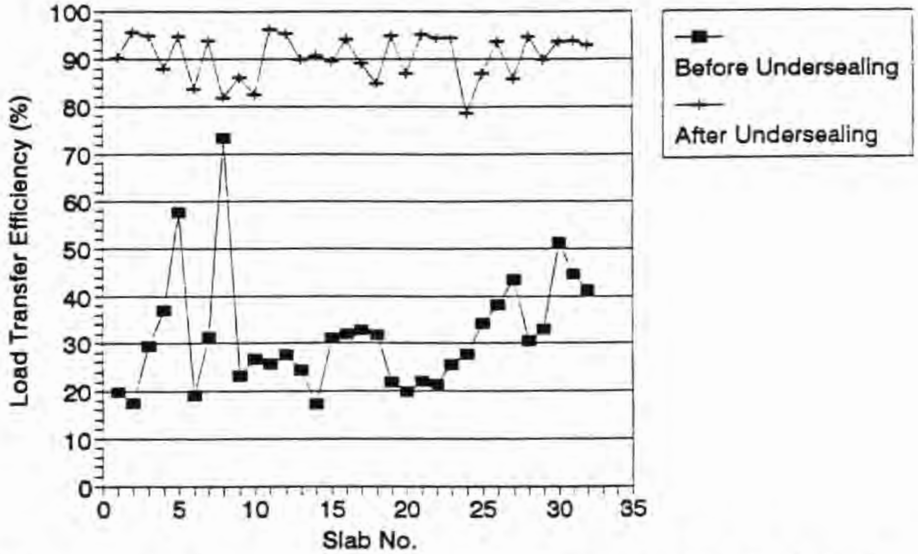


Figure 5.90. Load Transfer Efficiency Comparison for SD50, MRM 390, WB.

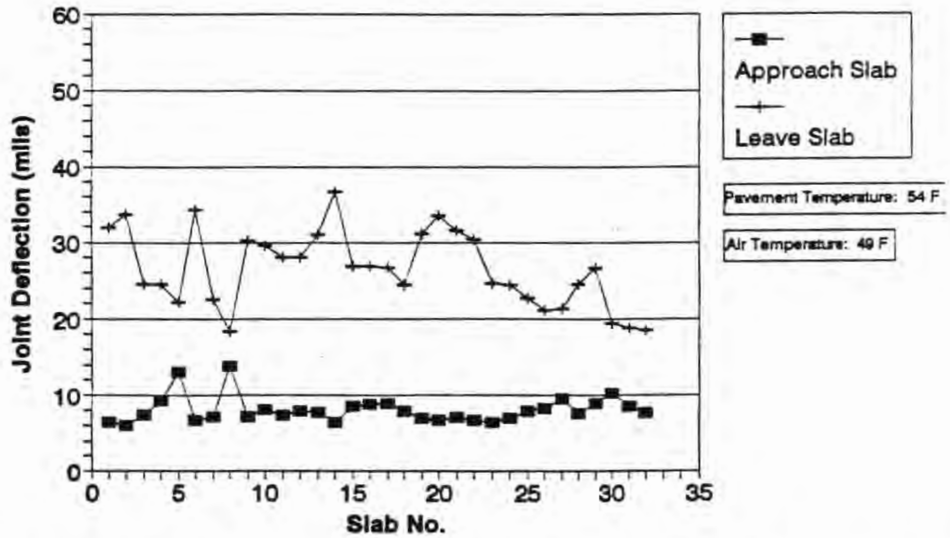


Figure 5.91. Deflection Profile for SD50, MRM 393 + 2700', WB, Prior to Undersealing.

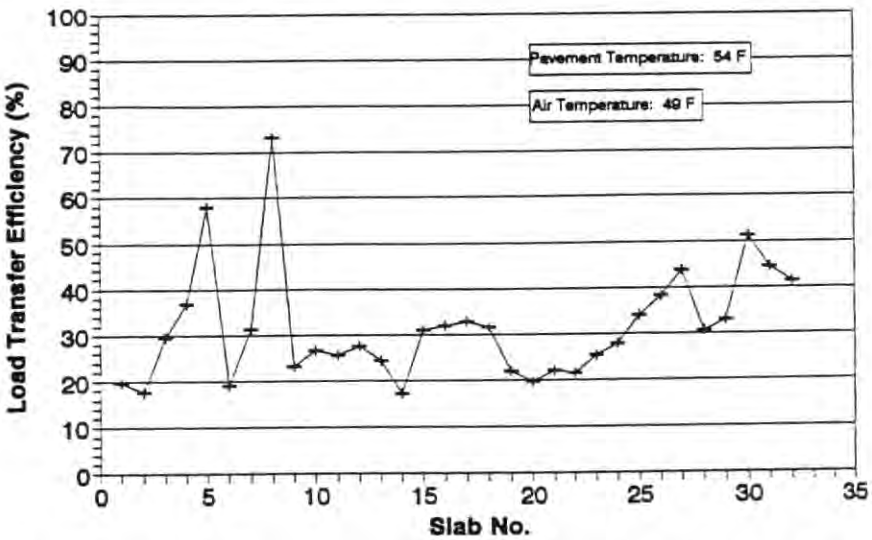


Figure 5.92. Load Transfer Efficiency for SD50, MRM 393 + 2700', WB, Prior to Undersealing.

was 0.0205 inch and the average joint efficiency was 31.7%. Figures 5.93 through 5.95 show the load versus deflection plots. The average void depth beneath the pavement is about 0.003 inch.

This subsection was tested after undersealing on September 22, 1993. The average pavement temperature was 64°F. Testing was only done on the unlocked case. The average leave slab deflection was 0.0171 inch, and the average load transfer efficiency was 62.6%. These values represent a 16.9% reduction in deflection and a 97% increase in load transfer efficiency. Figures 5.96 and 5.97 show the deflection and load transfer efficiency profiles, respectively. Figure 5.98 shows the load transfer comparison before and after undersealing was completed on this subsection.

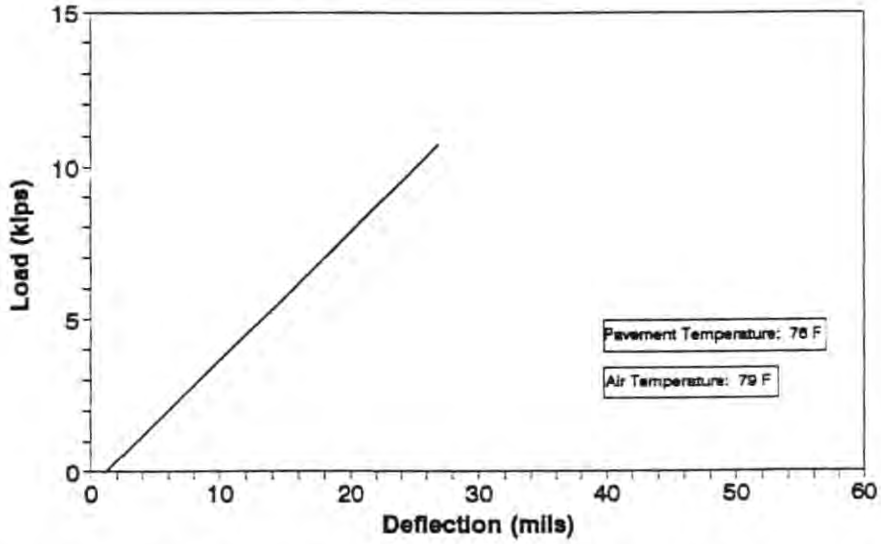


Figure 5.93. Load Versus Deflection for Slab 14, SD50, MRM 393 + 2700', WB, Prior to Undersealing.

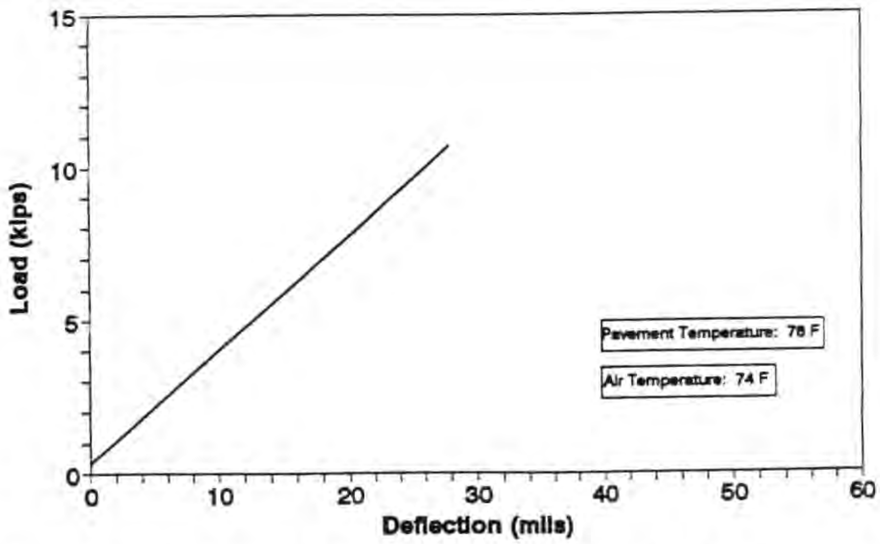


Figure 5.94. Load Versus Deflection for Slab 19, SD50, MRM 393 + 2700', WB, Prior to Undersealing.

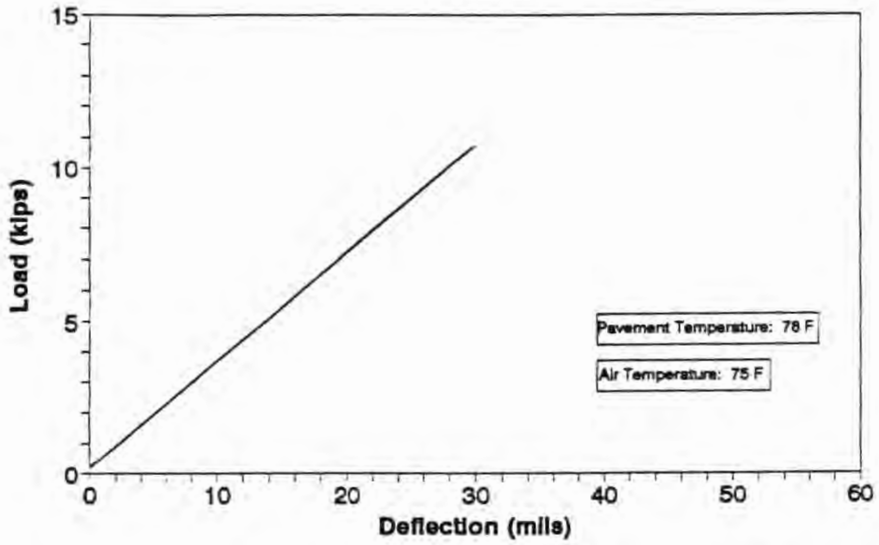


Figure 5.95. Load Versus Deflection for Slab 20, SD50, MRM 393 + 2700', WB, Prior to Undersealing.

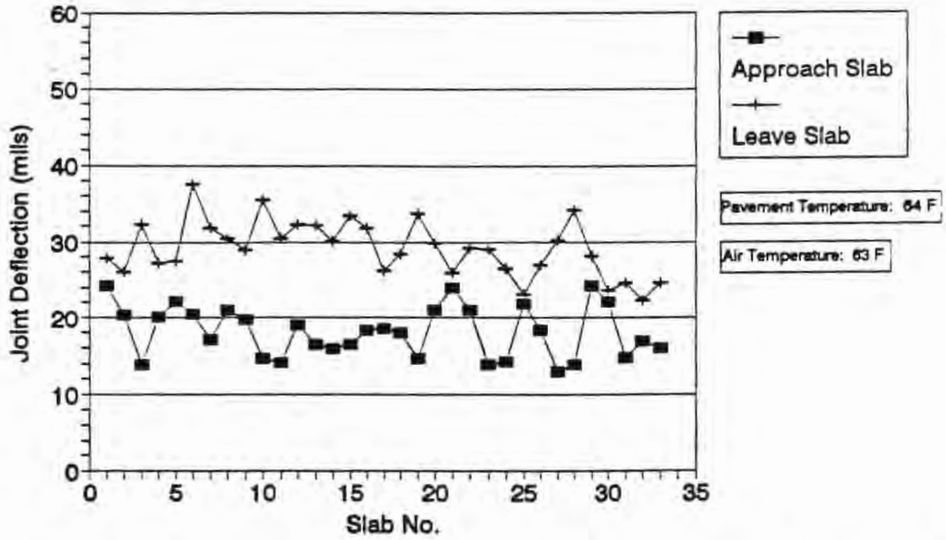


Figure 5.96. Deflection Profile for SD50, MRM 393 + 2700', WB, After Undersealing.

