THE WACO PONDING PROJECT

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This report presents results of field studies conducted between 1957 and 1972 on the effectiveness of ponding and lime stabilization of clay subgrade to minimize volume change beneath portland cement concrete pavements. Potential vertical rise (PVR) was calculated to identify sections in need of ponding, and the relationship of PVR to roughness and heaving of pavement is presented. The thickness of asphaltic concrete overlay required for pavement over untreated subgrade is compared to that required for concrete pavement over lime-stabilized subgrade, some of which was ponded. Although a study of underdrains was not intended as part of this project, it became noticeable that the result of connecting perforated underdrains to ditch drop inlets was to increase heaving and overlay repair thicknesses. A method for determining desired moisture content is presented, and it correlates fairly well with moisture contents obtained from below pavement after several years.

•THIS paper discusses work begun in Waco, Texas, in 1957 to decrease the detrimental effects of heaving suffered by portland cement pavement previously placed in that area.

The term Waco Ponding Project applies to special portions of an 8.133-mile (13.08-km) project on Interstate 35 in McLennan County, Texas (Figure 1). More specifically the term applies to 18 sections (26.7 percent of the total length of project) in the southbound main lanes between Elm Mott and a point just north of West (Figure 1 and Table 1). These sections varied in length from 200 to 1,600 ft (61 to 488 m) and were distributed throughout the length of the project. They were selected as representative of some of the highest volume change conditions on the project. The project was not set up for research study, but numerous investigations were performed, including moisture movements, pavement movements, and pavement performance over a period of several years. There were two concrete highways side by side; one was 35 years old and the other was 5 years old. Both highways were rough and in need of leveling to improve serviceability.

The basic geological units encountered on this location are Upper Eagle Ford group, Austin chalk, and Lower Taylor marl member. The Lower Taylor member is the predominant unit, for approximately 75 percent of the location is over this outcrop area.

The Taylor formation is a neritic marine unit deposited near the edge of the stable Texas craton. The Lower Taylor marl is a dark gray to dark yellow clay. Fresh exposures display blocky conchoidal fracture and develop poor fissility and lighter color upon weathering. The marl is composed of silt-sized quartz, calcite fragments, phosphate nodules, hematite, and finely disseminated pyrite and pyrite nodules. The dominant clay mineral is montmorillonite.

Soils identified by soil series as established by the Soil Conservation Service are as follows: Houston black clay, Houston clay, Wilson clay, Burleson clay, Axtell fine

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Figure 1. Location of study sections.

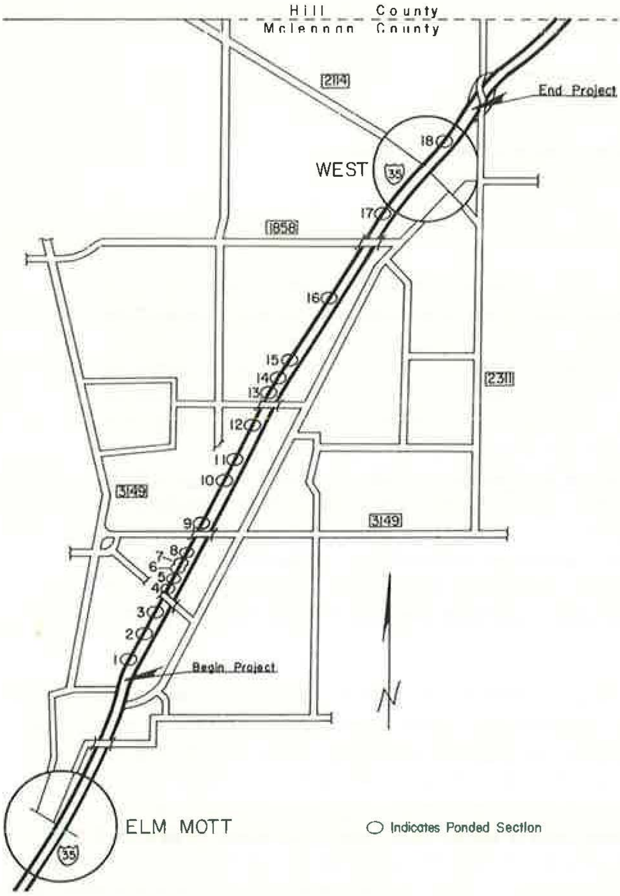


Table 1. Station limits of sections.

Section Number	Station Limits	Length (ft)	Percentage of Total Length
1	482+00 to 487+50	550	1.3
2	499+00 to 505+00	600	1.4
3	512+00 to 514+00	200	0.5
4	526+00 to 529+00	300	0.7
5	532+00 to 537+00	500	1.2
6[a]	539+00 to 545+00	600	1.4
7[a]	542+00 to 550+00	800	1.8
8	551+00 to 557+00	600	1.4
9	572+00 to 588+00	1,600	3.7
10	617+00 to 620+00	300	0.7
11	629+00 to 633+00	400	0.9
12	653+00 to 657+00	400	0.9
13	666+00 to 671+00	500	1.2
14	682+00 to 688+00	600	1.4
15	698+00 to 709+00	1,100	2.6
16	744+00 to 749+00	500	1.2
17	802+00 to 817+00	1,500	3.5
18	863+00 to 870+00	700	1.6

Note: 1 ft = 0.3 m.