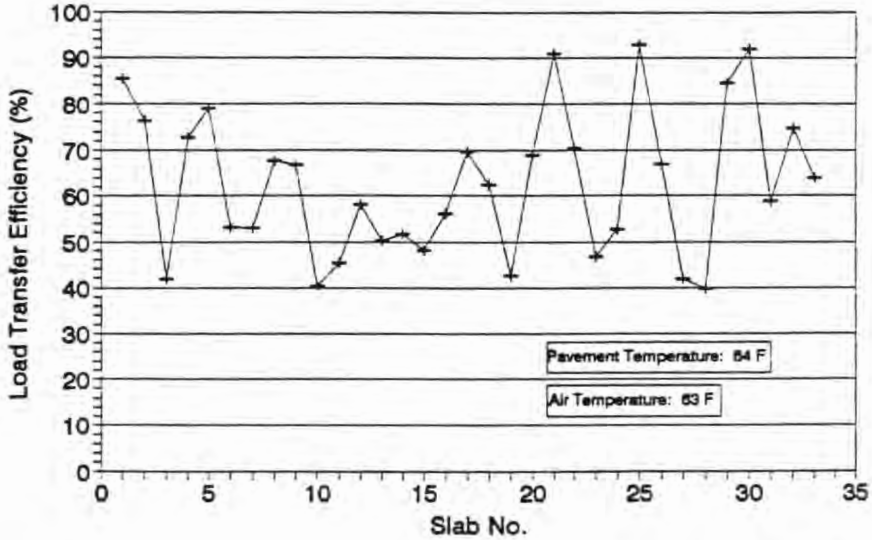


Figure 5.97. Load Transfer Efficiency Profile for SD50, MRM 393 + 2700', WB, After Undersealing.

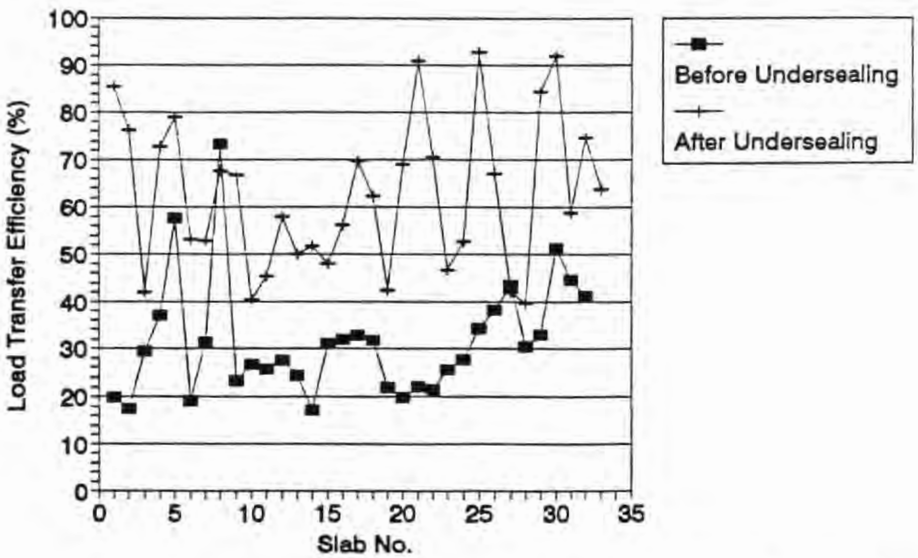


Figure 5.98. Load Transfer Efficiency Comparison for SD50, MRM 393 + 2700', WB.

5.5.4 Statistical Analysis

SD50, MRM 389, EB will not be included in this statistical analysis, due to the fact that there were not enough readings after undersealing to make a valid comparison. Tables 5.8 and 5.9 present the Duncan grouping for the project.

The undersealed sections have significantly lower deflections and higher load transfer efficiency values than the non-undersealed sections. However, there were significant differences between the two undersealed sections. SD50, MRM 390, WB has significantly lower deflections and higher load transfer efficiency values than SD50, MRM 393 + 2700', WB.

Table 5.8. Duncan Grouping for Load Transfer Efficiency Values (SD50).

Duncan Grouping	Mean LTE (%)	No. of Slabs	Subsection
A	90.7	34	390 WB AFTER
B	62.6	33	393 WB AFTER
C	37.8	31	390 WB BEFORE
D	31.7	32	393 WB BEFORE

Table 5.9. **Duncan Grouping For FWD Deflections (SD50).**

Duncan Grouping	Mean Deflection (inch)	No. of Slabs	Subsection
A	0.0278	32	390 WB BEFORE
B	0.0205	32	393 WB BEFORE
C	0.0171	33	393 WB AFTER
D	0.0101	34	390 WB AFTER

5.5.5. Comparison of Locked and Unlocked Conditions

Figures 5.99 and 5.100 show comparisons of load transfer efficiency in both the locked and unlocked conditions. In all cases, the load transfer efficiency values of the locked condition were always greater than that of the unlocked condition. Figures 5.101 and 5.102 show comparisons of leave slab deflections in both the locked and unlocked conditions. In all cases, the leave slab deflections of the locked condition were always less than that of unlocked condition.

5.6. Summary

The mean deflections and load transfer efficiency data for all four highways are provided in Table 5.10.

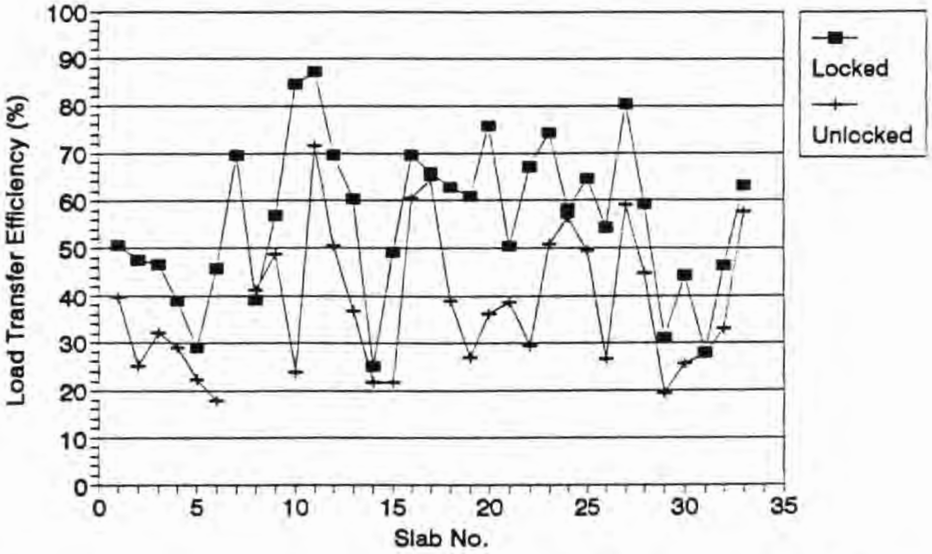


Figure 5.99. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in SD50, MRM 390, WB, Before Undersealing.

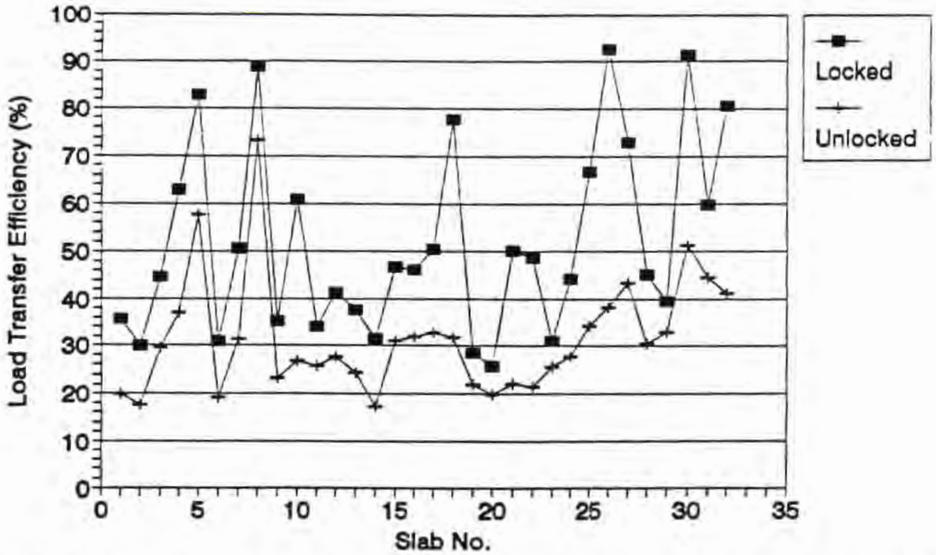


Figure 5.100. Comparison of Load Transfer Efficiency For Locked and Unlocked Conditions in SD50, MRM 393 + 2700', WB, Before Undersealing.

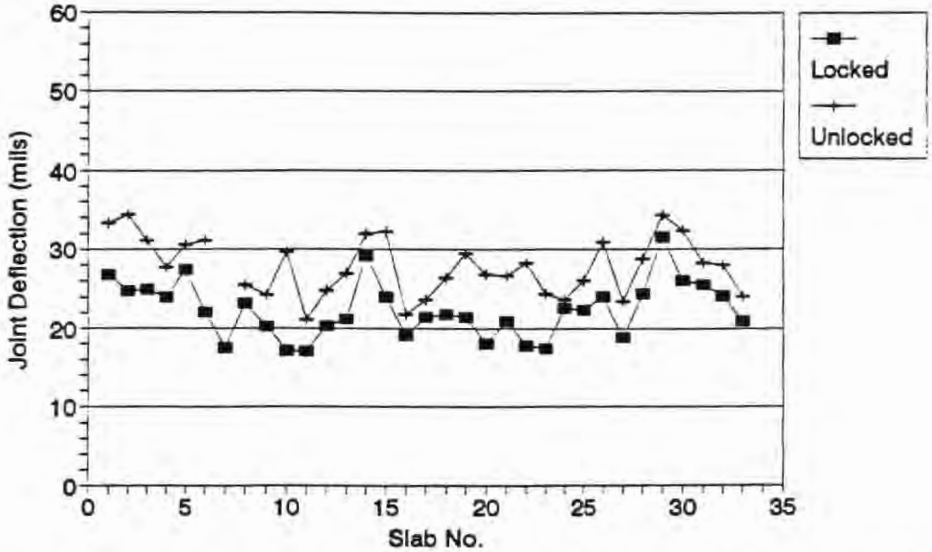


Figure 5.101. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in SD50, MRM 390, WB, Before Undersealing.