[a]Overlap area.

sandy loam, Irving sandy loam and clay loam, and Austin-Eddy soils over the Austin chalk outcrop area.

Most of the soils listed are heavy black to gray residual clays with high to extremely high plasticity indexes.

HISTORY AND DETAILS OF THE PROJECT

When the southbound main lanes of I-35 between Elm Mott and West were constructed in the late 1950s, the most promising techniques for improving highway pavement performance were tried in this area where pavement performance had been very poor. The intent was to decrease the detrimental effects of heaving suffered by portland cement concrete pavements previously placed in the same area.

Of all the factors influencing volume change, addition and control of moisture appeared to be the only practicable remedial procedure to consider. Compaction control using optimum moisture on the disturbed layers was also expected to contribute favorably on fill sections. To aid in moisture retention after ponding, the lime-stabilized subgrade was extended to the width of the ponded section. These sections were ponded and stabilized with lime in 1958.

When the construction project of 1957 was conceived, the existing facility consisted of a four-lane divided highway. The southbound section was a concrete pavement completed in 1933, and the northbound pavement was a concrete section completed in 1952. The serviceability of these two sections of pavement was so low that the construction project was to include leveling and overlay of these pavements. This loss in serviceability was not the result of loss in structural capabilities but was attributed primarily to the characteristic volume change in the naturally occurring soils in the area.

Prior to this date, the Portland Cement Association had conducted a brief experiment in the area that indicated that soaking could produce the desired volume change prior to construction (1) and could eliminate the differential vertical movement that was plaguing all construction in the area. McDowell's report (2) on potential vertical rise had correlated very closely with the findings of the Portland Cement Association.

As a result of this close correlation, limited sections of the subgrade were to be ponded prior to construction of the section that was to become the southbound main lanes of the Interstate highway in the area. The plan included construction of an east frontage road with flexible base and penetration surface and the use of the existing southbound lanes as the west frontage road. The existing northbound lanes were to remain as the northbound lanes, and new southbound lanes were to be constructed between the existing surfaces.

Certain areas of the existing ground were to be ponded prior to grading, and then the area was to be graded and the surface of the grading protected by lime stabilizing of the surface of the subgrade. Because funds would not be available to pond the entire area, basic criteria were established for selecting the sites to be ponded. The initial investigation was carried on by personnel of the District Laboratory of Texas Highway Department District 9 at Waco.

Areas to be ponded were selected on the basis of potential vertical rise (PVR) in excess of 1 in. (2.5 cm). The method used to calculate the PVR was generally the same as the present procedure Tex-124-E. According to this method, extensive push-barrel sampling is necessary to determine existing moisture content and soil constants. Because not all of the locations could be or needed to be ponded, it was decided to consider the areas where proposed fill was less than 6 ft (1.8 m).

This was based on the premise that moisture and density of the fills would be controlled so as to minimize swell from the fills themselves and that the surcharge load from 6 ft (1.8 m) of fill and 24 in. (0.6 m) of pavement would be sufficient to restrain swell of most sublayers. Deep cuts were investigated after the material was removed to approximately the proposed grade line. In areas of severe swell, it was found necessary to investigate to a depth of 20 ft (6 m) because the potential swell would not load out above 16 to 18 ft (4.9 to 5.5 m). The contractor was supplied with the limits of these areas, and he then diked and ponded 18 experimental sections in accordance with the plans and specifications, for 30 days, at which time he was allowed to remove the ponds and proceed with the grading in accordance with standard Texas Highway Department procedures.

A typical cross section of both the northbound and southbound main lanes is shown in Figure 2. The proposed southbound main lane (SBML) concrete pavement contained corrugated metal contraction joints on 15-ft (4.6-m) centers and no expansion joints except at bridge ends. The portland cement concrete was designed and constructed to have a minimum 7-day flexural strength of 650 psi (4480 kPa). The SBML probably is stronger structurally than the NBML; however, because all observed distortions causing uncomfortable riding appeared to be a result of heaving, it is doubtful that the structure strength had appreciable influence on the road's performance.

The structural section of the pavement (Figure 2) called for a 6-in. (15-cm) lime-stabilized subgrade, a 5-in. (13-cm) foundation course, and 12 in. (30 cm) of non-reinforced concrete pavement.

Studies of soils with a range in plasticity indexes of 25 to 55 indicated that the triaxial strength according to test method Tex-117-E improved from class 5 to class 1 with the addition of 6 percent hydrated lime by weight. During construction, 25 lb (11 kg) of hydrated lime per square yard (0.8 m^2) or approximately 6 percent lime by weight was added to the subgrade. The PI of the lime-treated soil taken from the road varied from 6 to 21. Preliminary results of unconfined compression tests on 18-day cured specimens containing 6 percent lime varied from 62 to 281 psi (427 to 1937 kPa).

Shoulders were constructed of 6-in. (15-cm) soil cement base using 8 percent cement by volume with a 1½-in. (3.8-cm) type D asphaltic surface. The intention of the specification was to require that the average minimum unconfined compressive strength be not less than 700 psi (4830 kPa) after specimens were moist cured for a period of 7 days.

Observations of pavement performance and moisture content tests were continued over a period of several years. In 1971 the Center for Highway Research entered into a cooperative agreement to write up the data for this project. It was then necessary to find some means of describing the distortion of the original pavement slabs. Because the experimental ponded sections were scattered throughout the length of the project and because pavement performance varied between as well as within sections, it became necessary to evaluate this project thoroughly before conclusions could be formed. Thirteen years had elapsed since the pavement had been placed, so it was necessary to determine the thickness of overlay, moisture contents, and performance records as a basis for evaluating ponding. At this time the pavement had been overlayed with hot-mixed asphaltic concrete and sealed so effectively that it was difficult to conclude whether any one section was better than another. No cracks were evident, so the possibility of making a crack survey was eliminated. Profilometer measurements were made on the entire 8-mile (13-km) project, but because no previous measurements had been made they failed to reveal the past behavior of these pavements. The measurements are on file at the District Laboratory of the Texas Highway Department in Waco.

Because field bump survey observations made during a 7-year period indicated that pavement roughness developed erratically between sections, it was decided to determine the thickness of asphaltic concrete (AC) overlay at regular intervals. After the project was restaked, the thickness of AC overlay was determined at each 100-ft (30.5-m) station for all experimental sections and each opposite station in the older northbound main lanes. Detailed logs of the core thickness, which varied from nearly 2 to more than 19 in. (5 to 48 cm), were made.

COLLECTION AND ANALYSIS OF DATA

Field exploration and testing were started in May 1957. The initial investigation consisted of classification and surface mapping of soil series and pertinent geological factors. Because fills in excess of 6 ft (1.8 m) were to be excluded, the limits of these areas were established. This information, along with test data obtained at all stages of operation, was recorded on a continuous roll profile plot. Thirty-seven locations were selected for detailed testing. One hole was drilled in each location for primary data.

The necessary data obtained from each pilot hole were soil constants, gradation, moisture content, and physical description. Generally, samples were taken every 6 in. (15 cm) in depth. Some deviation from this procedure was necessary because of material changes. After tests were completed, grouping into major units could be accomplished when control factors were in agreement. At the completion of this phase of investigation, 477 samples had been tested.

Calculations of PVR values were made from the pilot hole data from each location as tests were finished. After all locations were checked for PVR, 18 of these indicated swell potential in excess of 1 in. (2.5 cm). All test data were then plotted on the profile roll, and the 18 sections proposed for ponding were established. The sections in cut areas were investigated further after completion of excavation.

The equilibrium or desired moisture content was determined for each section selected for ponding. A detailed explanation of the procedure used in the calculations of desired moisture is given elsewhere (4).

Section 1 was diked and flooded on October 10, 1957. The next section was ponded the following day. These sections were observed and all details evaluated. Full-depth moisture tests were taken after 14, 20, and 24 days of soaking (Figure 3). Test information as well as experience gained from these first sections was essential to the planning of a workable procedure for the remainder of the operation. Moisture tests during the initial ponding indicated that about 30 days' soaking on most of the sections would be adequate to reach equilibrium moisture content. Sections 5 and 7 were ponded for less time because the required change in moisture (from start of soaking to desired content) was not so great as for the average section.

There had been considerable doubt whether the truck-mounted drill would work as a self-propelled unit in ponded areas. This was accomplished with the use of removable wrap-around plated tracks on the tandem rear wheels. With this equipment it was possible to move into a ponded area immediately after the water was drained and proceed with push-barrel sampling.

An approximate time requirement for soaking was established after the first two sections; however, confirmation moisture tests were continued through the remainder of the project. Extended soaking time was continued on a random basis on several sections. Results through 1964 were used in Figure 3. Moisture tests were taken in sections 1 and 9 as late as March 1972. The latest tests (taken at the edge of the concrete pavement) were not considered to be reliable because of the depth [6 to 10 ft (1.8 to 3 m)] of the crack that had developed between the pavement and shoulder. It is very probable that the condition of this joint has had a detrimental effect on the pavement serviceability in several sections.

More than 9,000 moisture tests were taken on this project. Approximately 85 percent of these tests were made during planning and construction. All moisture and soil constant test results are on file in the Waco District Laboratory. A future project of the same length over similar materials could probably be adequately planned and controlled with fewer moisture samples, but a reduction in soil constants (477) would be questionable.

The results of moisture content tests at various depths and at various stages of the project's history for sections 1 and 9 are shown in Figures 3 and 4. The moisture content curves are the averages of many samples taken at 6-in. (15-cm) depth intervals and are believed to represent the range of values in the other sections. The LL and PI values are shown along the right edge of Figures 3 and 4. The moisture content curves shown in Figures 3 and 4 are presented in such a manner as to depict the moisture conditions before, during, and after ponding at depths below the top of the concrete pavement grade. The data shown in Figure 3 are believed to be typical of most of the sections, especially those that performed well, and the curves shown in Figure 4 are believed to be typical of a few sections that performed poorly. Many

Figure 2. Typical highway cross section.