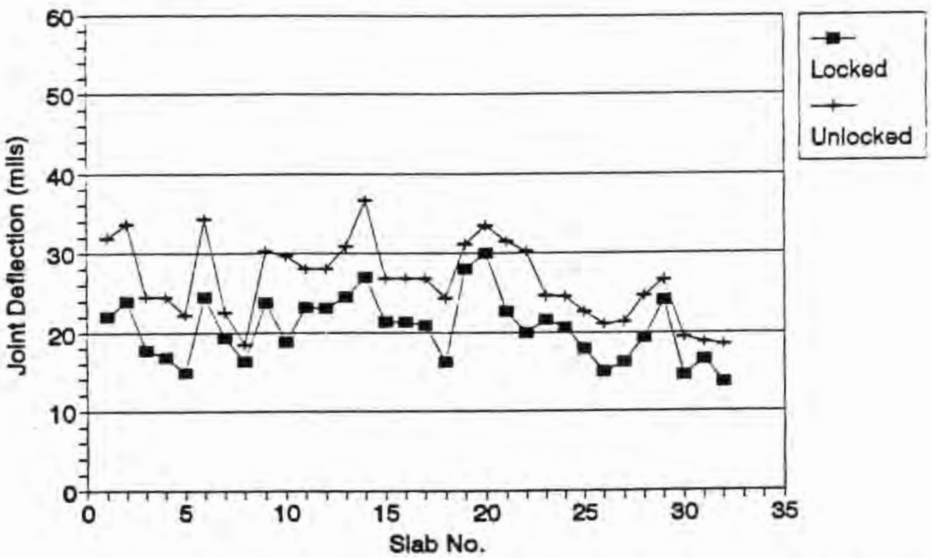


Figure 5.102. Comparison of Leave Slab Deflections For Locked and Unlocked Conditions in SD50, MRM 393 + 2700', WB, Before Undersealing.

Table 5.10. Summary of Mean Deflections and Load Transfer Efficiency Data.

Highway	MRM	Description	Mean Deflection (in)	Mean LTE (%)
US12	287 WB	Undersealed, 4 years	0.0266	46.1
US12	286 EB	Undersealed, 4 years	0.0271	31.1
US12	283 EB	Non-undersealed	0.0377	24.5
US12	283 WB	Non-undersealed	0.0427	21.9
US281	192 SB	Undersealed, 6 years	0.0196	51.3
US281	194 SB	Undersealed, 6 years	0.0225	46.7
US281	Melgaard Rd.	Non-undersealed	0.0245	44.6
US12	272.17	Undersealed, 3 days	0.01721	91.4
US12	272.17	Undersealed, 11 months	0.01521	85.7
US12	268.40	Undersealed, 28 days	0.01624	81.6
US12	268.40	Undersealed, 11 months	0.01704	62.7
US12	275.03	Before undersealing	0.03119	53.0
US12	275.03	Undersealed, 11 months	0.01430	74.7
SD50	389 EB	Before undersealing	0.0383	25.0
SD50	389 EB	After undersealing	0.0133	75.0
SD50	390 WB	Before undersealing	0.0278	37.8
SD50	390 WB	After undersealing	0.0101	90.7
SD50	393 WB	Before undersealing	0.0171	31.7
SD50	393 WB	After undersealing	0.0205	62.6

CHAPTER VI

PANEL REMOVAL

6.1. General

The purpose of the panel removal was to observe how the grout flowed under the slab, and if it had filled the voids or not. This was a major question that needed to be addressed regarding undersealing. It was also important to know if the grout was still in good condition, or if it had cracked and begun to erode.

6.2. Site Selection

Two sites were selected for the removal of two concrete panels. These sites were: 1) US12 near Aberdeen at MRM 286 EB at slab 15 of the test section, and 2) US281 at MRM 194 SB at slab 5 of the test section.

The basis for selecting the two locations was the low load transfer efficiency values obtained for the two slabs. Furthermore, the presence of faulting on US281 in addition to the high leave slab deflections measured on both US281 and US12 were used as a criteria in the selection process.

6.3. Method Used in Removing the Panels

There were several steps involved in removing the concrete panels. The first step was to saw the slab approximately 5 feet on each side of the joint from the centerline to approximately one foot onto the shoulder. The slabs were then sawed again nearly perpendicular to the first saw cuts in about the middle of the lane. This was necessary to reduce the size of the sections, and to make it easier to remove the different concrete sections. Finally, the shoulder was sawed approximately one foot from the edge of the pavement.

Cores were then taken of the slab and subbase at selected locations in the panels. Next, two hooks were installed in each separate panel. A chain was then attached to each hook and the panel was lifted out with a front-end loader.

Figures 6.1 and 6.2 show the locations of the saw cuts (dashed lines) and the cores removed (circles) at each of the two sites. Figures 6.3 through 6.7 show the basic steps involved in removing the panels.

6.4. Findings

The following sections will describe the results obtained at each site after the panels were removed.

6.4.1. Findings From US12 at MRM 286 EB at Slab 15

The panel removal on US12 provided much useful information. After removing the panel, it was verified that the grout did flow under the slab filling voids between the subbase and slab. The grout adhered to the bottom of the slab. However, the grout was cracked and this is typical of the brittleness of cement-fly ash grouts. A brittle material may easily crack under the combined influence of extreme temperature variations and traffic application. Figures 6.8 and 6.9 show the grout adhering to the bottom of the slab and the top of the subbase after the panel was removed.

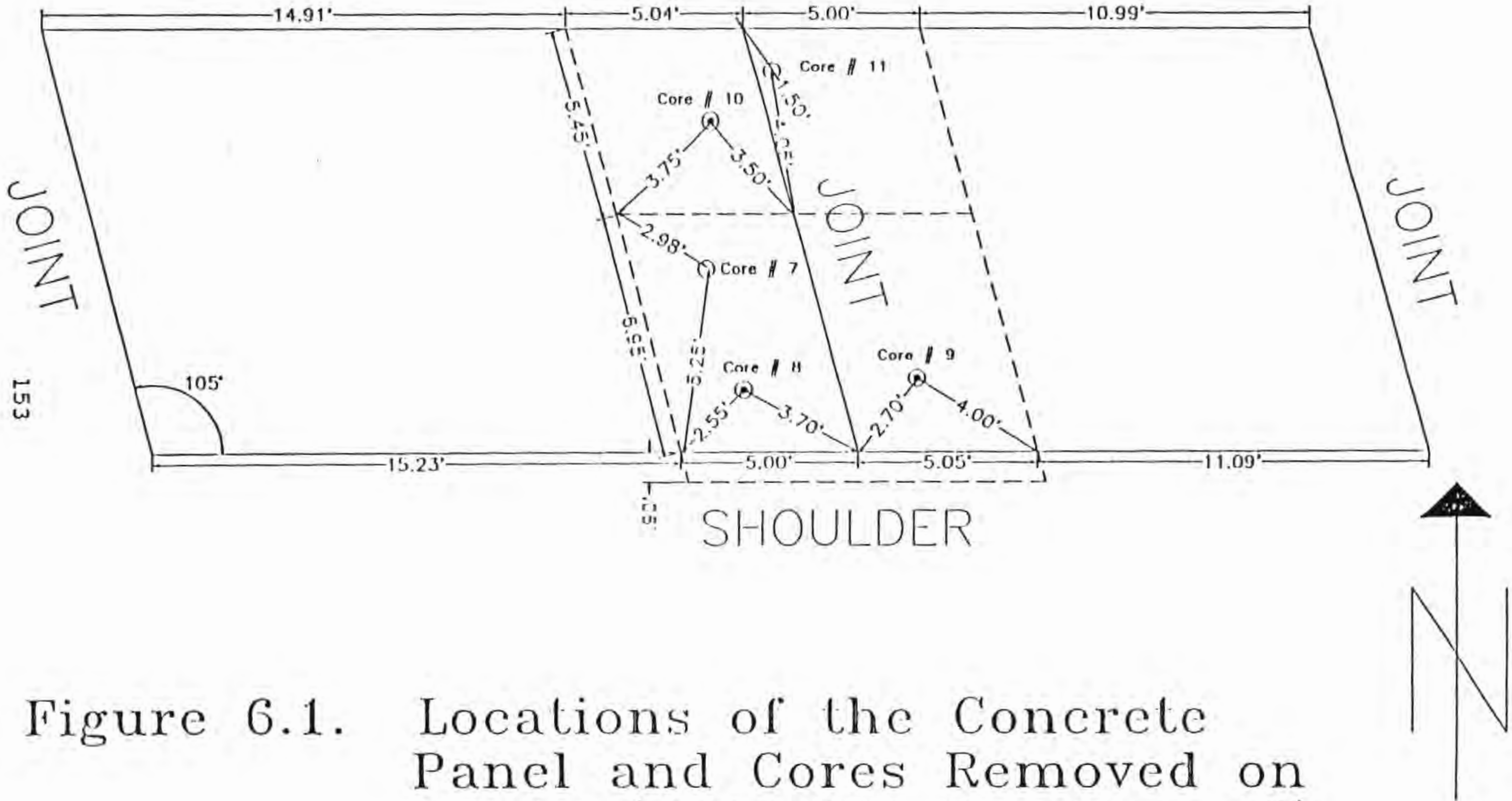


Figure 6.1. Locations of the Concrete Panel and Cores Removed on US 12 (MRM 286 EB, Slab 15).

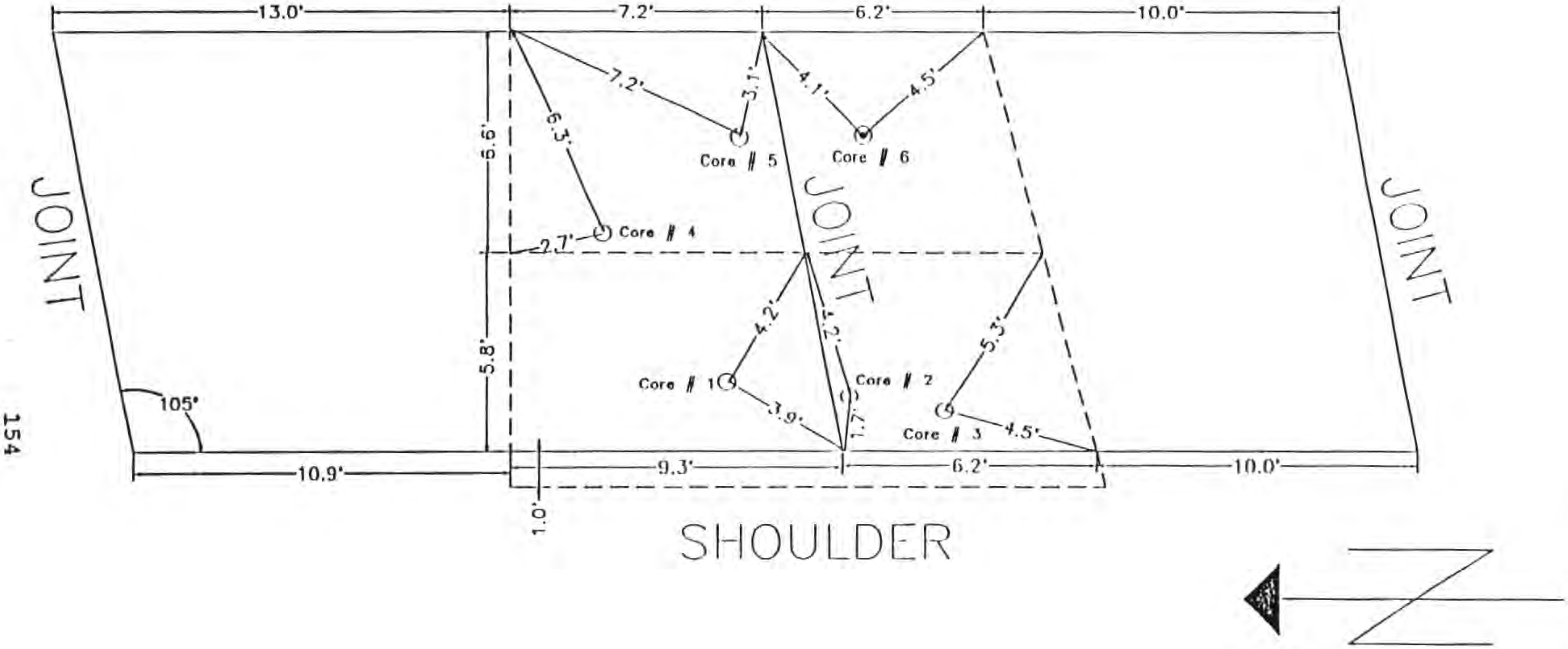


Figure 6.2. Locations of the Concrete Panel and Cores Removed on US 281 (MRM 194 SB, Slab 5).