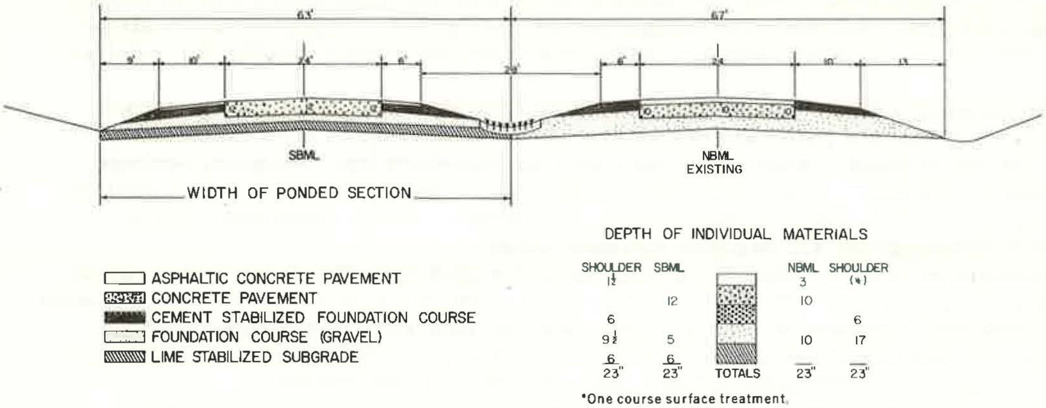


Figure 3. Depth versus moisture prior to and during ponding (section 1).

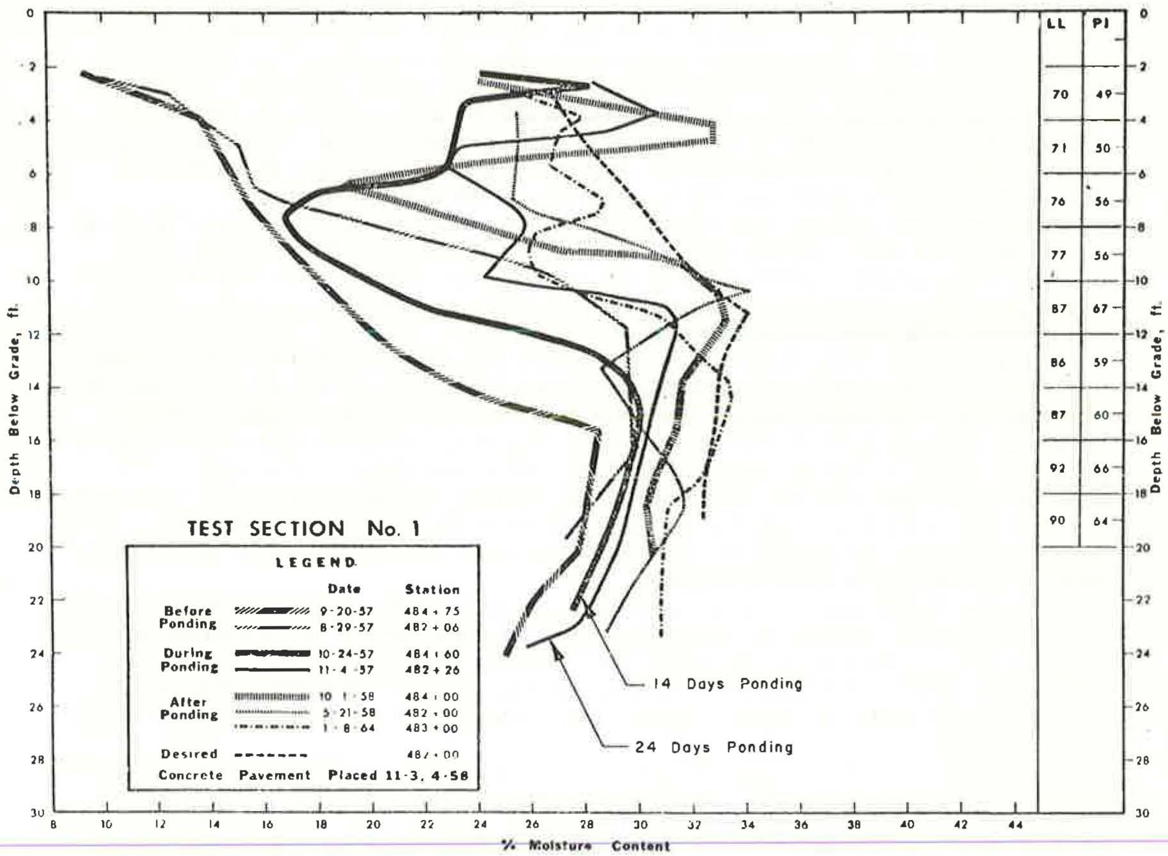


Figure 4. Depth versus moisture prior to and during ponding (section 9).