Figure 6.3. Sawing of the Panels Across the Slab.

Figure 6.4. Sawing of the Concrete Panel Along the Edge of the Shoulder.

Figure 6.5. Removing Cores From the Concrete Panel.

Figure 6.6. Removal of the Concrete Panels Using Hooks.

Figure 6.7. Removal of the Concrete Panels With a Front-End Loader.

Figure 6.8. Bottom of a Concrete Panel Removed From US12.

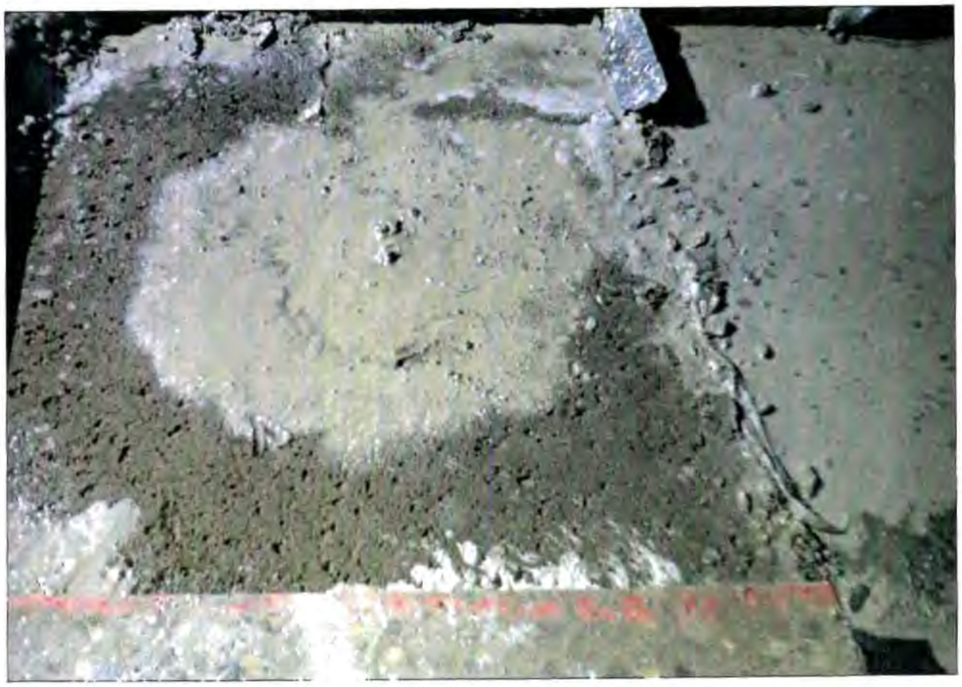


Figure 6.9. Top of the Subbase After the Removal of the Concrete Panel.

An unexpected finding was observed when inspecting the cores removed from this section. It was found that the grout had not only filled the voids between the subbase and slab, but it also had filled voids in the slab itself. This happened only in some of the cores which were taken where grout injection holes were located. These cores are designated with dots in the centers of the circles in both Figures 6.1 and 6.2. Figures 6.10 and 6.11 show the cores (core numbers 8 and 9) where grout has filled the voids within the concrete slab. The grout in the slab is outlined in black on both cores. The voids in the slab could have been caused by excessive pressure used during the drilling of the grout injection holes, causing the slab to crack.

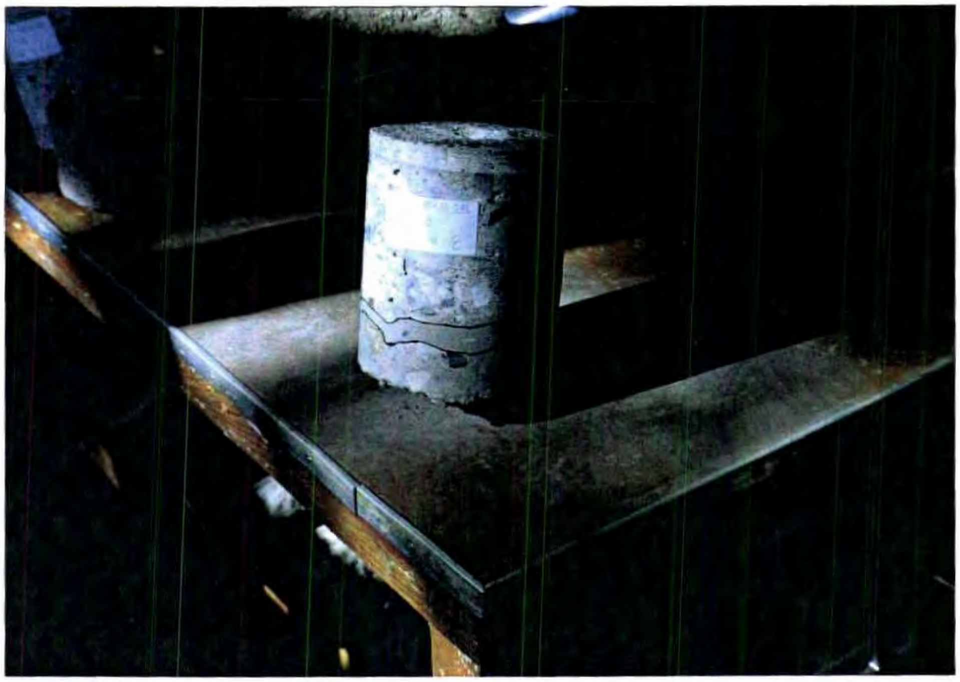


Figure 6.10. Grout Filling the Voids in the Concrete Slab in Core 8 (US12, MRM 286, EB).

Figure 6.11. Grout Filling the Voids in the Concrete Slab in Core 9 (US12, MRM 286, EB).

The top of the subbase that was removed with the cores at this location was in a poor condition (see Figure 6.12). The material was very soft, friable, and crumbled easily. Erodabilty of the subbase may lead to future void problems and eventual faulting of the pavement.

6.4.2. Findings From US281 at MRM 194 SB at Slab 5

Panel removal on US281 also provided much useful information. After removing the panel, it was verified that the grout did also flow under the slab filling voids between the subbase and slab. The grout adhered to the top of the subbase and was brittle and cracked. Figures 6.13 and 6.14 show the

Figure 6.12. Top of the Subbase Removed With the Core (US12, MRM 286, EB).

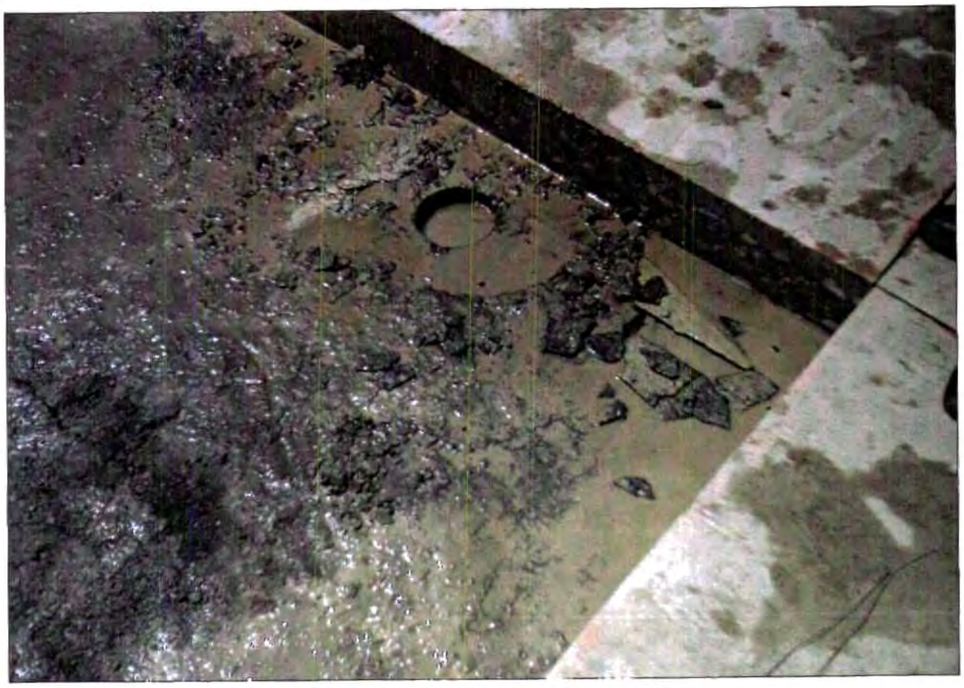


Figure 6.13. Adhesion of the Grout to the Top of the Subbase (US281, MRM 194, SB).

Figure 6.14. Bottom of the Concrete Panel Removed on US281, MRM 194, SB.

grout adhering to the top of the subbase and the bottom of the concrete panel. Figure 6.15 shows samples of grout obtained in this section. Figure 6.16 depicts core samples of the slab and subbase. The cracked grout on top of the subbase can be observed in Figure 6.16.

By examining the subbase and the cores obtained from US281, it was observed that the grout not only filled the voids between the subbase and the slab, but it also filled the voids within the subbase itself (see Figures 6.17 and 6.18).

6.5. Conclusions From the Panel Removal

Removal of the concrete panels indicated that the grout did flow under the slabs and filled the voids. It was also observed

Figure 6.15. Samples of Grout Removed From the Top of the Subbase on US281, MRM 194, SB.

Figure 6.16. Core Samples of the Slab and Subbase Removed from US281, MRM 194, SB.

Figure 6.17. Grout Filling the Voids in the Subbase on US281, MRM 194, SB.

Figure 6.18. Grout Filling the Voids in the Subbase in Core 6 (US281, MRM 194 SB).

that the grout also filled the voids within the slab and subbase. The grout at both sites was brittle in nature and cracked easily. This property is not desirable as erosion of the subbase may reassert itself again due to the combined actions of water, temperature, and traffic.

CHAPTER VII

PREDICT ANALYSIS

7.1. General

Each of the four projects selected for this study was analyzed using PREDICT. PREDICT is an IBM PC software application used to predict certain pavement distresses in JPCP (Jointed Plain Concrete Pavement) and JRCP (Jointed Reinforced Concrete Pavement). PREDICT was written by M. I. Darter of the University of Illinois-Urbana.

PREDICT uses the national regression models for transverse joint faulting, pumping, joint deterioration, slab cracking, and present serviceability rating (PSR) from COPES (Concrete Pavement Evaluation System) to estimate future distresses (1). COPES is the result of a study conducted under NCHRP Project 1-19, titled "Development of a System for Nationwide Evaluation of Portland Cement Concrete Pavements."

7.2. COPES

COPES was developed with the purpose of developing a system for state and national evaluation of concrete pavement performance. The system consists of three major components: data collection, storage and retrieval, and evaluation. Figure 7.1 shows a schematic of the COPES process. The data is stored on computer to increase the speed at which information can be retrieved and analyzed.