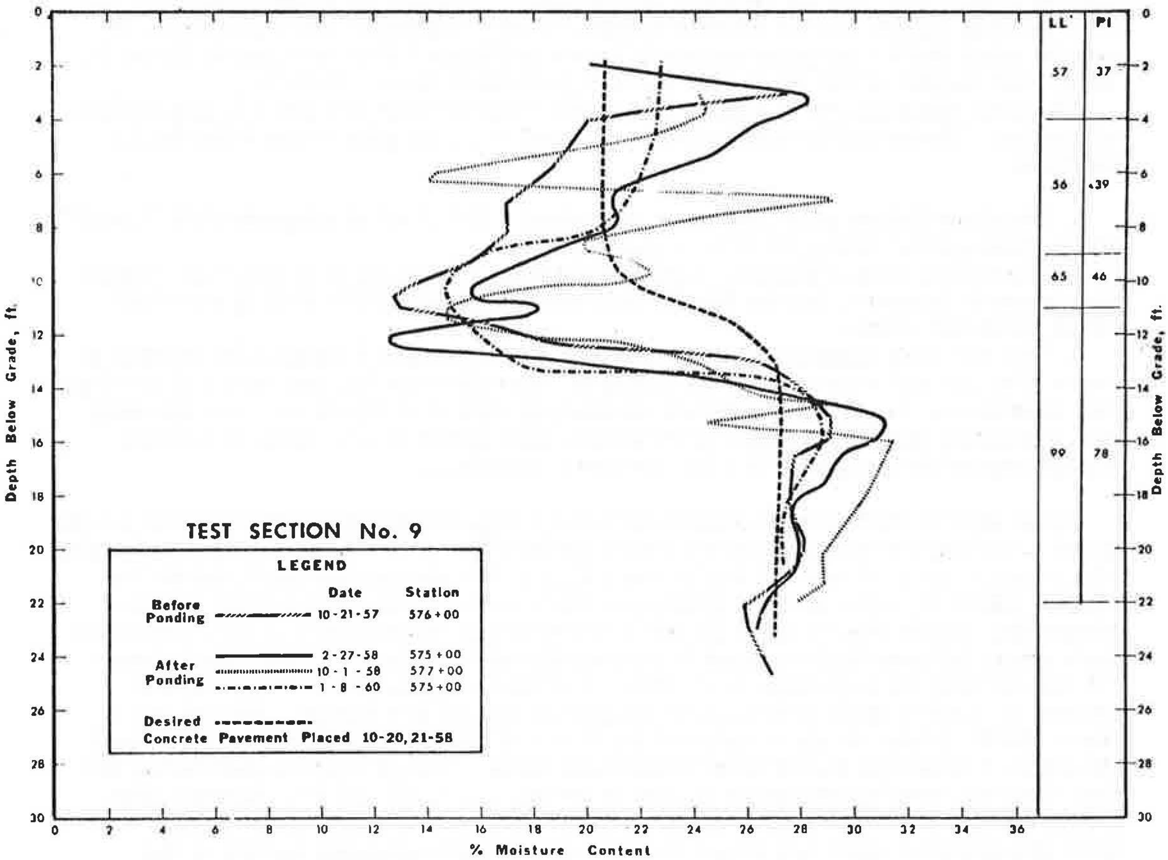


Table 2. Bumps occurring between ponded sections from 1958 to 1965.

Station Numbers	Section Numbers	Number of Bumps	Original Average PVR (in.)	Remarks
459+70.4 to 482+00	So. end to 1	4	2.0	Ponding probably would have prevented bumps.
487+50 to 499+00	1 to 2	1	No data	
505+00 to 512+00	2 to 3	0	No data	
514+00 to 526+00	3 to 4	3	1.0	Ponding probably would have prevented bumps.
529+00 to 532+00	4 to 5	1	No data	
537+00 to 539+00	5 to 6	0	No data	
550+00 to 551+00	7 to 8	0	No data	
557+00 to 572+00	8 to 9	2	0.8	Ponding probably would have prevented bumps.
588+00 to 617+00	9 to 10	4	1.4	Underdrains connected to ditch drop inlets.
620+00 to 629+00	10 to 11	1	No data	
633+00 to 653+00	11 to 12	9	3.2	Ponding probably would have prevented bumps.
657+00 to 666+00	12 to 13	1	0.9	Ponding probably would have prevented bumps.
671+00 to 682+00	13 to 14	0	0.2	
688+00 to 698+00	14 to 15	2	0.0	
709+00 to 744+00	15 to 16	2	0.8	Ponding probably would have prevented bumps.
749+00 to 802+00	16 to 17	0	1.0	
817+00 to 863+00	17 to 18	9	3.8	Ponding probably would have prevented bumps.
870+00 to 889+00	18 to No. end	4	1.9	Ponding probably would have prevented bumps.
Total		43[a]		

Note: 1 in. = 2.5 cm.

[a]38 or 88 percent could have been eliminated by ponding more extensively and/or avoiding the attachment of underdrains to ditch drop inlets.

other samples were taken in less detail in all of the other 18 sections, but for the sake of brevity they are not included in this paper. Usually the areas bounded by the after ponding curves and the desired moisture curve represent swell potential. It may be noted that these areas are much larger in Figure 4 than they are in Figure 3. This could be part of the reason for the poor performance of section 9.

The solid black curves in Figure 3 show the moisture contents after 14 and 24 days of ponding. These and the desired moisture content curve point to the following indications.

1. Moisture did not penetrate more than about 4 ft (1.2 m) of subgrade [6 ft (1.8 m) below finish grade] during 24 days of ponding.
2. During the time of ponding, moisture contents at 16 to 20-ft (4.9 to 6-m) depths also began to increase, leaving the driest areas at a depth of 5 to 10 ft (1.5 to 3 m) below pavement grade.
3. The dot-dash curve shows that moisture contents taken 7 years after ponding (6 years after paving) were slightly in excess of those shown at the conclusion of ponding. The dashed line, for desired moisture content, is located fairly close to the dot-dash line, indicating that the moisture content to be anticipated in clay soils, at various depths, can be determined with a fair degree of accuracy.