Under NCHRP Project 1-19, it was found that COPES is capable of efficiently collecting, processing, and evaluating

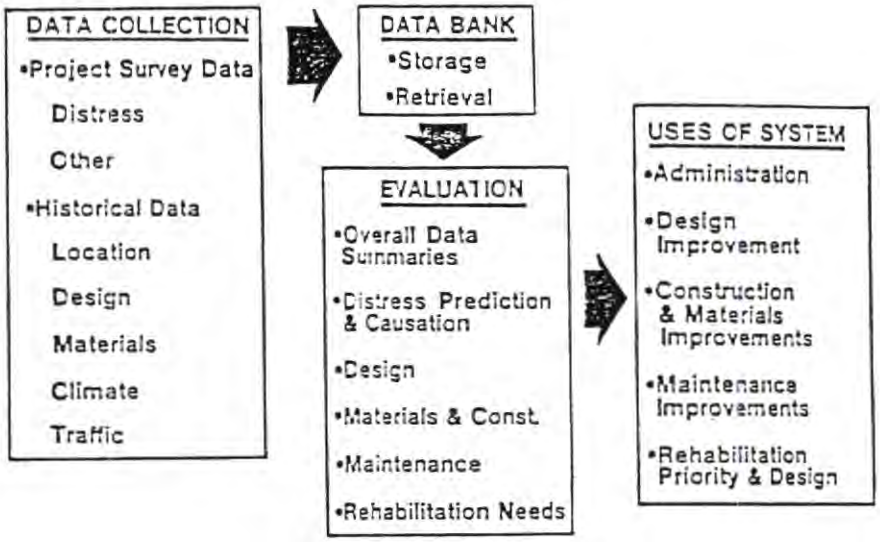


Figure 7.1. Illustration of the COPES Procedures (1).

large amounts of data to improve the design, construction, materials, and maintenance of concrete pavements (1).

The COPES data bank contains extensive information about 1305 miles (6 percent of the total miles) of Interstate concrete pavements. This data can be used to develop predictive models that can be used for estimating the remaining pavement life and future rehabilitation needs.

It will be necessary to expand the data base, however, to include more states with different climates, soils, traffic, and other conditions. Only then will it be possible to conduct a truly nationwide evaluation of concrete pavements (1).

7.3. National Regression Models

National regression models were developed for pumping, transverse joint faulting, joint deterioration, slab cracking and PSR (Present Servicabiltiy Rating) using the COPES data base.

The data sample used to develop the models was from 1305 miles of Interstate concrete pavement. This is approximately 6% of the total mileage of all Interstate concrete pavements. States from which the data were collected include: Illinois, Georgia, Utah, Minnesota, Louisiana, and California. A few sections from Nebraska were also included. The data for the Nebraska sections were collected under another study using the COPES method. Table 7.1 presents the number and total lengths of sections from each state.

Table 7.1. Data Used for National Regression Models (1).

State	JPCP		JRCP	
	Sections	Miles	Sections	Miles
California	45	141	0	0
Utah	33	98	0	0
Georgia	28	263	0	0
Illinois	38	2	184	409
Minnesota	1	7	52	233
Louisiana	5	22	24	122
Nebraska	0	0	8	8
Total	150	533	268	772

National regression models were developed for both JRCP and JPCP for each of the four distresses and PSR. The models were developed using both multiple linear regression and nonlinear regression. The determination of which dependent variables were having a significant effect on the independent variables was accomplished using multiple linear regression. The final models were then formed using nonlinear regression to assign coefficients and exponents to those independent variables found significant from multiple linear regression.

It should be noted that the national models should be considered as "initial" models (1). More time will be required to expand the models to address more variables and improve their functional form. Also, each model is based on available data. The models should not be extended outside the range of data from which they were developed.

In the following sections, the national regression models for pumping and transverse joint faulting in JPCP will be defined. Only those two models will be discussed, since no JRCP sections were evaluated in this study and pumping and faulting are the major pavement distresses addressed in this report.

7.3.1. National Model for Pumping in JPCP

The final national model for pumping of JPCP is as follows:

$$\begin{aligned} PUMP = {} & ESAL(0.443)[-1.479 + 0.255(1 - SOILCRS) \\ & + (0.0605)SUMPREC^{0.5} + 52.65/THICK^{1.747} \\ & + (0.0002269)FI^{1.205}] \end{aligned}$$

where:

PUMP = 0, no pumping; 1, low severity; 2, medium severity; 3, high severity;

ESAL = accumulated 18-kip equivalent single-axle loads, millions;

SOILCRS = 0, fine-grained subgrade soil; 1, coarse-grained subgrade soil;

SUMPREC = average annual precipitation, cm;

THICK = slab thickness, in; and

FI = freezing index.

Statistics for this model are as follows:

$R^2 = 0.68$;

SEE (Standard Error of Estimate) = 0.42; and

n (no. of data points) = 289.

Figure 7.2 shows the general effect of different variables on pumping. This figure shows that slab thickness has a major effect on pumping. This is probably due to the effect that slab thickness has on pavement deflections, which are part of the pumping mechanism (1). Coarse-grained subgrade soils also tend to reduce pumping, which reflects the ability of the granular material to drain moisture from under the pavement structure (1). Increased precipitation also tends to cause an increase in pumping.

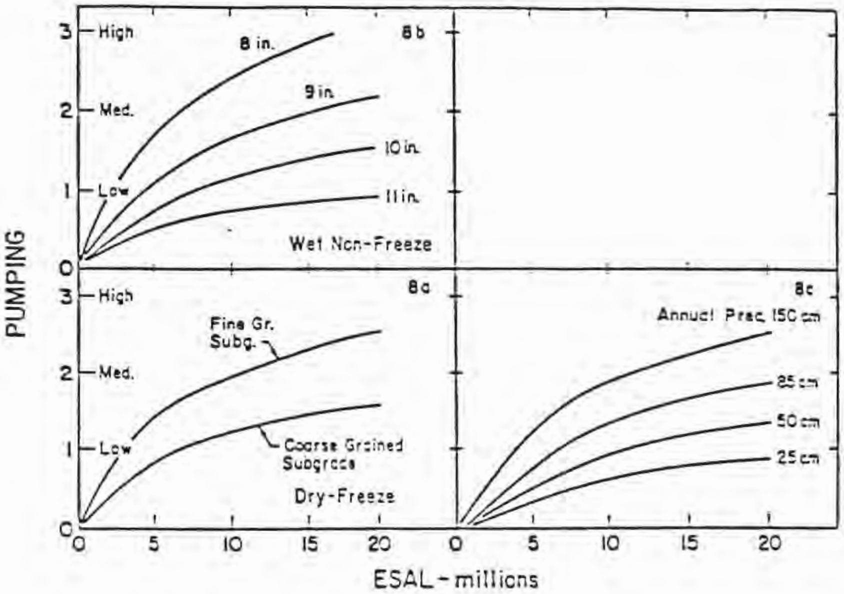


Figure 7.2. General Effects of Major Variables on Pumping (1).

7.3.2. National Model for Transverse Joint Faulting in JPCP

The final national model for transverse joint faulting in JPCP is as follows:

$$\begin{aligned} FAULT = ESAL(0.144)[&-0.2980 + 0.2761/THICK(0.3184) \\ &- (0.0285)BASETYP + 0.00406(FI + 1)^{0.3598} \\ &- (0.0462)EDGESUP + 0.2384(PUMP + 1)^{0.0109} \\ &- (0.0340)DOW^{2.0587}] \end{aligned}$$

where:

FAULT = mean transverse joint faulting, in.;

ESAL = accumulated 18-kip equivalent single axle loads, millions;

BASETYP = 0, if granular base; 1, if stabilized base (asphalt, cement, etc.);

THICK = slab thickness, in.;

EDGESUP = 0, if AC shoulder; 1, if tied PCC shoulder;

FI = freezing index;

PUMP = 0, if no pumping; 1, if low severity; 2, if medium severity; 3, if high severity;

DOW = diameter of dowel bar, in.;

= 0, if no dowel bars exist.

The statistics for this model are as follows:

$R^2 = 0.79$

SEE = 0.02 in.

n = 259.

Figure 7.3 shows the general effects of different variables on faulting. The figure shows that dowel bar diameter has a major effect on faulting, with decreased faulting being related to larger dowel bar diameters. Base type also has an effect on faulting. Use of stabilized bases appears to result in less faulting when compared to a non-stabilized granular base. Thicker slabs also tend to have less faulting. Finally, the use of a PCC shoulder significantly reduces faulting when compared to an asphalt shoulder.

7.4. Results of the PREDICT Program

In the following sections, the results of the PREDICT program will be presented for each site. Where pavement condition data are available, comparisons will be made with those results obtained from PREDICT. Also, when pavement condition data are collected after rehabilitation, the year of

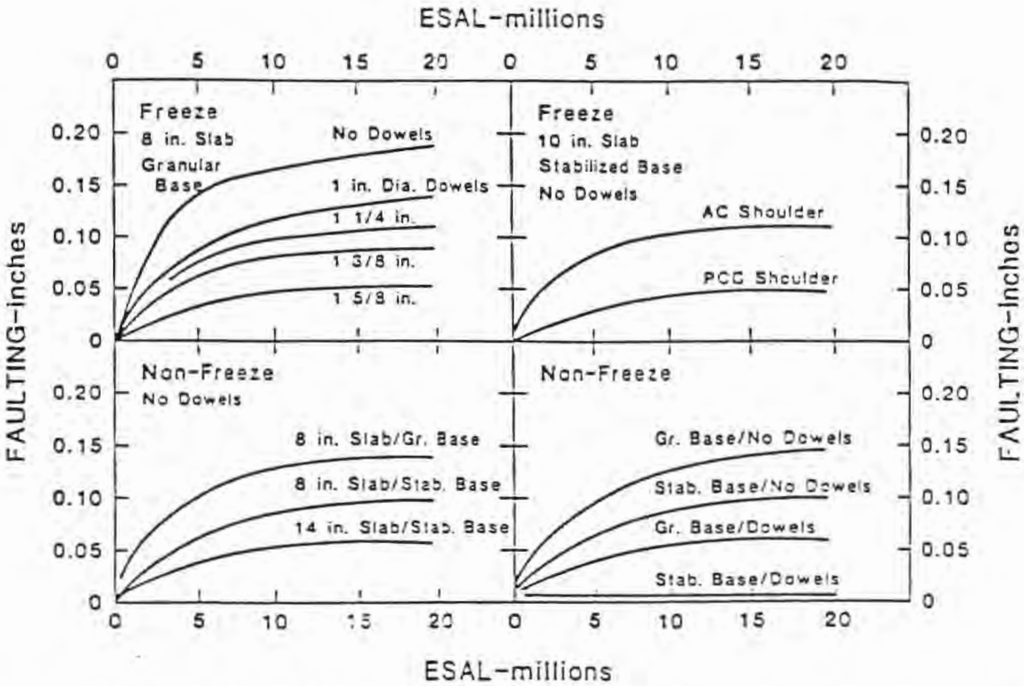


Figure 7.3. General Effects of Major Variables on Faulting (1)

rehabilitation will be assumed time zero, and comparisons will be made with respect to this time. If the section has not been rehabilitated, comparisons will be made with respect to the original construction year.

7.4.1. US12 From West of Mina at MRM 275 East 12.2 Miles to Aberdeen in Edmunds and Brown Counties

This project was rehabilitated in 1989. The pavement sections at MRM 287 WB and MRM 286 EB were undersealed and ground. The sections at MRM 283 EB and MRM 283 WB were not undersealed. The original construction date was not available.

Pavement condition data collected in April and May 1993 showed that the undersealed sections exhibited no visible pumping or faulting. The non-undersealed sections did exhibit some pumping and faulting. The section at MRM 283 WB showed little or no pumping, but had an average faulting of 0.125 inch. The section at MRM 283 EB exhibited low severity pumping and no significant faulting.

The results of the PREDICT program for pumping and faulting are shown in Figures 7.4 and 7.5. For the undersealed sections, the estimated pumping is expected to be of low severity, and faulting is expected to be approximately 0.08 inch according to PREDICT after 4 years. For the non-undersealed sections, faulting of 0.11 inch would be expected after 22 years, while medium severity pumping is expected to occur in 22 years. A comparison between the PREDICT results and the pavement condition survey data indicates reasonable matching

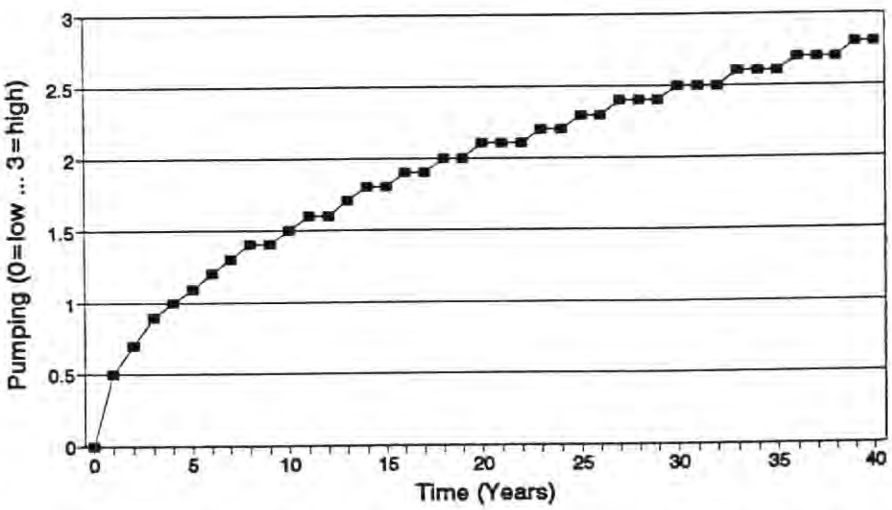


Figure 7.4. Expected Pumping using PREDICT for US12 from West of Mina to Aberdeen.

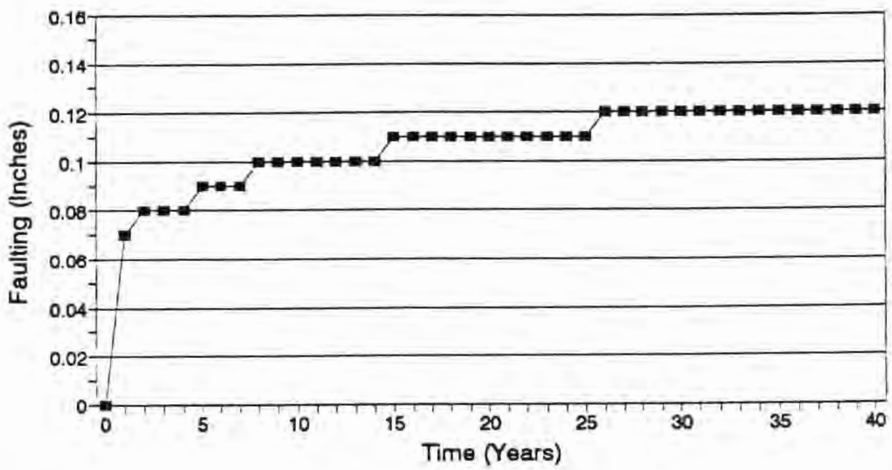


Figure 7.5. Expected Faulting using PREDICT for US12 from West of Mina to Aberdeen

of values for the non-undersealed sections, but not for the undersealed pavements.

7.4.2. US281 From US12 South 3.2 Miles in Brown County

This section was originally constructed in 1971. It was rehabilitated in 1987 when part of the pavement was undersealed and ground. At MRM 192 SB, only the driving lane was undersealed. The section at MRM 194 SB had both the driving and passing lanes undersealed. The section at Melgaard Road NB was not undersealed.

Pumping and faulting data were collected on these sections in April 1992. At MRM 192 SB, no pumping was observed. However, there was an average faulting of 0.09375 inch in the undersealed lane and faulting of 0.1875 inch in the non-undersealed section. The section at MRM 194 SB had an average faulting of 0.10625 inch and there was clear water being ejected from the joints near the shoulder during FWD testing. The Melgaard Road NB section had an average faulting of 0.25 inch in the driving lane and 0.125 inch in the passing lane. Also, clear water was being ejected from the joints in this section when the FWD load was applied.

The results of the PREDICT program are shown in Figures 7.6 and 7.7. These figures indicate that pumping in the undersealed sections is expected to be low to medium, and faulting is expected to be 0.10 inch after six years. On the non-

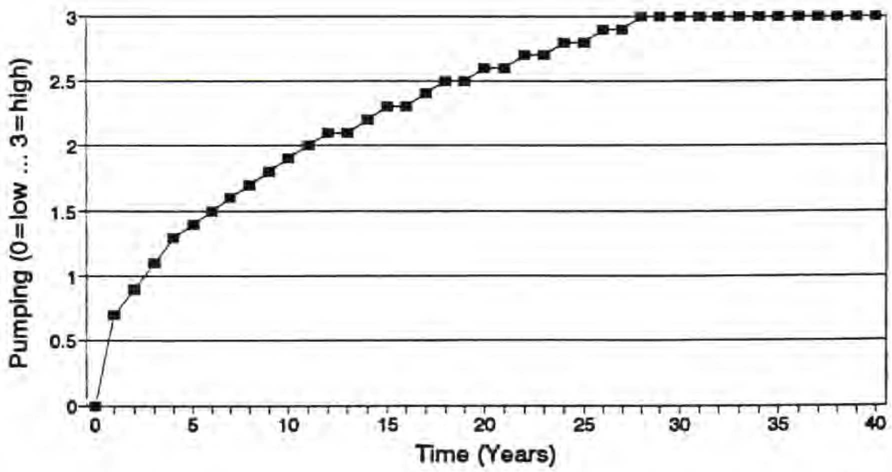


Figure 7.6. Expected Pumping using PREDICT for US281 from US12 South 3.2 Miles

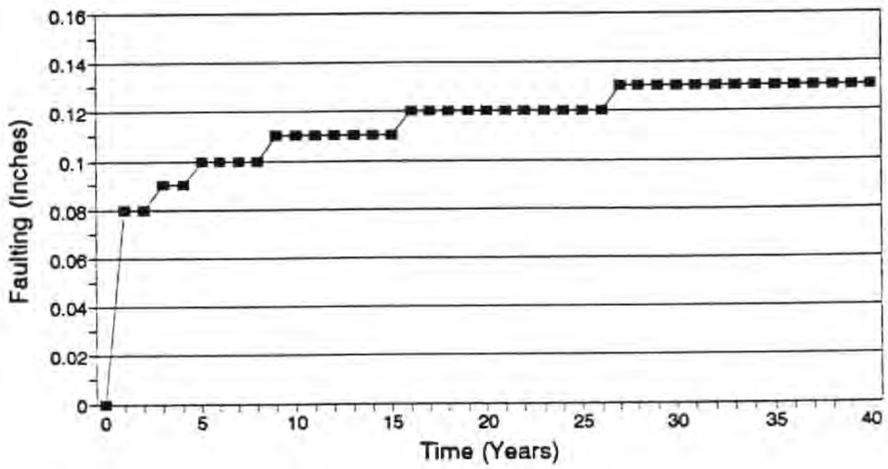


Figure 7.7. Expected Faulting using PREDICT for US281 from US12 South 3.2 Miles

undersealed sections, pumping is expected to be medium to high, and faulting is expected to be 0.12 inch. Here, the PREDICT results and the actual pavement condition surveys match well for the undersealed sections, but not for the non-undersealed pavements.

7.4.3. US12 From Ipswich East 11.4 Miles in Edmunds County

This pavement section was originally constructed in 1974. All test sections (MRM 268.4 WB, MRM 272.17 WB, MRM 275.03 WB, and MRM 279) were undersealed and were ground in 1992.

A visual inspection of the pavement sections was conducted in May 1993. None of the test sections were experiencing any visible pumping or faulting.

The results of the PREDICT program are shown in Figures 7.8 and 7.9. The figures indicate that after one year, pumping is expected to be moderately low to none, and faulting is expected to be 0.04 inch.

7.4.4. SD50 in Yankton County

This section was undersealed and was ground in July 1993. All test sections (MRM 389 EB, MRM 390 WB, and MRM 393 + 2700' WB) were undersealed. The year of original construction was not available.

A pavement condition survey was conducted in April 1993 before undersealing. The test section at MRM 389 EB had an average faulting of 0.2625 inch and no visible pumping. At MRM 390 WB, the average faulting was 0.125 inch and no visible pumping. At MRM 393 + 2700', the average faulting was 0.15625 inch with no visible pumping.

The results of the PREDICT program are shown in figures 7.10 and 7.11. The data indicate that the maximum expected faulting is about 0.10 inch after 27 years. Pumping is expected to be low to medium after 27 years of service.

7.5. Conclusions About PREDICT

When comparing the actual measured pumping and faulting to the estimated values obtained from the PREDICT program, it becomes apparent that the PREDICT results are inconsistent with the actual data. In some cases the PREDICT results matched the actual measured data fairly close, but in many cases they did not. This indicates that the current PREDICT software may not be

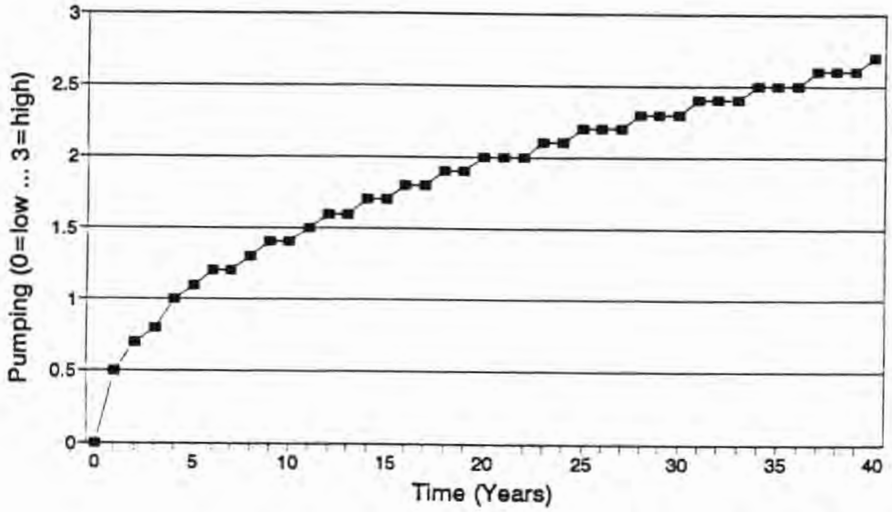


Figure 7.8. Expected Pumping using PREDICT for US12 From Ipswich 11.4 Miles East.

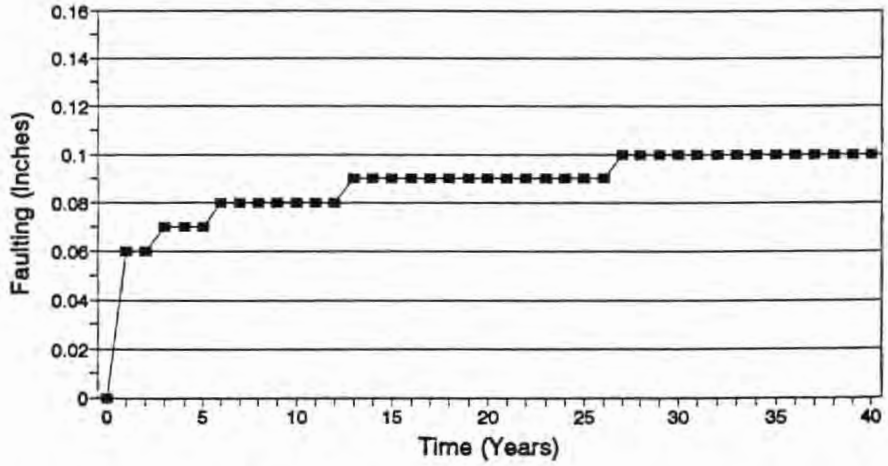


Figure 7.9. Expected Faulting using PREDICT for US12 From Ipswich 11.4 Miles East.