From 1958 to 1965 several observations for bumps in both the northbound and southbound main lanes were made for the entire project (Table 2). This was done by driving a passenger car at 60 mph (96 km/h) and noting on the plan profile sheet the station number where the deformations causing uncomfortable riding were located. It is interesting to note that on April 12, 1961, three bumps had occurred in the northbound main lanes, but none had occurred in the southbound main lanes. Subsequent observations for heaving were made until 1965, at which time placement of intermittent patches of overlay made it difficult to accurately record new bumps. After 3 more years (1968), it was decided to place 2 in. (5 cm) of hot-mixed AC overlay throughout the entire 8 miles (13 km) of southbound main lanes. Figure 5 shows that during the first 7 years one bump occurred in each of sections 5, 8, 17, and 18. Another four bumps occurred in section 9. The cross-hatched bars on the right side of Figure 5 show that twice as many bumps per mile occurred in the unponded portion of the southbound main lanes as in the ponded sections. Reference to the plans reveals that sections 9, 17, and 18 are the only ponded sections that have drop inlets connected to perforated underdrains. Section 9 has ditch drop inlets for large drainage areas connected to underdrains. When it rains, a head of water can back up into the underdrains, and during dry weather wide belts of soil can dry out due to evaporation. This makes for extreme fluctuations of moisture and volume change in this section of high volume change soils. It is possible that these bumps might not have occurred if underdrains had not been connected to drop inlets. In this case, there would have been four bumps in all sections or only about one-fourth as many per mile as occurred in the unponded portion. One bump each occurred in sections 17 and 18. These sections were in small drainage areas such as those at underpasses, and the drop inlets were at curbs. If all bumps from ponded sections that have drop inlets connected to underdrains are ruled out, there would be only two bumps left in all sections, and the unponded portion could be said to have between seven and eight times as many bumps per mile as the ponded sections. The reader's interpretation of these findings will depend a great deal on his background and attitude, but the most conservative reader must conclude that ponding was beneficial. The authors believe that it would have been possible to construct large portions of this project so that they would have remained relatively free from heaving.

Because PVR has been used as a basis for determination of areas to be ponded, it was decided to relate this factor to the number of bumps occurring in the unponded portions of the southbound main lanes. Figure 6 shows that the number of bumps can be expected to increase as PVR increases. The heavy dashed line in Figure 6 shows that when PVR exceeds 2 in. (5 cm) a rapid increase in the number of bumps per mile can be expected. Because this chart is based on a 7-year study, it does not clearly

Figure 5. Bumps recorded between 1958 and 1965.

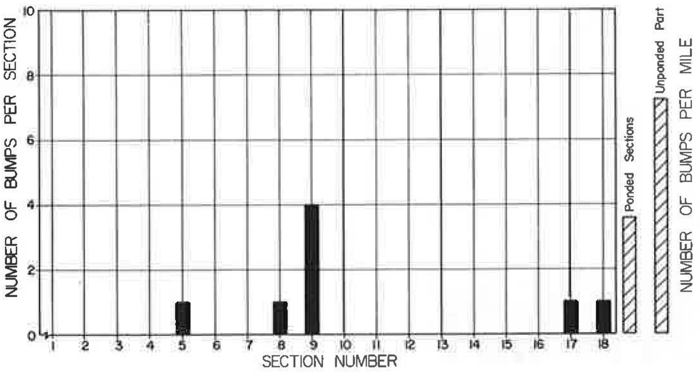


Figure 6. Relation of number of bumps for unponded portion of SBML after 7 years of traffic to average PVR.

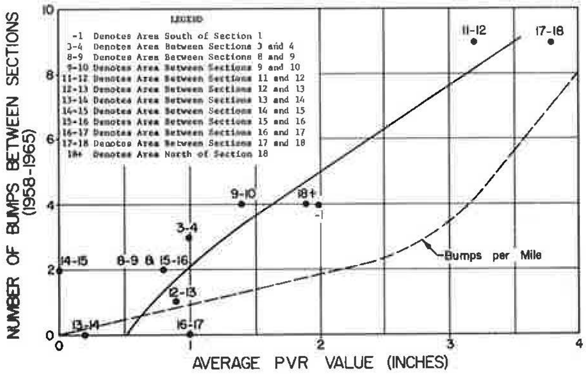


Figure 7. Relation of original PVR to movements as measured by levels.

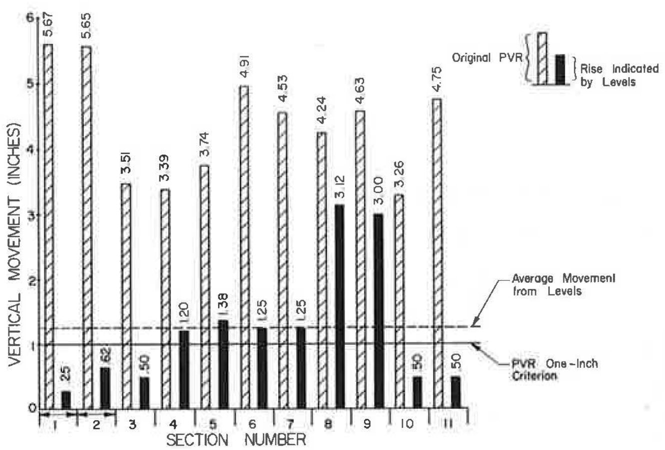


Figure 8. Average thickness of asphaltic concrete overlay (1971).

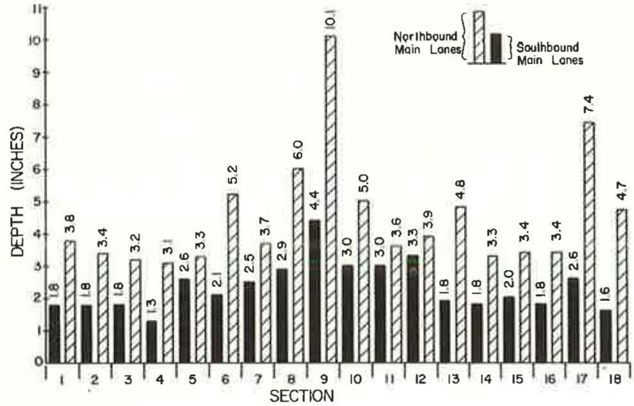


Figure 9. Maximum variation in thickness of asphaltic concrete overlay.

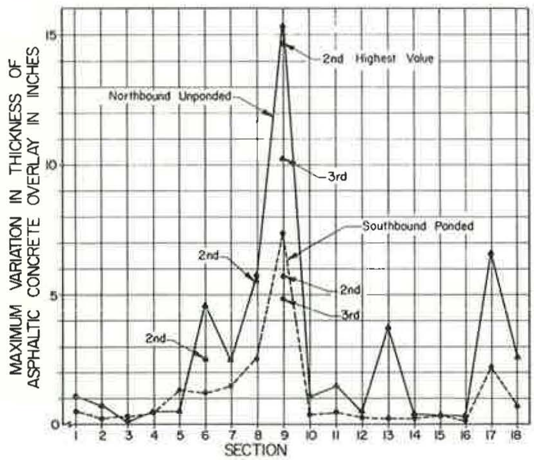
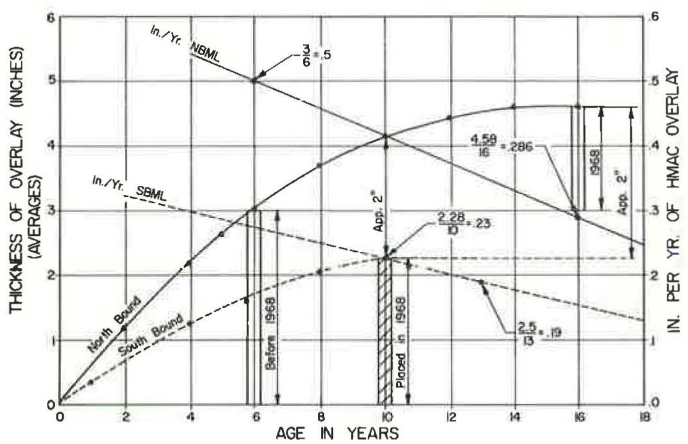


Figure 10. History of hot-mixed asphaltic concrete leveling.



indicate the magnitude of the minimum PVR for design, but the figure is probably somewhere between ½ and 1 in. (1.3 and 2.5 cm).

Figure 7 shows the original PVR values in relation to level measurements of movements recorded in May 1965. Paving grade level notes for ponded sections 1 through 11 were all that could be found in the files. This chart shows the original PVR values, which vary from 3.26 to 5.67 in. (8.28 to 14.4 cm). It may be noted that the average movement as measured from levels is approximately 1¼ in. (3.2 cm), whereas the target for ponding was 1 in. (2.5 cm).

In 1971 it was agreed that probing for thicknesses of AC overlay would be helpful in studying the manner in which the pavement performed. Figure 8 shows the average thickness of overlay for each section. Section 9 (the section with ditch drop inlets connected to underdrains) had the greatest average overlay thicknesses of any section. The overlay thicknesses are much greater for the NBMLs than they are for the SBMLs. This is as expected since the NBMLs are about 6 years older; however, a difference of 3 to 8 in. (7.6 to 20 cm) for several sections seems unusual, to say the least.

Figure 9 shows that the maximum variation in thickness (within sections) of AC overlay of the northbound main lanes is greater than that of the southbound main lanes in nearly all sections. This means that the original slabs of the SBMLs are not out of grade so much as those of the NBMLs. It should be kept in mind that the NBMLs are not the same age as the SBMLs. Points marked 2nd and 3rd represent the second and third highest values of thickness variation. They are presented to show that the peak values are not unusual values.

As stated before, it is difficult to compare overlay thicknesses of the NBMLs and SBMLs because of the difference in their service lives. In an attempt to place these data in their proper perspective, the histogram shown in Figure 10 was prepared. The coefficients for the depth of overlay per year were determined by dividing the thickness of overlay by the age of the road at the time the overlay was placed. Points for the curved lines were determined by multiplying the age in years by the corresponding coefficients taken from the scale on the right edge of the chart. Given that both the northbound and southbound lanes are of adequate structural strength to carry the traffic loads (there were no overload failures), it appears that the use of a combination of ponding and lime stabilization required approximately 2 in. (5 cm) less AC overlay during the first 10 years.