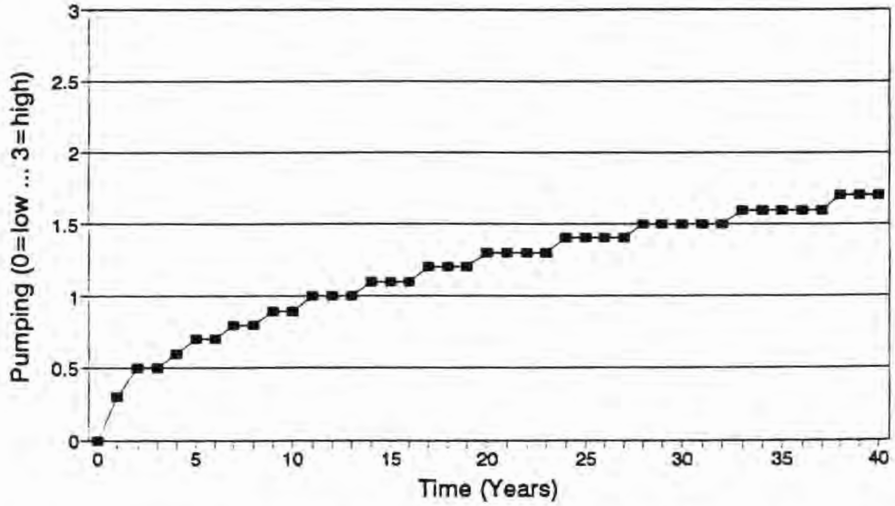


Figure 7.10. Expected Pumping using PREDICT for SD50 in Yankton County.

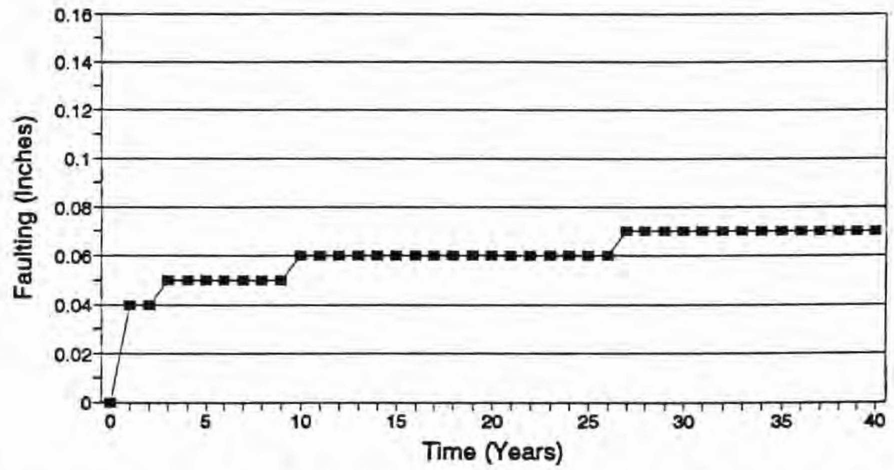


Figure 7.11. Expected Faulting using PREDICT for SD50 in Yankton County.

an effective method of estimating future pavement distresses. To improve the accuracy of the regression models, it will be necessary to expand the data base and models to include a wider range of variables.

7.6. Summary of Results

Observed and predicted pumping and faulting data for all four highways are provided in Table 7.2.

Table 7.2. Summary of Observed and Predicted Pumping and Faulting Data.

Highway	Observed Pumping Severity	Observed Faulting (inch)	Predicted Pumping Severity	Predicted Faulting (inch)
US12 West of Mina Non-undersealed Sections	low	0.125	medium	0.11
US12 West of Mina Undersealed Sections	none	none	low	0.08
US281 From US12 South Non-undersealed Sections	low	0.22	medium to high	0.12
US281 From US12 South Undersealed Sections	low	0.10	low to medium	0.10
US12 From Ipswich East Undersealed Sections	none	none	none to low	0.04
SD50 West of Yankton Before Undersealing	none	0.18	low to medium	0.10

CHAPTER VIII

COST ANALYSIS

8.1. General

In the following sections, a cost analysis will be carried out for each site to determine if undersealing is economically feasible. Data were furnished by the SDDOT Aberdeen Area Office. SD50 will not be included in this analysis, since no data were available.

8.2. Limitations

There are some limitations that will affect the cost analysis. First, undersealing does not work on its own. Other restoration works (such as concrete repair, joint sealing, grinding, etc...) must be considered in conjunction with undersealing.

Another limitation is that the selection of rehabilitation methods varies from one site to another. For instance, edge drains were installed only on US12 near Aberdeen, and not on any of the other sites included in this analysis.

A final limitation is that the quantities of materials required and their associated unit costs varies considerably. This is due to a number of factors, including pavement condition at time of rehabilitation, availability of materials, bid price and quantities, and many others.

8.3.1. US12 From West of Mina at MRM 275 East 12.2 Miles to Aberdeen

This site consisted of 8.2 two-lane miles and 4.3 four-lane miles, which is equivalent to 16.8 two-lane miles. All but 2 miles of the two-lane roadway were undersealed. Table 8.1 presents the different rehabilitation techniques implemented on the site, project costs, percent of total project cost per item, and cost per two-lane mile. The data indicate that the cost of undersealing was about $15,100 per two-lane mile and was about 9 percent of the total project cost. The total rehabilitation project cost was $154,580 per two-lane mile.

Table 8.1. Cost Breakdown for US12 From West of Mina to Aberdeen.

Rehabilitation Method	Project Cost ($)	Percent of Project Cost	Cost per Two-Lane Mile
Concrete Repair	881,600	35%	52,480
Undersealing	222,900	9%	*15,100
Grinding	250,300	10%	14,900
Reseal Joints	330,800	13%	19,700
Shoulder Restoration	621,400	28%	37,000
Edge Drains	258,400	10%	15,400
Total Cost	$2,565,400		$154,580

*This is based on 14.8 two-lane miles undersealed.

8.3.2. US281 From US12 South 3.2 Miles in Brown County

Undersealing on this section was done in conjunction with rehabilitation on US12. The total project consisted of 19.14 two-lane miles. Undersealing was done only on US281. Table 8.2 presents the project costs, the different rehabilitation techniques performed, the cost per two lane mile, and the percent of total project cost for each item. The cost of undersealing was about $22820 per two-lane mile on the section that was

Table 8.2. Cost Breakdown for the Rehabilitation Project Including US281 From US12 South 3.2 Miles in Brown County.

Rehabilitation Method	Project Cost ($)	Percent of Project Cost	Cost per Two-Lane Mile
Concrete Repair*	1,312,200	62%	68,570
Undersealing**	165,200	8%	8,630
Grinding	389,200	18%	20,330
Slope Modification	139,400	6%	7,280
Shoulder Restoration	128,100	6%	6,690
Total Costs	$2,134,100		$111,500

*The concrete repair item included new median concrete on US12 west of US281 and at the east edge of Aberdeen.

**Undersealing was done only on US281 and actual cost was $22,820 per two-lane mile which if done throughout at this rate would have been more like 18% of total cost.

actually undersealed. This would be equivalent to 18 percent of the total project cost if undersealing was done throughout the project at this rate. This would change the cost per two lane mile for the entire project to $125,690 per two-lane mile.

8.3.3. US12 From Ipswich East 11.4 Miles in Edmunds County

This project was undersealed in 1992 and it consisted of 13.88 two-lane miles of complete restoration. The cost breakdown of each item and its percent of the total cost is presented in Table 8.3. The cost of undersealing on this project was about $31,491 per two lane mile. This turns out to be approximately 19 percent of the total project cost. The whole rehabilitation project cost was approximately $167,042 per two-lane mile.

Table 8.3. Cost Breakdown for the Rehabilitation Project Including US12 From Ipswich 11.4 Miles East.

Rehabilitation Method	Project Cost ($)	Percent of Project Cost	Cost per Two-Lane Mile
Concrete Repair	792,751	34%	57,115
Undersealing	437,094	19%	31,491
Grinding	734,941	32%	52,950
Resealing Joints	353,740	15%	25,486
Total Cost	$2,318,526		$167,042

8.4. Conclusions About the Costs of Undersealing

The average cost per two-lane for rehabilitation projects considered in this analysis is approximately $149,000.
To determine if undersealing is cost-effective, it will be necessary to determine the length of time that pavement life can be extended by undersealing. If undersealing done in conjunction with other restoration works can prolong the life of the pavement 10 to 15 years (at the meeting with SDDOT personnel in December 1992, this was indicated as the goal of rehabilitation projects including undersealing), then undersealing may be an economic alternative. If undersealing was found not to prolong the pavement life 10 to 15 years, then other alternatives might be considered for rehabilitation, such as a crack, seat, and overlay the pavement.

CHAPTER IX

FINDINGS, CONCLUSIONS, AND RECOMMENDATIONS

9.1. General

The project was divided into several tasks, including: (1) Review of the literature relevant to undersealing, (2) National survey of current highway undersealing practices, (3) Meeting with SDDOT field engineers that have worked on undersealing projects, (4) Field evaluation of four projects which were undersealed in South Dakota in 1987, 1989, 1992, and 1993, (5) Execution of the PREDICT computer program to estimate future pumping and faulting, (6) Brief cost analysis, (7) The development of conclusions, and (8) The development of recommendations.

The project was performed by faculty and graduate students from the Civil Engineering Department at South Dakota State University during the period October 1, 1992 through February 28, 1994. The results obtained on the above tasks are summarized below.

9.2. Findings

9.2.1. Review of the Literature

A review of the literature indicated that the factors involved in the development of pumping and faulting in undoweled plain jointed PCC pavements are: (1) free water under the slab; (2) unstabilized or erodible material under or adjacent to the slab; (3) cracks or joints in the pavement; and (4) deflection of the slabs under heavy moving wheel loads. If any of these factors is absent, pumping and faulting will not occur.

The rehabilitation procedures necessary to correct pumping and faulting problems depend upon the condition of the pavement and may include slabjacking, undersealing, full/partial depth repairs, load transfer restoration, installation of edge drains, crack sealing, joint resealing, diamond grinding, shoulder improvement, and resurfacing. The point at which to choose between pavement restoration and resurfacing is still very much in debate. Generally, this will be dictated by the condition of the pavement and funding availability.

The design of an undersealing project includes the selection of an acceptable undersealing material, testing of the pavement for voids, estimation of undersealing quantities, specification of adequate construction practices, and determination of the optimal time in a pavement's life cycle to perform undersealing work.

9.2.2. National Survey

The survey conducted in this research project revealed the following;

1. Out of 33 states who responded to the survey, undersealing is performed on a limited basis by 16 state highway agencies.
2. Visual surveys and Falling Weight Deflectometer (FWD) testing are the primary methods used for void detection.
3. Corner deflections between 0.01 and 0.035

inch or a load transfer efficiency less than 65 percent have generally been adopted as a criteria in the determination of those slabs that need undersealing.

4. Fly ash-cement grout is the most widely used material in undersealing.
5. Maximum allowable pumping pressure (excluding the initial surge pressure) varies from 20 to 100 psi.
6. Maximum allowable vertical slab movement varies from 0.0175 to 0.25 inch.
7. Estimating quantities for undersealing is generally based on judgement and previous experience.
8. Grout pumping is generally stopped when grout starts to flow between the holes or comes out of joints; or when maximum allowable pumping pressure or maximum allowable vertical slab movement has been attained.
9. Long term effectiveness of undersealing is generally established by observing that pumping and faulting distresses are not resumed in the concrete panels.

9.2.3. SDDOT Design Practices

The SDDOT incorporates in its special provision for undersealing many of the design practices reported by the survey participants. One notable exception is the corner deflection criterion. The SDDOT uses a corner deflection of greater than 0.01 inch to select those slabs that need undersealing. All other states use higher deflections in their specifications. The SDDOT should investigate relaxing the corner deflection criterion as cost savings may be realized from undersealing fewer slabs. Furthermore, undersealing will probably be more effective in reducing slab deflections if the initial corner deflections were higher than 0.01 inch.