CONCLUDING REMARKS AND RECOMMENDATIONS

Moisture content tests taken before, during, and after ponding indicate the following.

1. Moisture from ponding did not penetrate the subgrade more than 4 ft (1.2 m) downward during a period of 24 days. A study of the horizontal movement of moisture was not made, but in one instance ponding was believed to have caused the tilting of the northbound main lanes. There was a distance of 20 ft (6.1 m) between the edge of the pond and the edge of the portland cement concrete.
2. Moisture contents at depths of 16 to 20 ft (4.9 to 6.1 m) began to increase after a period of several days' ponding and continued to increase all the way up to the 4-ft (1.2-m) level [6 ft (1.8 m) below finished grade] within a period of approximately 24 days. Although no data were taken to prove it, the vertical travel of moisture (up or down) was probably somewhat dependent on elevation of water tables.
3. Tests indicate that moisture contents below the pavement in ponded sections have remained fairly constant for 13 years since placement of the pavement. There has been some fluctuation of moisture contents at various depths, but it is believed that these have not been sufficient to cause severe movement of pavement in most of the ponded sections. Moisture content of samples taken in 1972 below the cement-stabilized shoulders, which had severely cracked away from the edge of the portland cement concrete, was very erratic. Such samples are probably not representative of conditions below the portland cement concrete.
4. The moisture contents found under the pavement at various depths, before the

shoulders cracked away from the concrete, are in fairly close agreement with the desired moisture contents calculated in accordance with the method given in the complete report (3).

The maximum movements measured by profile levels are in general agreement with the movements predicted by use of the potential vertical rise method for after ponding conditions. The PVR method was very useful in helping to select locations for ponding and determining moisture contents required before termination of ponding.

The bump surveys made after the SBML pavement was 7 years old showed that the ponded sections had only one-half as many bumps per mile as the unponded portion of the same lanes. Of a total of eight bumps occurring in all ponded sections, one-half of these occurred in section 9. Strangely enough this is the only section on the project where underdrains were connected to drainage ditch drop inlets that were supposed to handle fairly large drainage areas. If the bumps in this section were excluded from the data, there would have been four times as many bumps per mile in the unponded portion of the project as in the ponded sections.

A study of the overlay leveling applications shows the following:

1. The leveling overlay thickness for the unponded NBML sections is considerably thicker than it is for the ponded sections in the southbound lanes.
2. The roughness of portland cement concrete slabs, as measured by maximum variation in overlay thickness, shows that 14 of the 18 sections contain rougher slabs in the unponded lanes than in the southbound ponded lanes.
3. The validity of conclusions 1 and 2 is in jeopardy because the northbound pavement lanes are 6 years older than the southbound lanes. Accordingly, a histogram of overlay thicknesses was made, and it showed that ponding and lime stabilization of swelling subgrades can be expected to reduce the required depth of AC overlay by approximately 2 in. (5 cm) within the first 10 years of pavement life.

In general, it is concluded that ponding and lime stabilization of subgrade were highly successful in preventing heaving in all sections except the one section where underdrains were connected to ditch surface drainage by use of drop inlets.

A study of the unponded areas between and beyond the ponded sections (Table 2 and Figure 6) indicates that, if ponding had been used more extensively and if underdrains had not been connected to ditch drop inlets, probably only five of the 43 bumps recorded would have occurred in these areas. Table 2 indicates that the higher the PVR is the greater is the number of bumps expected and that if few to no bumps are desired a PVR criterion of $\frac{1}{2}$ in. (1.2 cm) should be used. If this criterion had been followed during construction, only two bumps should have occurred in the ponded sections and five in the remaining portions. If only seven bumps had occurred on the entire length of the SBML, overlaying the entire project with hot-mixed AC probably would not have been necessary for many more than 10 years.

The foregoing conclusions appear to justify the following recommendations.

1. For all subgrades with PI in excess of 35, calculate potential vertical rise values and determine whether ponding of subgrade is necessary and feasible. PVR data should be used to calculate desired moisture contents at various depths below pavement to determine when to cease ponding.
2. The use of ponding should be given serious consideration where a heavy traffic facility is involved and it has been determined that ponding is necessary and feasible.
3. If ponding is used, every effort should be made to prevent evaporative drying before placement of the pavement. One of the most practical ways to accomplish this is by the use of a wide belt of lime-stabilized subgrade. The use of this stabilizer allows the work to proceed before excessive drying takes place. In addition, the lime-treated subgrade helps form a strong working table and if extended widely enough makes an excellent barrier to evaporative drying and shrinking. Various granular materials will also decrease evaporation but usually will not form a strong working table unless placed in very thick layers.

4. Underdrains should be used sparingly in swelling soils, and they should not be connected to drainage ditch drop inlets.

5. For all future projects in swelling soils, ponding should be used more extensively than was done on this project, and its use in conjunction with deep plow mixing of lime should be investigated.

6. Special effort should be made to prevent cracks formed by shoulders shrinking away from the edges of pavement.

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