9.2.4. Field Evaluation

The field evaluation task consisted of visual surveys, FWD testing and analysis, and the removal of two concrete panels to allow visual inspection. The following four projects were selected for evaluation:

1. US281 from US12 South 3.2 miles in Brown county (undersealed/non-undersealed pavement sections).
2. US12 from West of Mina at MRM 275 East 12.6 miles to Aberdeen in Edmunds and Brown counties (undersealed/non-undersealed sections).
3. US12 from Ipswich East 11.4 miles in Edmunds county.
4. SD50 from East of Yankton at MRM 386.64 East 7.258 miles in Yankton county.

The basis for selecting these projects was the availability of FWD data prior to undersealing, the presence of non-undersealed pavement sections, and the age of the pavement after undersealing. Profile plots of deflections versus joint number, plots of load transfer efficiency versus joint number, and load versus deflection graphs were generated for all pavement sections. Furthermore, two concrete panels were removed from US281 and US12 from West of Mina. The analysis of the field data revealed the following:

1. The project on US281 was undersealed in 1987. In April 1993, an average faulting of 0.1 inch was measured in the undersealed sections in comparison to a measured faulting of 0.19 inch in the non-undersealed sections. Low severity pumping and a few corner breaks were also observed on this highway. However, no transverse and longitudinal cracking was observed in the pavement sections. Average corner deflections and load transfer efficiency values for the undersealed sections were 0.021 inch and 49 percent, respectively. For the non-undersealed sections, average corner deflection and load transfer efficiency values were 0.025 inch and 45 percent, respectively.

2. The project on US12 from West of Mina was undersealed in 1989. No faulting or any other type of distress were observed in the undersealed sections (with the exception of low severity D-cracking). However, the non-undersealed sections had low severity pumping and

an average faulting of 0.125 inch. Average corner deflection and load transfer efficiency values for the undersealed sections were 0.027 inch and 39 percent, respectively. For the non-undersealed sections, average corner deflection and load transfer efficiency values were 0.042 inch and 23 percent, respectively.

3. The project on US12 from Ipswich East was undersealed in 1992. No signs of pumping or faulting were observed on this highway. Average corner deflection and load transfer efficiency values of 0.016 inch and 75 percent were obtained, respectively.

4. The project on SD50 was undersealed in 1993. Average corner deflection and load transfer values of 0.013 inch and 76 percent were obtained after undersealing, respectively.

5. Removal of two concrete panels from US281 and US12 from West of Mina indicated that the cement-fly ash grout did flow beneath the slabs. In addition to stabilizing the voids present between the slab and the subbase, the grout filled voids present in the slab and the subbase. The grout at both sites was generally cracked.

9.2.5. PREDICT Analysis

PREDICT is an IBM PC software application developed under NCHRP project 1-19 to estimate future pavement distresses in jointed plain and jointed reinforced concrete pavements. PREDICT uses regression models for faulting, pumping, slab cracking, joint deterioration, and present serviceability rating (PSR). These models were developed based on data collection from 1305 miles of Interstate concrete pavements.

PREDICT was executed for the previous four projects in order to estimate future pumping and faulting. While the observed and estimated values matched fairly close in some instances, they did not in many other cases. PREDICT seems to underestimate the extent of faulting with time.

9.2.6. Cost Analysis

Many variables should be considered in any cost analysis for pavement rehabilitation projects. Whether to perform restoration work or resurfacing on a pavement depends entirely on the condition of the pavement and the availability of funds. Undersealing is only one item in the whole restoration process. Other items include concrete repair, grinding, shoulder restoration, resealing joints, edge drains, and safety improvement.

Based on cost data provided by the SDDOT Aberdeen Area Office, the cost of undersealing varied from 9 to 46 percent of the total project cost. This can be equivalent to $15,000 to $41,000 per two-lane mile. Depending on the condition of the pavement, the total cost of performing restoration varied from $80,000 to $191,000 per two-lane mile. If resurfacing (crack, seat, and overlay) is selected as the rehabilitation option, the total cost will vary from $175,000 to $190,000 dollars per two-lane mile. However, such estimates will be misleading if total life-cycle costs are not considered in the final cost analysis.

9.3. Conclusions

The results of the study warrant the following conclusions:

1. All undersealed pavement sections performed better than the non-undersealed sections.
2. Best performance was obtained immediately after grouting. Performance generally degraded with time.
3. Undersealing is effective in filling voids and delays but does not prevent faulting development.
4. The cement-fly ash grout was generally cracked and may indicate long-term durability problems.
5. With the exception of minor faulting (0.1 inch) and few corner breaks in the undersealed pavement sections on US281, there were no load-associated cracks or high corner deflections to indicate any structural integrity problems (after seven years). This suggests that any further rehabilitation on this highway could be delayed a few more years.
6. No faulting or any other type of distress was observed on US12 from West of Mina (undersealed sections) after five years. Rehabilitation on this highway is still a few years away.
7. The current SDDOT special provision for undersealing is acceptable based on field performance and other highway agencies' experiences. However, the SDDOT should consider the use of higher corner deflection criterion

and other grouts for long term performance and cost savings.

8. The question of when and where undersealing is appropriate and should be addressed within the context of a pavement management system. If the SDDOT elects to restore rather than resurface a pavement, then undersealing should be part of the restoration plan provided that faulting does not exceed 0.20 inch and the pavement is still in a fair condition.

9.4. Recommendations

The primary recommendations for this research project are as follows:

1. Undersealing should be part of the overall strategy in the restoration process of jointed concrete pavements.
2. Based on the results of this study and the evaluation of design and construction guidelines used by other states, it is recommended that the SDDOT implement the following regarding their undersealing practices:
    a. Undersealing should be performed on concrete slabs that have corner deflections greater than or equal to 0.025 inch. This will provide cost savings from undersealing fewer slabs and probably yield better performance from undersealing only those slabs that have high deflections.
    b. Undersealing would be more effective when faulting does not exceed 0.20 inch and the pavement is

still in a fair condition (i.e. no extensive cracking). When loss of support has progressed to extensive pavement distresses (such as transverse and longitudinal cracking, corner breaks, and transverse joint spalling); cracking, seating, and overlaying the pavement will be more effective than performing other restoration works.

c. Undersealing and other restoration works should be performed on undoweled jointed concrete pavements that are about 22 years old. This is true if the pavement is still in a fair condition and is not experiencing extensive cracking, joint spalling, and corner breaks. In this situation, the main mode of distress will be an average faulting of 0.20 inch or less. Undersealing done in conjunction with other restoration works will probably extend the life of the pavement another 10 years. After that, the pavement will need major structural support and undersealing will not be effective.

d. Installation of edge drains should be part of any restoration plan in the rehabilitation of undoweled jointed concrete pavements. At the time of testing, US12 from West of Mina (where edge drains were installed) did not experience any pumping in comparison to US281 (no edge drains) where pumping was observed. If pumping is reduced or prevented, then faulting and loss of structural support in the pavement will be minimized.

   e. Current SDDOT design, specifications, and construction procedures regarding undersealing are acceptable except as noted in 2a, 2b, and 2d.

3. All pavements evaluated in the research study consist of 7 inches of plain jointed concrete with 4 inches of lime-treated cushion base. The subgrade soils are generally characterized by low plasticity clays (CL). Rehabilitation on such pavements was performed after about 18 years of service. Therefore, on new construction, it is recommended that the SDDOT consider the implementation of the following:

   a. Lean concrete or asphalt treated bases may work better to minimize or prevent pumping. The literature review and other DOT experiences indicated that cement treated bases are also susceptible to pumping.

   b. The use of concrete rather than asphalt shoulders seems to provide better performance.

   c. The literature indicates that increasing the slab thickness by one inch will reduce deflection by 10% which in turn reduces pumping and faulting.

   d. DOT experiences indicate that the use of mechanical load transfer devices such as dowel bars can provide a high degree of joint load transfer efficiency and thus minimize or prevent faulting.

e. The literature indicates that water infiltration at the slab bottom near the joints can be minimized by providing proper drainage, using an open-graded drainage layer, and adequately sealing joints.

4. The SDDOT should investigate the use of polyurethane as an undersealant (this may provide better long term performance than cement-fly ash grout).

5. The SDDOT should continue monitoring and data collection on the pavement sections and the SHRP Specific Pavement Studies (SPS) sections in the state.

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APPENDIX A

A SAMPLE OF THE SURVEY FORM

The Civil Engineering Department at South Dakota State University is conducting a study titled "EVALUATION OF UNDERSEALING OF UNDOWELED PLAIN JOINTED PCC PAVEMENTS" for the South Dakota Department of Transportation. We would appreciate your response to the following questions. Results of the survey will be shared with you upon completion

---

Name:______________________________ Date:__________________

Title:______________________________ Agency:__________________

Please Mark With An (X) Where Appropriate:

A. DOES YOUR AGENCY IMPLEMENT SLAB UNDERSEALING AS PART OF ITS PAVEMENT RESTORATION/REHABILITATION PROGRAM? ☐ Yes ☐ No IF YOUR ANSWER IS NO, PLEASE INDICATE WHY NOT?______________________________

B. WHEN AND HOW DO YOU DECIDE TO IMPLEMENT UNDERSEALING WORK?______________________________

C. MATERIALS USED FOR UNDERSEALING

(a) Type of Undersealing Material

☐ Epoxy ☐ Limestone Dust-Cement ☐ Pozzolan-Cement ☐ Asphalt Cement

☐ Polyurethane ☐ Others__________________

(b) Use of Additives

☐ Superplacticizer ☐ Water Reducer ☐ Fluidifiers ☐ Calcium Chloride

☐ Expanding Agent ☐ Others__________________

(c) Mix Design Requirements

Compressive Strength__________________ Flowability__________________

Mix Proportions______________________________

Specific Gravity__________________ Durability__________________

Others______________________________

(d) Basis for the Estimation of Material Quantity Used in an Undersealing Project______________________________

D. CONSTRUCTION METHODS

(a) Means of Determining Locations of Voids to Underseal

☐ Visual ☐ Falling Weight Deflectometer ☐ Dynaflect ☐ Road Rater

☐ Benckelman Beam ☐ Thumper ☐ Ground Penetrating Radar ☐ Others__________________

(b) Basis or Criteria as to Which Voids Will be Undersealed______________________________

(c) Any Weather Limitations for Performing an Undersealing Operation________________

(d) Drilling Holes for Material Injection

Hole Pattern________________

____

Diameter________________ Penetration Depth________________

Others________________

(e) Maximum Allowable Pumping Pressure________________

(f) When Do You Stop Pumping?________________

____

(g) Maximum Allowable Vertical Slab Movement During Pumping________________

____

(h) Types of Materials Used in the Sealing of Injection Holes________________

____

(i) Basis for the Acceptance of an Undersealing Project________________

____

(j) How Soon are Lanes Open to Traffic After Work Completion?________________

E. METHOD OF PAYMENT

____

____

____

F. OTHER WORK PERFORMED IN CONJUNCTION WITH UNDERSEALING________________

____

____

G. HOW DO YOU DETERMINE THE EFFECTIVENESS OF AN UNDERSEALING PROJECT?________________

____

____

H. OTHER COMMENTS________________

____

____

____

Please Return the Questionnaire to the Following Address:

Ramzi Taha, Assistant Professor
Civil Engineering Department
South Dakota State University
Brookings, SD 57007-0495

THANK YOU FOR YOUR COOPERATION

APPENDIX B

LIST OF ATTENDEES

Name	Affiliation
1. Vern Schaefer	SDSU
2. Gary Dejong	SDDOT
3. Scott Schneider	SDDOT
4. David Drake	SDDOT
5. John Ritterhaus	SDDOT
6. Larry Afdahl	SDDOT
7. Blair Lunde	SDDOT
8. Merle Swenson	SDDOT
9. Toby Crow	SDDOT
10. Don Anderson	SDDOT
11. Wayne Cramer	SDDOT
12. Ramzi Taha	SDSU
13. Sa'd Hasan	SDSU
14. Matthew Brey	SDDOT
15. Jim Stodhiem	SDDOT
16. Ernest Stolz	SDDOT
17. Ali Selim	SDSU
18. Robert Orcutt	SDDOT